GRAHAM BABA ARCHITECTS

DATE	30 August 2019		
PROJECT	Shabalin Liborski Residence 856 W Lake Sammamish Parkway NE Bellevue, WA 98008	**FROM**	Ellen Cecil Graham Baba Architects 1507 Belmont Avenue Suite 200 Seattle, WA 98122
TO	City of Bellevue Development Services 450 110th Ave NE Bellevue, WA 98004 425.452.6800	**ARCHITECT'S PROJECT NO**	1853

Project Information

Site Address:	856 W Lake Sammamish Parkway NE
Parcel No:	743050-0480
Land Use Classification:	R-2.5 Residential
Shoreline Overlay District	
Shoreline Residential (SR) environment	
Critical Areas:	Steep Slope Floodplain
Steep Slope Structure Setback:	75' from Toe of Steep Slope
Floodplain:	EL 36.1' and below
Floodplain Structure Setback:	25' from EL 36.1', but Shoreline Setback is more restrictive (see below)
Shoreline Structure Setback:	50' from Ordinary High Water Mark

Project Narrative:

The subject site is an 11,631 square-foot R-2.5 residential lot on the northwestern shore of Lake Sammamish. Vehicular access to the property is via NE Rosemont Place, a paved private road/easement off West Lake Sammamish Parkway NE. NE Rosemont Place traverses the property in a way that separates a steep slope critical area to the west (between W. Lake Sammamish Parkway and NE Rosemont Place) from a less steep, non-critical portion to the east (between NE Rosemont Place and the Lake Sammamish shore.) A 4-car carport exists at the bottom of the steep slope portion of the site, and a single-family residence exists on the east portion of the site toward Lake Sammamish. Because the residence is partially embedded in the hillside, the landscape between the house and NE Rosemont Place is terraced using landscape blocks and is planted with typical residential plantings. Vehicles park in the carport to the west of NE Rosemont Place, and pedestrian access to the residence is via wood stairs to a level approximately eight feet below NE Rosemont. The entire carport and part of the existing residence are located within the 75' Steep Slope Structure Setback.

This submittal proposes areas of work within the 75' Steep Slope: 1) construction of an elevated walkway from NE Rosemont Place to the proposed new residence, and 2) modification of the existing 4-car carport.

1. Elevated Walkway (refer to fig. 1):

The owners want to make their new residence accessible. The simplest and least impactful way to provide access is via a small elevated walkway that spans from Rosemont Place to the new residence, whose entry level will be at approximately the same level. The east portion of the walkway will be supported by the wall of the residence itself, and the west portion will be supported by a small concrete foundation that is underpinned per the attached April 12, 2019 Geotechnical Engineering Study. The floor of the walkway will be open metal grating or spaced wood decking to ensure its permeability. This proposed expansion is the minimum necessary to achieve the intended function of *accessible* access to the residence. Other options are more disruptive: create a

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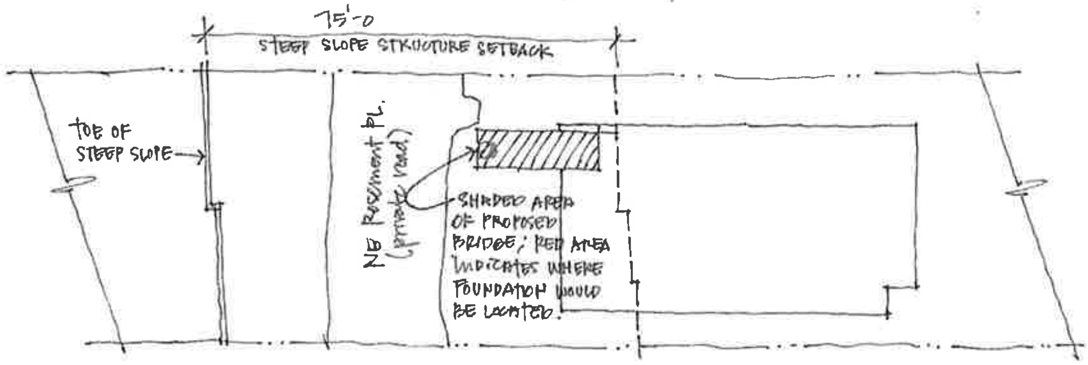
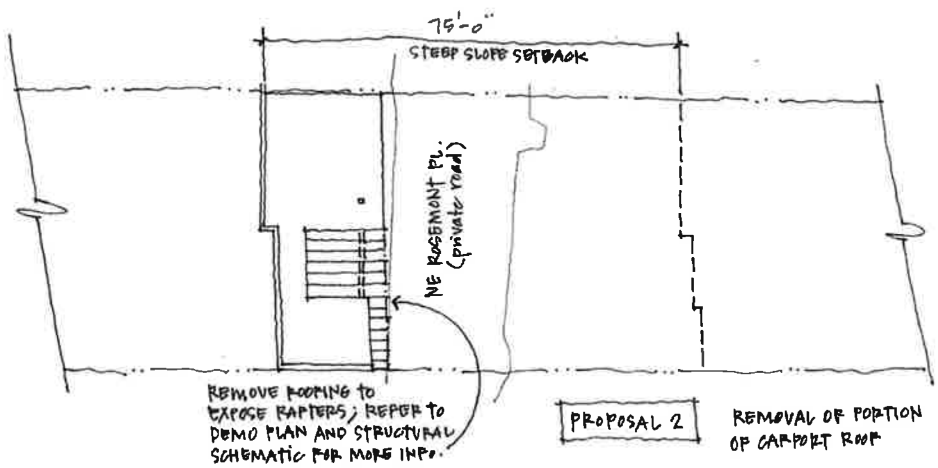
land bridge between Rosemont and the new residence, or move the proposed residence closer to Rosemont, further impinging on the steep slope setback.



PROPOSAL 1: PEDESTRIAN BRIDGE FROM PRIVATE ROAD TO PROPOSED NEW RESIDENCE

2. <u>Modification of carport (refer to fig. 2)</u>:

The owners would like to modify the existing carport by removing approximately 214 sf of roofing and reinforcing the remaining roof framing with tension rods per the attached detail from the structural engineer. The back (west) wall of the carport is at the toe of the steep slope and will remain. The attached sketch describes the scope of the work, and summary letters from the geotechnical and structural engineers recommend strategies and methods for executing this work in a way that will maintain the integrity of the retaining wall without impact to the steep slope.



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Decision criteria from LUC 20.30P.140:

The Director may approve or approve with modifications an application for a Critical Areas Land Use Permit if:
A. The proposal obtains all other permits required by the Land Use Code; It is our intent that all other appropriate permits will be obtained through the Bellevue DSD prior to construction; and
B. The proposal utilizes to the maximum extent possible the best available construction, design and development techniques which result in the least impact on the critical area and critical area buffer; We believe the proposed design has the least impact on the steep slope setback while allowing reasonable use of - and access to- the owners' home; and
C. The proposal incorporates the performance standards of Part 20.25H LUC to the maximum extent applicable; and
D. The proposal will be served by adequate public facilities including streets, fire protection, and utilities; This proposal will not change how the garage or residence are served by existing public facilities; and
E. The proposal includes a mitigation or restoration plan consistent with the requirements of LUC 20.25H.210; except that a proposal to modify or remove vegetation pursuant to an approved Vegetation Management Plan under LUC 20.25H.055.C.3.i shall not require a mitigation or restoration plan; Only the plants directly at the foundation for the pedestrian access bridge will need to be removed. Any other plantings that may need to be temporarily moved before and during access will be replanted to their original state once construction is complete; and
F. The proposal complies with other applicable requirements of this code. (Ord. 5683, 6-26-06, § 27); It is our intent that this proposal comply with all other applicable requirements of this code.

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GEOTECH
CONSULTANTS, INC.

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June 19, 2019

JN 19090

Valeri Liborski and Kira Shabalin
856 West Lake Sammamish Parkway Northeast
Bellevue, Washington 98008

via email: liborski@gmail.com; kshabalin@gmail.com

Subject: **Addendum to Geotechnical Engineering Study**
Proposed New Residence
856 West Lake Sammamish Parkway Northeast
Bellevue, Washington

Reference: *Geotechnical Engineering Study*, same site and project; April 12, 2019; Geotech Consultants, Inc.

Greetings:

As anticipated at the time of our above-referenced *Geotechnical Engineering Study*, the project will go through the Critical Area Land Use Permit (CALUP) process with the City of Bellevue. The western portion of the existing house currently encroaches into the prescriptive 75-foot toe-of-slope structure setback. The new residence will not increase this nonconformity. An elevated footbridge will extend between Northeast Rosemont Place and the front entry of the residence, within the 75-foot prescriptive toe-of-slope structure setback. As a part of the development, we understand that a portion of the roof for the existing carport at the toe of the steep slope would be removed. The existing concrete retaining wall that supports the ground on the west side of the carport, as well as the floor slab of the carport, would remain in place.

Geotechnical considerations for the planned new home and elevated footbridge are discussed in our April 12, 2019 report. These recommendations for foundations, excavations, subsurface drainage, etc. are intended to prevent the planned development from adversely impacting the stability of the steep slope located to the west of Northeast Rosemont Place. The new residence will not encroach closer to the slope than the existing house, and excavation for the western side of the new home will be shored. The planned footbridge would be supported on pipe piles. The use of this type of construction for the footbridge will avoid the more extensive site disturbance and excavation within the 75-foot setback that would be necessary to construct retaining walls to create a flat pedestrian access from Northeast Rosement Place. The only disturbance necessary would be small excavations for the pile caps on which the vertical posts would sit. Minimizing the earthwork for this aspect of the project also has the benefit of reducing the potential for erosion problems on the lakefront lot.

The western side of the existing carport consists of a reinforced concrete wall that retains the cut originally made into the toe of the slope for the carport's construction. This wall is much more substantial than the landscape block walls, rockeries, or unsupported cuts that have been completed over the years on many of the properties along Northeast Rosemont Place to create parking spaces. Removal of a portion of the carport's roof should be viable, but will require consideration of the potential minimal lateral bracing that may be provided by the existing roof.

Prior to removing any portion of the roof structure, helical anchors should be installed through the wall to provide lateral support above the floor slab. These anchors will be torqued into dense, glacially-compressed soil in the core of the hillside, which will: 1) prevent adverse impacts to the stability of the wall and the slope above it, and 2) improve the lateral stability of the retaining wall. During the design phase of the project, following the CALUP process, we would work with the project structural engineer to develop the design criteria for this helical anchor bracing of the wall.

Please contact us if there are any questions regarding this letter, or for further assistance during the design and construction phases of this project.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

MARC R. McGINNIS
STATE OF WASHINGTON
27845
REGISTERED
PROFESSIONAL ENGINEER

06/19/19

Marc R. McGinnis, P.E.
Principal

cc: **Graham Baba Architects** – Jeff King
via email: jeff@grahambaba.com

MRM:kg

GEOTECH
CONSULTANTS, INC.

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August 26, 2019

JN 19090

Valeri Liborski and Kira Shabalin
856 West Lake Sammamish Parkway Northeast
Bellevue, Washington 98008

via email: *liborski@gmail.com*; *kshabalin@gmail.com*

Subject: **Partial Removal of Existing Carport Roof**
856 West Lake Sammamish Parkway Northeast
Bellevue, Washington

Reference: *Geotechnical Engineering Study*, same site and project; April 12, 2019; Geotech Consultants, Inc.
Addendum to Geotechnical Engineering Study, same site and project; June, 19, 2019; Geotech Consultants, Inc.

From our ongoing discussions with the project team, we understand that less of the roof of the existing western carport is proposed to be removed than was anticipated at the time of our June 19, 2019 letter. In order to accomplish this removal, the project structural engineer (Lund Opsahl) has designed a system of bracing utilizing the existing foundations. This bracing system is intended to maintain any lateral restraint for the western retaining wall that may currently be provided by the portion of the roof that will be removed. This would avoid the need to install lateral ground anchors, as were discussed in our previous letter.

From a geotechnical standpoint, if the revised bracing provides the same restraint for the western wall, the removal of a portion of the roof will not adversely impact the stability of the slope above, on the site or the surrounding lots.

Please contact us if there are any questions regarding this letter, or for further assistance during the design and construction phases of this project.

Respectfully submitted,
GEOTECH CONSULTANTS, INC.

MARC R. McGINNIS
STATE OF WASHINGTON
27845
REGISTERED
PROFESSIONAL ENGINEER
08/26/19

Marc R. McGinnis, P.E.
Principal

cc: **Graham Baba Architects** – Ellen Cecil
via email: *ellen@grahambaba.com*

MRM:kg

GEOTECH
CONSULTANTS, INC.

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April 12, 2019

JN 19090

Valeri Liborski and Kira Shabalin
856 West Lake Sammamish Parkway Northeast
Bellevue, Washington 98008

via email: *liborski@gmail.com*; *kshabalin@gmail.com*

Subject: **Transmittal Letter – Geotechnical Engineering Study**
Proposed New Residence
856 West Lake Sammamish Parkway Northeast
Bellevue, Washington

Greetings;

Attached to this transmittal letter is our geotechnical engineering report for the proposed new residence to be constructed in Bellevue. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide recommendations for general earthwork and design considerations for foundations, retaining walls, subsurface drainage, and temporary excavations and shoring. This work was authorized by your acceptance of our proposal, P-10283, dated January 28, 2019.

The attached report contains a discussion of the study and our recommendations. Please contact us if there are any questions regarding this report, or for further assistance during the design and construction phases of this project.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

Marc R. McGinnis, P.E.
Principal

cc: **Graham Baba Architects** – Ellen Cecil
via email: *ellen@grahambaba.com*

MKM/MRM:kg

GEOTECHNICAL ENGINEERING STUDY
Proposed New Residence
856 West Lake Sammamish Parkway Northeast
Bellevue, Washington

This report presents the findings and recommendations of our geotechnical engineering study for the site of the proposed new residence to be located in Bellevue.

We were provided with preliminary plans and a topographic map. Graham Baba Architects developed these plans, which are dated January 15, 2019 and Site Surveying, Inc. developed the topographic map, which is dated August 20, 2018. Based on these plans and conversations with Graham Baba Architects, we understand that the proposed development for the site will include first demolishing the existing, older home. The site will then be redeveloped with a new, larger residence. The residence will be two stories in height and will contain a basement that daylights to the east. The footprint of the new residence is shown to not extend any further westward than the existing house footprint, but it will extend eastward to the shoreline setback line established on the preliminary site plan. Several different options for potential designs have been presented as part of the preliminary plan set, with interior layouts changing, and foundation layouts differing between each option. Decks are proposed at each of the above-grade floors and a patio is shown extending from the daylight basement level. An elevated pedestrian bridge is also proposed, providing access from Northeast Rosemont Place to the upper floor of the residence. No alterations are proposed to be made to the existing upslope carport that lies on the western side of Northeast Rosemont Place.

Based on these preliminary plans, excavations in excess of 10 feet will likely be needed to reach the basement foundation level and setbacks of 10 and 5 feet are shown from the north and south property lines, respectively. The western edge of the proposed residence as well as the existing house is shown to be located within the 75-foot default toe-of-slope buffer from the base of the adjacent western steep slope. Along with the carport, no disturbance of the steep slope is planned.

If the scope of the project changes from what we have described above, we should be provided with revised plans in order to determine if modifications to the recommendations and conclusions of this report are warranted.

SITE CONDITIONS

SURFACE

The Vicinity Map, Plate 1, illustrates the general location of the site on the western side of Lake Sammamish. The rectangular shaped site has approximate dimensions of 49 feet in the north-south direction, and 232 to 241 feet in the east-west direction. The site is bounded to the north and south by newer, large single-family residences, to the east by Lake Sammamish, and to the west by West Lake Sammamish Parkway Northeast. Both adjacent residences have basements with foundations that appear to step down with the changing topography from Northeast Rosemont Place to Lake Sammamish. These terraces, especially for the southern residence, do not exactly match the topography of the subject site, and the southern residence is set several feet lower than the site in places. The northern adjacent property is set at a slightly higher elevation than the subject site.

The site is currently developed with an older, two-story house located in the rough center of the property. The lower level of the house consists of a basement that daylights to the west. An elevated deck extends off the eastern side of the house at the main, upper level. Access to the house is provided via a stairway extending off Northeast Rosemont Place and a detached carport provides parking along the western side of that private road. Several small outbuildings consisting of a small shed, outdoor bar area, and boathouse are located across the remainder of the eastern half of the lot, and several walkways and patios cover the eastern, lower portion of the lot. A small drainage stream is situated on the southeastern portion of the site, flowing down to the lake. The remainder of the lot is landscaped, with grass covering the remainder of the eastern portion of the yard, and terraced block walls covering the area to the west of the house.

The grade across the property slopes downward from west to east, with a total elevation change of 88 feet across the site. Much of this elevation change occurs between West Lake Sammamish Parkway Northeast and Northeast Rosemont Place, where the slope drops steeply at an inclination of approximately 73 to 89 percent. This steep slope terminates at the backside of the carport retaining walls located on the western side of the Northeast Rosemont Place right-of-way. The grade flattens out across Rosemont Place before dropping moderately downward across three terraced block retaining walls to the western side of the house. The site grade transitions between the main level and lower level by the eastern perimeter of the house, and continues to slope moderately to the tops of two approximately 4 foot tall concrete retaining walls. The grade again flattens out across a paver patio area before dropping again down another 4 feet to the boathouse and dock.

Research conducted on the City of Bellevue GIS files indicates that the entire western slope extending downward from West lake Sammamish Parkway Northeast to Northeast Rosemont Place is designated as a Steep Slope Area. Based on the provided topographic survey, this slope meets the criteria for a Steep Slope Area, with an elevation change of approximately 48 feet at an inclination of 73 to 89 percent across the slope area.

SUBSURFACE

The subsurface conditions were explored by drilling borings at the approximate locations shown on the Site Exploration Plan, Plate 2. Our exploration program was based on the proposed construction, anticipated subsurface conditions and those encountered during exploration, and the scope of work outlined in our proposal.

The borings were drilled on March 28, 2019 using a portable Acker drill. This drill system utilizes a small, gasoline-powered engine to advance a hollow-stem auger to the sampling depth. Samples were taken at approximate 2.5 and 5-foot intervals with a standard penetration sampler. This split-spoon sampler, which has a 2-inch outside diameter, is driven into the soil with a 140-pound hammer falling 30 inches. The number of blows required to advance the sampler a given distance is an indication of the soil density or consistency. A geotechnical engineer from our staff observed the drilling process, logged the test borings, and obtained representative samples of the soil encountered. The Test Boring Logs are attached as Plates 3 through 5.

Soil Conditions

Boring 1 was drilled on the western side of the existing house atop the first terrace of the block retaining walls. Beneath the ground surface, approximately 13 feet of loose, wet fill soils were encountered. Beneath the fill, native silt and very silty sand were encountered.

This soil was observed to contain compressed layers of organics, and was in a dense to very dense state, extending to the base of the boring at a depth of 24 feet. The dense to very dense soil has been glacially compressed.

Boring 2 was conducted on the eastern, lower portion of the site near the proposed eastern limit of the new residence. Approximately 7 feet of fill was encountered beneath the ground surface, and was underlain by native silty sand, gravel, and very gravelly silty sand with cobbles. The native soil was observed to be in a loose and medium-dense state. The boring was terminated at a depth of 13 feet due to refusal on heavy cobbles.

Boring 3 was conducted to the north of Boring 2. Beneath the ground surface, approximately 7 feet of loose fill was encountered. Native, medium-dense gravelly sand with cobbles was encountered beneath the fill, extending to a depth of 11.5 feet. Very stiff silt was encountered beneath the sand, and was also observed to contain small layers of compressed organics. Dense, gravelly, silty sand was encountered beneath the silt at a depth of 16 feet where auger refusal was met at 16.5 feet.

Obstructions in the form of large cobbles and rocks were revealed by our explorations in Borings 2 and 3. Debris, buried utilities, and old foundation and slab elements are commonly encountered on sites that have had previous development.

Groundwater Conditions

Perched groundwater seepage was observed at a depth of 7.5 to 13 feet and from 17 to 21 feet in Boring 1. Groundwater seepage was also observed beneath 22 feet in Boring 1 and below 2.5 feet in Borings 2 and 3 during drilling. The borings were left open for only a short time period. Therefore, the seepage levels on the logs represent the location of transient water seepage and may not indicate the static groundwater level. Groundwater levels encountered during drilling can be deceptive, because seepage into the boring can be blocked or slowed by the auger itself.

It should be noted that groundwater levels vary seasonally with rainfall and other factors. We anticipate that groundwater could be found in more permeable soil layers and between the looser near-surface soil and the underlying denser soil.

The stratification lines on the logs represent the approximate boundaries between soil types at the exploration locations. The actual transition between soil types may be gradual, and subsurface conditions can vary between exploration locations. The logs provide specific subsurface information only at the locations tested. If a transition in soil type occurred between samples in the borings, the depth of the transition was interpreted. The relative densities and moisture descriptions indicated on the boring logs are interpretive descriptions based on the conditions observed during drilling.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

THIS SECTION CONTAINS A SUMMARY OF OUR STUDY AND FINDINGS FOR THE PURPOSES OF A GENERAL OVERVIEW ONLY. MORE SPECIFIC RECOMMENDATIONS AND CONCLUSIONS ARE CONTAINED IN THE REMAINDER OF THIS REPORT. ANY PARTY RELYING ON THIS REPORT SHOULD READ THE ENTIRE DOCUMENT.

The borings conducted for this study encountered dense silt and silty sand at depths ranging from 13 to 16 feet across the site. All new foundation loads need to bear on, or into the dense, competent soil, in order to prevent excessive post-construction settlement. Based on the borings conducted for this study, and the preliminary house layout options presented to us, it appears that the excavations needed to reach suitable bearing soil along, or within the residence footprint would not be feasible costly. Considering this, it is our professional opinion that deep foundations consisting of driven pipe piles should be utilized to provide suitable foundation support for the new residence. an expanded discussion can be found in the ***Pipe Piles*** section of this report. We also recommend that any settlement sensitive structures, such as decks, stairs and cosmetic site features be supported on pipe piles. Furthermore, we recommend that the new building floors consist of either a structural slab that is designed to span between pile supported foundations, or a framed floor over a crawlspace.

The adjacent houses and structures are likely supported on conventional foundations that bear on compressible soils. As a result, it is likely that they have undergone excessive settlement already. There is always some risk associated with demolition and foundation construction near structures such as this. It is imperative that unshored excavations do not extend below a 3:1 (Horizontal:Vertical) imaginary bearing zone sloping downward from existing footings and retaining walls. Contractors working on the demolition and construction of the new residence must be cautioned to avoid strong ground vibrations, which could cause additional settlement in the neighboring foundations. During demolition, strong pounding on the ground with the excavator, which is often used to break up debris and concrete, should not occur. Large equipment and vibratory compactors should not be used close to the property lines. Additionally, in order to protect yourselves from unsubstantiated damage claims from the adjacent owners, 1) the existing condition of the foundation should be documented before starting demolition, and 2) the footings should be monitored for vertical movement during the demolition, excavation, and construction process. These are common recommendations for projects located close to existing structures that may bear on loose soil and have already experienced excessive settlement. We can provide additional recommendations for documentation and monitoring of the adjacent structures, if desired.

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

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

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

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

While no basement floor elevations have been provided at the time of writing this report, the new foundations for the residence are shown at a slightly lower elevation than the existing house basement slab. Based on the soil encountered in our borings, and the soil that will be excavated during construction, an inclination no steeper than a 1.5:1 (Horizontal:Vertical) is appropriate for temporary excavations. No vertical cuts should be made on the property lines, or near any settlement sensitive structure and unshored cuts should not extend beneath a 3:1 (H:V) line from any adjacent foundations, unless the foundations are underpinned. The foundation elevations for both the northern and southern adjacent residences as well as the perimeter site walls and features will need to be determined prior to construction to determine if adjacent foundations will need to be underpinned, and if additional surcharge loads will need to be added to the shoring design.

It appears that the excavation required for the western portion of the residence will be reasonably extensive to reach the basement foundations, and that the proposed foundations may extend deeper than the level of the existing basement, which would undermine the western basement retaining wall. It does not appear that excavation easements will be able to be obtained due to the limited width of the site and proximity of the surrounding structures to the planned excavation. Temporary shoring in the form of soldier piles will likely be needed along the northwestern, western, and southwestern portion of the residence footprint to facilitate excavation and retain the upper terraced block wall slope leading to Northeast Rosemont Place. Less aggressive shoring systems, such as ecology block walls, will likely not be suitable, due to the loose soil conditions and the presence of groundwater. Further recommendations are presented in the ***Temporary Shoring*** section of this report. We do not anticipate that the existing basement wall can be used as a temporary shoring wall at this time. This is due to the unknowns of what soil the foundation bears on, the construction of the wall, the ability of the wall to withstand the upslope soil pressure without the use of extensive bracing, and the potential for the new residence to be founded beneath the existing basement foundation.

An elevated entry walkway is proposed to provide access from Northeast Rosemont Place to the upper floor of the new residence. The excavation for the new walkway will likely be minimal outside of a small excavation required for foundation construction and site grades will most likely not be significantly modified within this area. Due to the loose soils encountered, we recommend that the walkway be supported on pipe piles that are driven to refusal in the underlying dense soils.

The steep slope on the west portion of the property is underlain by competent, glacially-compressed soils. Shallow landslides affecting the near-surface few feet of weathered soils have occurred on this slope, but it does not have a history of deeper slides. The planned residence will be located across Northeast Rosemont Place from the steep, western slope. As a result, the planned development should have no adverse impacts. Also, the existing carport, and the distance between the proposed house and the slope should be sufficient to protect the residence from damage in the event of future shallow slides on the steep slope.

The basement of the new residence will be excavated below a perched groundwater layer and beneath the groundwater table on its lower, eastern side. Excavation dewatering should be expected by the contractor. The water pumped from the excavation must be free of silt to be discharged toward Lake Sammamish or into a storm sewer. Silty water may need to be pumped to a temporary holding tank, such as a Baker tank, to be hauled off site. We recommend installing an underslab drainage system beneath the basement slab of the new residence. This system would consist of a layer of clean crushed rock beneath the interior structural slab or crawlspace. The rock layer should be at least 12 inches thick and contain 4-inch diameter, perforated PVC pipes at no more than 15-foot center to center spacings. The entire rock layer and pipe system should be covered with a thick vapor retarder/barrier. The perforated pipes should tie int the exterior footing drains. A waterproof vapor barrier, rather than a vapor retarder, should be planned below the slab, as added protection against potential water problems in the basement. The ***Drainage Considerations*** section of this report contains an expanded discussion of our subsurface drainage recommendations.

The erosion control measures needed during the site development will depend heavily on the weather conditions that are encountered during the site work. The location of the site on the shore of Lake Sammamish will make proper erosion control implementation important to prevent adverse impacts to the lake. However, we have been associated with numerous waterfront projects that have avoided siltation of the lake and surrounding properties by exercising care and being pro-active with the maintenance and potential upgrading of the erosion control system through the entire construction process. One of the most important considerations, particularly during wet weather, is to immediately cover any bare soil to prevent accumulated water or runoff from the work area from becoming silty in the first place. Silty water cannot be discharged to the lake, so a temporary holding tank should be planned for wet weather earthwork. A wire-backed silt fence bedded in compost, not native soil or sand, should be erected as close as possible to the planned work area, and the existing vegetation between the silt fence and the lake left in place. Rocked construction access and staging areas should be established wherever trucks will have to drive off of pavement, in order reduce the amount of soil or mud carried off the property by trucks and equipment. It will also be important to cap any existing drain lines found running toward the lake until excavation is completed. This will reduce the potential for silty water finding an old pipe and flowing into the lake. Covering the base of the excavation with a layer of clean gravel or rock is also prudent to reduce the amount of mud and silty water generated. Utilities reaching between the house and the lake should not be installed during rainy weather, and any disturbed area caused by the utility installation should be minimized by using small equipment. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Soil stockpiles should be minimized. Following rough grading, it may be necessary to mulch or hydroseed bare areas that will not be immediately covered with landscaping or an impervious surface.

The drainage and/or waterproofing recommendations presented in this report are intended only to prevent active seepage from flowing through concrete walls or slabs. Even in the absence of active seepage into and beneath structures, water vapor can migrate through walls, slabs, and floors from the surrounding soil, and can even be transmitted from slabs and foundation walls due to the concrete curing process. Water vapor also results from occupant uses, such as cooking, cleaning, and bathing. Excessive water vapor trapped within structures can result in a variety of undesirable conditions, including, but not limited to, moisture problems with flooring systems, excessively moist air within occupied areas, and the growth of molds, fungi, and other biological organisms that may be harmful to the health of the occupants. The designer or architect must consider the potential vapor sources and likely occupant uses, and provide sufficient ventilation, either passive or mechanical, to prevent a build up of excessive water vapor within the planned structure.

As with any project that involves demolition of existing site buildings and/or extensive excavation and shoring, there is a potential risk of movement on surrounding properties. This can potentially translate into noticeable damage of surrounding on-grade elements, such as foundations and slabs. However, the demolition, shoring, and/or excavation work could just translate into *perceived* damage on adjacent properties. Unfortunately, it is becoming more and more common for adjacent property owners to make unsubstantiated damage claims on new projects that occur close to their developed lots. Therefore, we recommend making an extensive photographic and visual survey of the project vicinity, prior to demolition activities, installing shoring, and/or commencing with the excavation. This documents the condition of buildings, pavements, and utilities in the immediate vicinity of the site in order to avoid, and protect the owner from, unsubstantiated damage claims by surrounding property owners. Additionally, any adjacent structures should be monitored during demolition and construction to detect soil movements. To monitor their performance, we recommend establishing a series of survey reference points to measure any horizontal deflections of the shoring system. Control points should be established at a distance well away from the walls and slopes, and deflections from the reference points should be measured throughout construction by survey methods.

Geotech Consultants, Inc. should be allowed to review the final development plans to verify that the recommendations presented in this report are adequately addressed in the design. Such a plan review would be additional work beyond the current scope of work for this study, and it may include revisions to our recommendations to accommodate site, development, and geotechnical constraints that become more evident during the review process.

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

CRITICAL AREA REPORT COMPONENTS

The following are our replies to specific items in the Bellevue Land Use Code (LUC 20.25H.125 and 20.25H.145) that are related to steep slope performance standards and Critical Areas Report (CAR) requirements.

LUC 20.25H.125:

A. We anticipate that the project will include minimal re-grading or altering of the existing topography outside of the building footprints. However, this will not occur near the toe of the Steep Slope.
B. The proposed residence will be located on the eastern portion of the property, away from the steep slope that covers the western half of the site. It appears that the toe of the western steep slope was oversteepened when Northeast Rosemont Place was cut into the original ground surface. This toe-of-slope cut is protected by the rear foundation wall of the existing carport. The proposed new residence will be located within the existing western bounds of the existing house, and no disturbance to the steep slope will occur as a result of the proposed development. The new proposed elevated walkway will be located closer to the western toe of the slope, and will have a negligible impact on the stability of the western steep slope, as the only excavations will be needed to install pipe piles that will be driven into the dense core of the site and surrounding slope.
C. The recommendations presented in this report are intended to prevent the planned development from adversely impacting the stability of the neighboring properties. This work will not necessitate increased buffers on the surrounding lots.
D. We recommend any fill placed on the subject site be retained by an engineered retaining wall bearing on pipe piles that are driven to refusal into the underlying dense soils.
E. While the proposed development is in the early planning stages and plans have not yet been developed, minimizing impervious areas downslope of the Steep Slope will have no positive or negative benefit with regard to stability of the slope.
F. Although detailed plans have not been provided to us, we recommend any fill placed on the site be retained by an engineered retaining wall as discussed above. No grading Is planned to occur along the toe of the Steep Slope. We should be notified if this changes in the future.

G. We anticipate that the western perimeter foundation of the proposed residence will be used to retain the soil directly upslope of it. Rockeries or landscape walls are expected to be minimal.
H. The only planned development within the Steep Slope critical area buffer is the proposed entry walkway and western edge of the proposed residence. As previously discussed, no modification to the toe of the nearby steep slope will occur.
I. Deck structures are not expected in the Steep Slope area.
J. A restoration plan for the development area will be included as a part of the permit application for this project.

LUC 20.25H.145:

A. As discussed above, the proposed development will not increase the geologic hazard to either the surrounding properties or the site itself, including the Steep Slope. The proposed residence and entryway will not impact, or modify the toe of the adjacent steep slope that appears to have been modified during the construction of Northeast Rosemont Place.
B. The proposed work will not adversely impact other critical areas if completed in general accordance with our recommendations and the approved drawings.
C. The recommendations of this report are intended to mitigate the risks posed by the Steep Slope to a level that would exist if the critical area was not modified.
D. The recommendations of this report are intended to prevent the planned development from adversely impacting stability of the critical areas, and to make the completed project safe under the anticipated surface and subsurface conditions.
E. This *Geotechnical Engineering Study* follows the guidelines of the City of Bellevue submittal requirements for geotechnical reports.
F. The planned development should comply with our recommendations.
G. To the best of our knowledge, the planned work is not expected to adversely impact habitat associated with species of local importance.

SEISMIC CONSIDERATIONS

In accordance with the International Building Code (IBC), the site class within 100 feet of the ground surface is best represented by Site Class Type D (Stiff Soil). As noted in the USGS website, the mapped spectral acceleration value for a 0.2 second (S_s) and 1.0 second period (S_1) equals 1.3g and 0.49g, respectively.

The IBC and ASCE 7 require that the potential for liquefaction (soil strength loss) during an earthquake be evaluated for the peak ground acceleration of the Maximum Considered Earthquake (MCE), which has a probability of occurring once in 2,475 years (2 percent probability of occurring in a 50-year period). The MCE peak ground acceleration adjusted for site class effects (F_{PGA}) equals 0.52g. The upper soils beneath the site are susceptible to seismic liquefaction under the ground motions of the MCE due to their loose nature and the presence of near-surface groundwater. However, the competent underlying soils that the pipe piles will be driven into are not susceptible to seismic liquefaction under the ground motions of the MCE due to their dense nature. This should provide mitigation against the potential for foundation collapse in the event of seismic liquefaction.

PIPE PILES

Three- or 4-inch-diameter pipe piles driven with a 850- or 1,100- or 2,000-pound hydraulic jackhammer to the following final penetration rates may be assigned the following compressive capacities.

INSIDE PILE DIAMETER	FINAL DRIVING RATE (850-pound hammer)	FINAL DRIVING RATE (1,100-pound hammer)	FINAL DRIVING RATE (2,000-pound hammer)	ALLOWABLE COMPRESSIVE CAPACITY
3 inches	10 sec/inch	6 sec/inch	2 sec/inch	6 tons
4 inches	16 sec/inch	10 sec/inch	4 sec/inch	10 tons

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

As a minimum, Schedule 40 pipe should be used. The site soils are located near Lake Sammamish and will be driven below the groundwater table. We recommend that corrosion protection such as galvanizing be utilized for the pipe piles.

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

Lateral loads due to wind or seismic forces may be resisted by passive earth pressure acting on the vertical, embedded portions of the foundation. For this condition, the foundation must be either poured directly against relatively level, undisturbed soil or be surrounded by level compacted fill. We recommend using a passive earth pressure of 250 pounds per cubic foot (pcf) for this resistance. If the ground in front of a foundation is loose or sloping, the passive earth pressure given above will not be appropriate. We recommend a safety factor of at least 1.5 for the foundation's resistance to lateral loading, when using the above ultimate passive value.

The City of Bellevue recently adopted the City of Seattle's Director's Rule 10-2009 regarding small-diameter pipe piles. Seattle Director's Rule 10-2009 contains several prescriptive requirements related to the use of pipe piles having a diameter of less than 10 inches. Under Director's Rule 10-2009, load tests are required on 3 percent of the installed piles up to a maximum of 5 piles, with a minimum of one pile load test on each project. Additionally, full-time observation of the pile installation by the geotechnical engineer-of-record is required by Director's Rule 10-2009.

FOUNDATION AND RETAINING WALLS

Retaining walls backfilled on only one side should be designed to resist the lateral earth pressures imposed by the soil they retain. The following recommended parameters are for walls that restrain <u>level</u> backfill:

PARAMETER	VALUE
Active Earth Pressure *	
Western Basement Wall	50 pcf
Other Walls	40 pcf
Passive Earth Pressure	250 pcf
Soil Unit Weight	130 pcf

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

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

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

The values given above are to be used to design only permanent foundation and retaining walls that are to be backfilled, such as conventional walls constructed of reinforced concrete or masonry. It is not appropriate to use the above earth pressures and soil unit weight to back-calculate soil strength parameters for design of other types of retaining walls, such as soldier pile, reinforced earth, modular or soil nail walls. We can assist with design of these types of walls, if desired.

The passive pressure given is appropriate only for a shear key poured directly against undisturbed native soil, or for the depth of level, well-compacted fill placed in front of a retaining or foundation wall. The values for friction and passive resistance are ultimate values and do not include a safety factor. Restrained wall soil parameters should be utilized the wall and reinforcing design for a distance of 1.5 times the wall height from corners or bends in the walls, or from other points of restraint. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

<u>*Wall Pressures Due to Seismic Forces*</u>

The surcharge wall loads that could be imposed by the design earthquake can be modeled by adding a uniform lateral pressure to the above-recommended active pressure. The recommended surcharge pressure is 8**H** pounds per square foot (psf), where **H** is the design retention height of the wall. Using this increased pressure, the safety factor against sliding and overturning can be reduced to 1.2 for the seismic analysis.

Retaining Wall Backfill and Waterproofing

Backfill placed behind retaining or foundation walls should be coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. A minimum 12-inch width of free-draining gravel and drainage composite similar to Miradrain 6000 should be placed against the backfilled retaining walls. The gravel and drainage composites should be hydraulically connected to the foundation drain system. Free-draining backfill should be used for the entire width of the backfill where seepage is encountered. For increased protection, drainage composites should be placed along cut slope faces, and the walls should be backfilled entirely with free-draining soil. The later section entitled ***Drainage Considerations*** should also be reviewed for recommendations related to subsurface drainage behind foundation and retaining walls.

The purpose of these backfill requirements is to ensure that the design criteria for a retaining wall are not exceeded because of a build-up of hydrostatic pressure behind the wall. Also, subsurface drainage systems are not intended to handle large volumes of water from surface runoff. The top 12 to 18 inches of the backfill should consist of a compacted, relatively impermeable soil or topsoil, or the surface should be paved. The ground surface must also slope away from backfilled walls at one to 2 percent to reduce the potential for surface water to percolate into the backfill.

Water percolating through pervious surfaces (pavers, gravel, permeable pavement, etc.) must also be prevented from flowing toward walls or into the backfill zone. Foundation drainage and waterproofing systems are not intended to handle large volumes of infiltrated water. The compacted subgrade below pervious surfaces and any associated drainage layer should therefore be sloped away. Alternatively, a membrane and subsurface collection system could be provided below a pervious surface.

It is critical that the wall backfill be placed in lifts and be properly compacted, in order for the above-recommended design earth pressures to be appropriate. The recommended wall design criteria assume that the backfill will be well-compacted in lifts no thicker than 12 inches. The compaction of backfill near the walls should be accomplished with hand-operated equipment to prevent the walls from being overloaded by the higher soil forces that occur during compaction. The section entitled ***General Earthwork and Structural Fill*** contains additional recommendations regarding the placement and compaction of structural fill behind retaining and foundation walls.

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

BUILDING FLOORS

The building floors can be constructed as either structural slabs designed to span between the pile supported foundations without any reliance on soil bearing, or as a framed floor over a crawlspace. Even where the exposed soils appear dry, water vapor will tend to naturally migrate upward through the soil to the new constructed space above it. This can affect moisture-sensitive flooring, cause imperfections or damage to the slab, or simply allow excessive water vapor into the space above the slab. The basement slab or crawl space surface should be underlain an underslab drainage system, as discussed in the ***General*** section, and depicted on Plate 7.

As noted by the American Concrete Institute (ACI) in the *Guides for Concrete Floor and Slab Structures*, proper moisture protection is desirable immediately below any on-grade slab that will be covered by tile, wood, carpet, impermeable floor coverings, or any moisture-sensitive equipment or products. ACI recommends a minimum 10-mil thickness vapor retarder for better durability and long term performance than is provided by 6-mil plastic sheeting that has historically been used. A vapor retarder is defined as a material with a permeance of less than 0.3 perms, as determined by ASTM E 96. It is possible that concrete admixtures may meet this specification, although the manufacturers of the admixtures should be consulted. Where vapor retarders are used under slabs, their edges should overlap by at least 6 inches and be sealed with adhesive tape. The sheeting should extend to the foundation walls for maximum vapor protection.

As discussed in the ***General*** section, a vapor barrier should be provided below a basement slab. A vapor barrier, as defined by ACI, is a product with a water transmission rate of 0.01 perms when tested in accordance with ASTM E 96. Reinforced membranes having sealed overlaps can meet this requirement.

We recommend that the contractor, the project materials engineer, and the owner discuss these issues and review recent ACI literature and ASTM E-1643 for installation guidelines and guidance on the use of the protection/blotter material.

EXCAVATIONS AND SLOPES

Temporary excavation slopes should not exceed the limits specified in local, state, and national government safety regulations. Also, temporary cuts should be planned to provide a minimum 2 to 3 feet of space for construction of foundations, walls, and drainage. Temporary cuts to a maximum overall depth of about 4 feet may be attempted vertically in unsaturated soil, if there are no indications of slope instability. However, vertical cuts should not be made near property boundaries, or existing utilities and structures, and unshored excavations should not extend beneath a 31 (H:V) declination from the adjacent house foundations or existing foundations and retaining walls. Unless approved by the geotechnical engineer of record, it is important that vertical cuts not be made at the base of sloped cuts. Based upon Washington Administrative Code (WAC) 296, Part N, the soil at the subject site would generally be classified as Type C. Therefore, temporary cut slopes greater than 4 feet in height should not be excavated at an inclination steeper than 1.5:1 (Horizontal:Vertical), extending continuously between the top and the bottom of a cut. Shoring will be required for any cuts extending below the anticipated groundwater level.

The above-recommended temporary slope inclination is based on the conditions exposed in our explorations, and on what has been successful at other sites with similar soil conditions. It is possible that variations in soil and groundwater conditions will require modifications to the inclination at which temporary slopes can stand. Temporary cuts are those that will remain unsupported for a relatively short duration to allow for the construction of foundations, retaining walls, or utilities. Temporary cut slopes should be protected with plastic sheeting during wet weather. It is also important that surface runoff be directed away from the top of temporary slope cuts. Cut slopes should also be backfilled or retained as soon as possible to reduce the potential for instability. Please note that loose soil can cave suddenly and without warning. Excavation, foundation, and utility contractors should be made especially aware of this potential danger. These recommendations may need to be modified if the area near the potential cuts has been disturbed in the past by utility installation, or if settlement-sensitive utilities are located nearby.

All permanent cuts into native soil should be inclined no steeper than 2:1 (H:V). Water should not be allowed to flow uncontrolled over the top of any temporary or permanent slope. All permanently exposed slopes should be seeded with an appropriate species of vegetation to reduce erosion and improve the stability of the surficial layer of soil.

SOLDIER PILE SHORING

This section presents design considerations for temporary and permanent cantilevered soldier pile walls. Cantilevered soldier pile systems have proven to be an efficient method for providing excavation shoring where the depth of excavation is less than 15 feet.

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

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

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

Soldier Pile Wall Design

Temporary soldier pile retaining walls that are cantilevered, and that have a level backslope, should be designed for an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 35 pounds per cubic foot (pcf). Temporary soldier pile shoring used for the western basement wall cut into the western terraced slope should be designed for an active soil pressure of 45 pcf. If soldier pile walls will permanently retain soil, they should be designed for earth pressures presented in the ***Permanent Foundation and Retaining Walls*** section.

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

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

Lateral movement of the soldier piles below the excavation level will be resisted by an ultimate passive soil pressure equal to that pressure exerted by a fluid with a density of 350 pcf. No safety factor is included in the given value. This soil pressure is valid only for a level excavation in front of the soldier pile; it acts on two times the grouted pile diameter. Cut slopes made in front of shoring walls significantly decrease the passive resistance. This includes temporary cuts necessary to install internal braces or rakers. The minimum embedment below the floor of the excavation for cantilever soldier piles should be equal to the height of the "stick-up."

EXCAVATION AND SHORING MONITORING

As with any shoring system, there is a potential risk of greater-than-anticipated movement of the shoring and the ground outside of the excavation. This can translate into noticeable damage of surrounding on-grade elements, such as foundations and slabs. Therefore, we recommend making an extensive photographic and visual survey of the project vicinity, prior to demolition activities, installing shoring or commencing excavation. This documents the condition of buildings, pavements, and utilities in the immediate vicinity of the site in order to avoid, and protect the owner from, unsubstantiated damage claims by surrounding property owners.

Additionally, the shoring walls and any adjacent foundations should be monitored during construction to detect soil movements. To monitor their performance, we recommend establishing a series of survey reference points to measure any horizontal deflections of the shoring system. Control points should be established at a distance well away from the walls and slopes, and deflections from the reference points should be measured throughout construction by survey methods. At least every other soldier pile should be monitored by taking readings at the top of the pile. Additionally, benchmarks installed on the surrounding buildings should be monitored for at least vertical movement. We suggest taking the readings at least once a week, until it is established that no deflections are occurring. The initial readings for this monitoring should be taken before starting any demolition or excavation on the site.

DRAINAGE CONSIDERATIONS

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

Footing drains should be used where: (1) crawl spaces or basements will be below a structure; (2) a slab is below the outside grade; or, (3) the outside grade does not slope downward from a building. Drains should also be placed at the base of all earth-retaining walls. These drains should be surrounded by at least 6 inches of 1-inch-minus, washed rock that is encircled with non-woven, geotextile filter fabric (Mirafi 140N, Supac 4NP, or similar material). At its highest point, a perforated pipe invert should be at least 6 inches below the bottom of a slab floor or the level of a crawl space. The discharge pipe for subsurface drains should be sloped for flow to the outlet point. Roof and surface water drains must not discharge into the foundation drain system. A typical footing drain detail is attached to this report as Plate 6. For the best long-term performance, perforated PVC pipe is recommended for all subsurface drains. Clean-outs should be provided for potential future flushing or cleaning of footing drains.

The preliminary site plans include an elevator in its design. If the structure includes an elevator, it may be necessary to provide special drainage or waterproofing measures for the elevator pit. If no seepage into the elevator pit is acceptable, it will be necessary to provide a footing drain and free-draining wall backfill, and the walls should be waterproofed. If the footing drain will be too low to connect to the storm drainage system, then it will likely be necessary to install a pumped sump to discharge the collected water. Alternatively, the elevator pit could be designed to be entirely waterproof; this would include designing the pit structure to resist hydrostatic uplift pressures.

As discussed in the ***General*** section, an underslab drainage system should be installed below the residence's lowest finished floor slab. A typical detail for underslab drainage is attached to this report as Plate 7.

As a minimum, a vapor retarder, as defined in the ***Building Floors*** section, should be provided in any crawl space area to limit the transmission of water vapor from the underlying soils. Crawl space grades are sometimes left near the elevation of the bottom of the footings. As a result, an outlet drain is recommended for all crawl spaces to prevent an accumulation of any water that may bypass the footing drains. Providing a few inches of free draining gravel underneath the vapor retarder is also prudent to limit the potential for seepage to build up on top of the vapor retarder.

Groundwater was observed during our field work. If seepage is encountered in an excavation, it should be drained from the site by directing it through drainage ditches, perforated pipe, or French drains, or by pumping it from sumps interconnected by shallow connector trenches at the bottom of the excavation.

The excavation and site should be graded so that surface water is directed off the site and away from the tops of slopes. Water should not be allowed to stand in any area where foundations, slabs, or pavements are to be constructed. Final site grading in areas adjacent to walls and foundations should slope away at least one to 2 percent, except where the area is paved. Surface drains should be provided where necessary to prevent ponding of water behind foundation or retaining walls. A discussion of grading and drainage related to pervious surfaces near walls and structures is contained in the ***Foundation and Retaining Walls*** section.

GENERAL EARTHWORK AND STRUCTURAL FILL

All building and pavement areas should be stripped of surface vegetation, topsoil, organic soil, and other deleterious material. It is important that existing foundations be removed before site development. The stripped or removed materials should not be mixed with any materials to be used as structural fill, but they could be used in non-structural areas, such as landscape beds.

Structural fill is defined as any fill, including utility backfill, placed under, or close to, a building, or in other areas where the underlying soil needs to support loads. All structural fill should be placed in horizontal lifts with a moisture content at, or near, the optimum moisture content. The optimum moisture content is that moisture content that results in the greatest compacted dry density. The moisture content of fill is very important and must be closely controlled during the filling and compaction process.

The allowable thickness of the fill lift will depend on the material type selected, the compaction equipment used, and the number of passes made to compact the lift. The loose lift thickness should not exceed 12 inches, but should be thinner if small, hand-operated compactors are used. We recommend testing structural fill as it is placed. If the fill is not sufficiently compacted, it should be recompacted before another lift is placed. This eliminates the need to remove the fill to achieve the required compaction.

The following table presents recommended levels of relative compaction for compacted fill:

LOCATION OF FILL PLACEMENT	MINIMUM RELATIVE COMPACTION
Beneath walkways	95%
Filled slopes and behind retaining walls	90%
Beneath pavements	95% for upper 12 inches of subgrade; 90% below that level

Where: Minimum Relative Compaction is the ratio, expressed in percentages, of the compacted dry density to the maximum dry density, as determined in accordance with ASTM Test Designation D 1557-91 (Modified Proctor).

LIMITATIONS

The conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our exploration and assume that the soil and groundwater conditions encountered in the borings are representative of subsurface conditions on the site. If the subsurface conditions encountered during construction are significantly different from those observed in our explorations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Unanticipated conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking samples in borings. Subsurface conditions can also vary between exploration locations. Such unexpected conditions frequently require making additional expenditures to attain a properly constructed project. It is recommended that the owner consider providing a contingency fund to accommodate such potential extra costs and risks. This is a standard recommendation for all projects.

This report has been prepared for the exclusive use of Valeri Liborski, Kira Shabalin and their representatives, for specific application to this project and site. Our conclusions and recommendations are professional opinions derived in accordance with our understanding of current local standards of practice, and within the scope of our services. No warranty is expressed or implied. The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. Our services also do not include assessing or minimizing the potential for biological hazards, such as mold, bacteria, mildew and fungi in either the existing or proposed site development.

ADDITIONAL SERVICES

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

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

The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3 - 5	Boring Logs
Plate 6	Typical Footing Drain Detail
Plate 7	Typical Underslab Drainage Detail
Plate 8	Typical Shoring Drain Detail

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

Respectfully submitted,

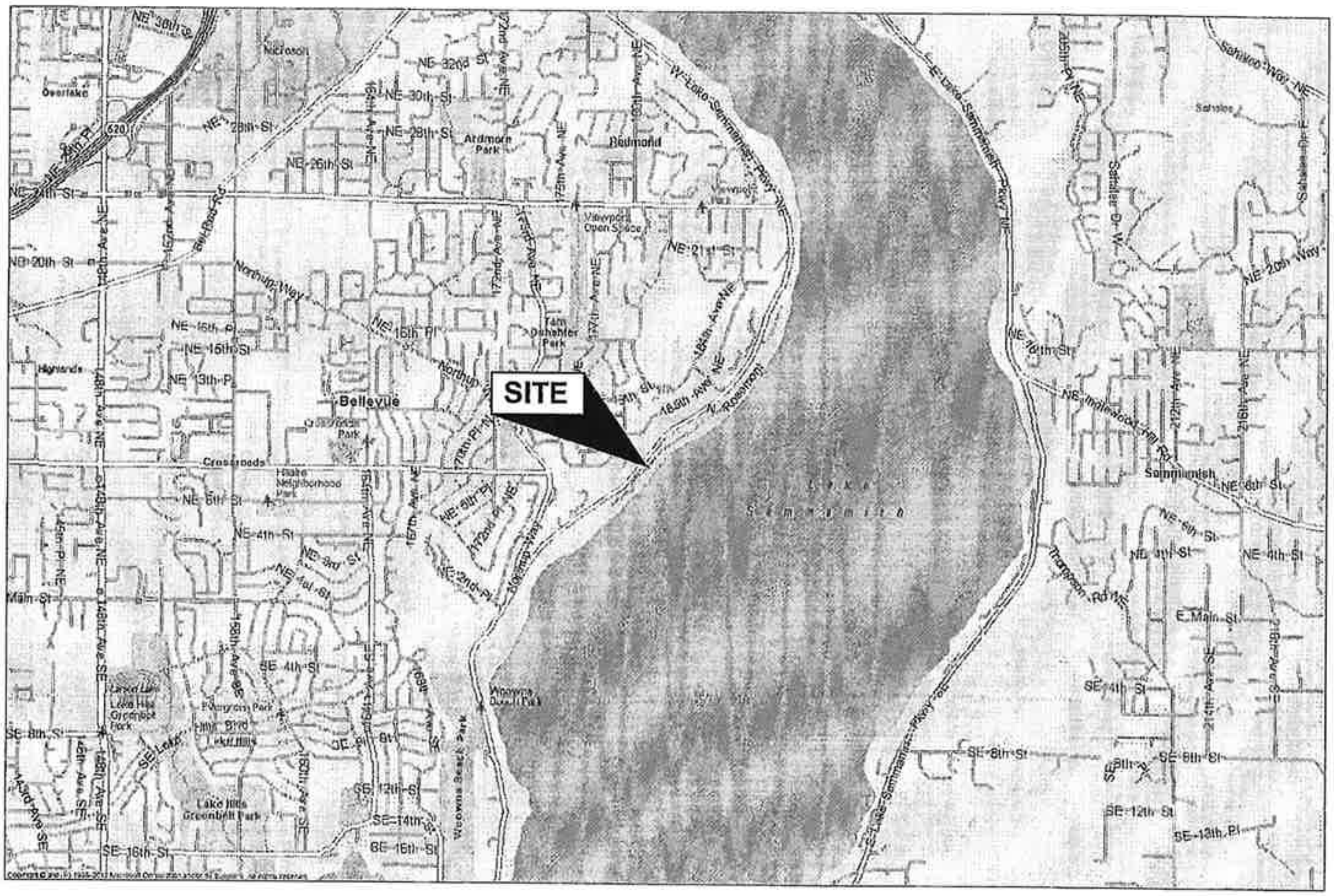
GEOTECH CONSULTANTS, INC.

04/12/19

Marc R. McGinnis, P.E.
Principal

MKM/MRM:kg

NORTH



(Source: Microsoft MapPoint, 2013)

GEOTECH
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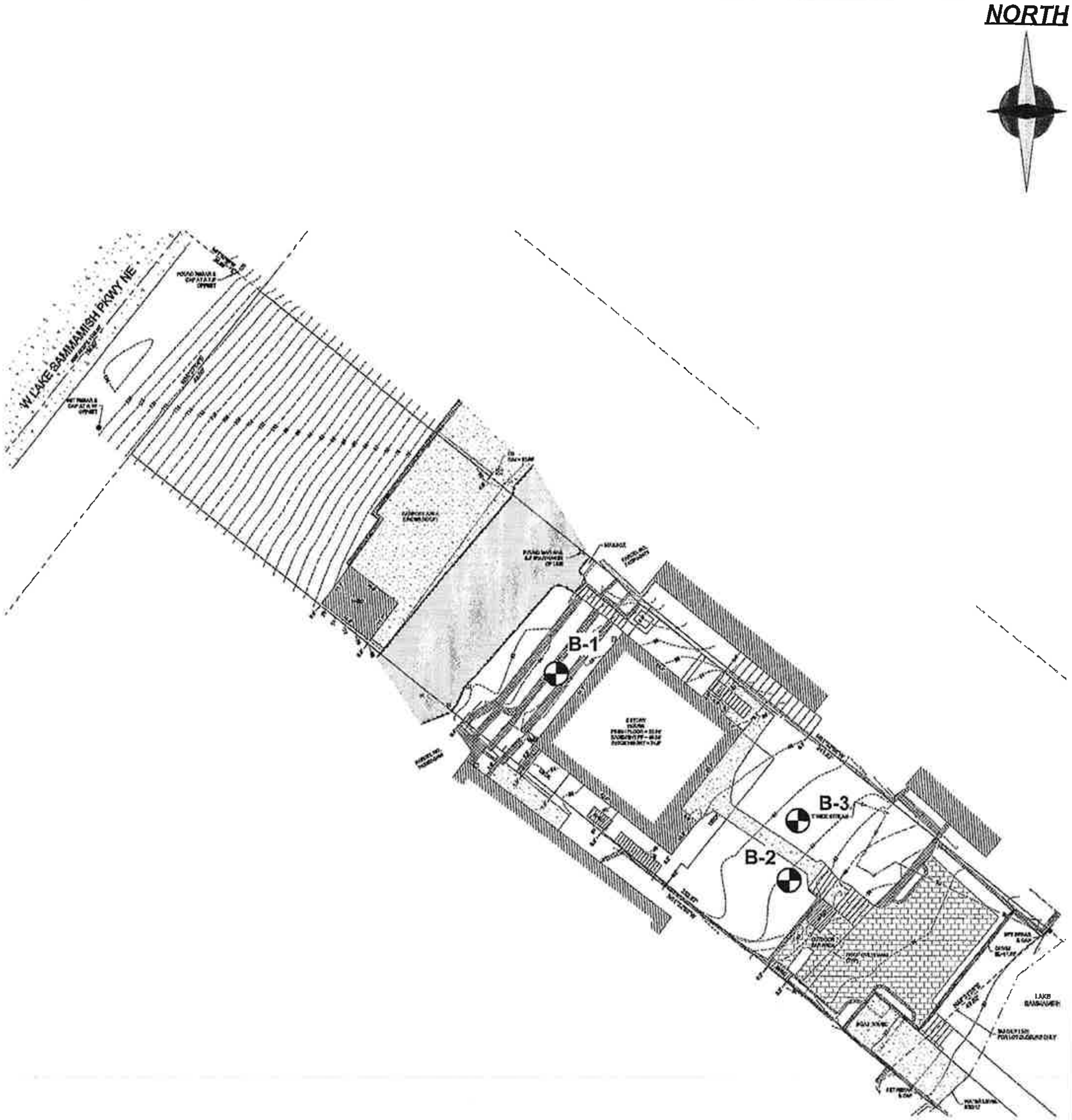
VICINITY MAP

856 West Lake Sammamish Parkway NE
Bellevue, Washington

Job No:	Date:		Plate:
19090	Apr. 2019		1



Legend:

Test Boring Location

GEOTECH
CONSULTANTS, INC.

SITE EXPLORATION PLAN

856 West Lake Sammamish Parkway NE
Bellevue, Washington

Job No:	Date:		Plate:
19090	Apr. 2019	No Scale	2

BORING 1

Blows per Foot	Sample	USCS	Description
4	1	FILL	Brown gravelly SAND, fine-grained, moist, loose **(FILL)**
3	2	FILL	-becomes reddish-brown, very moist, very loose
8	3	FILL	-with small chunks of gray-brown silt, becomes wet
3	4	FILL	-becomes very gravelly, fine to coarse-grained
44	5	ML/SM	Bluish-gray SILT and very silty SAND with layers of compressed organics, non-plastic, fine-grained, moist, dense
54	6	ML/SM	-with thinner layers of organics, becomes wet, very dense -becomes moist
49	7	ML/SM	-becomes wet, with layers of gravelly sand

Depth (ft.): 5, 10, 15, 20, 25

* **Test boring was terminated at 24 feet on March 28, 2019.**
* **Perched groundwater seepage was observed from 7.5 to 13 feet and from 17 to 21 feet during drilling.**
* **Groundwater was encountered below 22 feet during drilling.**

GEOTECH
CONSULTANTS, INC.

TEST BORING LOG

856 West Lake Sammamish Parkway NE
Bellevue, Washington

Job	Date:	Logged by:	Plate:
19090	Apr. 2019	MKM	3

BORING 2

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
		▼				
			6	1	FILL	Black and brown gravelly, silty SAND with organics, fine-grained, wet, loose **(FILL)**
5			4	2		-becomes dark-gray to black
			3	3	SM	Dark-gray and blue gravelly, silty SAND, fine to medium-grained, wet, loose
10			**35	4	GP	Gray sandy GRAVEL with fractured rocks, fine to medium-grained, wet, medium-dense
			12	5	SM	Blue-gray very gravelly, silty SAND with cobbles, fine to medium-grained, wet, medium-dense
15						
20						
25						

*** Test boring was terminated at 13 feet on March 28, 2019 due to auger refusal on cobbles.**

*** Groundwater seepage was observed below 2.5 feet during drilling.**

**** Overstated blow counts due to heavy gravel content.**

GEOTECH
CONSULTANTS, INC.

TEST BORING LOG

856 West Lake Sammamish Parkway NE
Bellevue, Washington

Job	Date:	Logged by:	Plate:
19090	Apr. 2019	MKM	4

BORING 3

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
		▼	4	1	FILL	Brown gray, dark-brown and black gravelly, silty SAND with organics, fine-grained, wet, loose **(FILL)**
5			3	2		
			23	3	SP	Dark-gray gravelly SAND with cobbles, fine to medium-grained, wet, medium-dense
10						-becomes fine to coarse-grained
			27	4	ML	Brown SILT with gravel, slightly plastic, wet, very stiff
15						-becomes gray with layers of compressed organics
			44	5	SM	Gray gravelly, silty SAND with organics, fine-grained, wet, dense
20						
25						

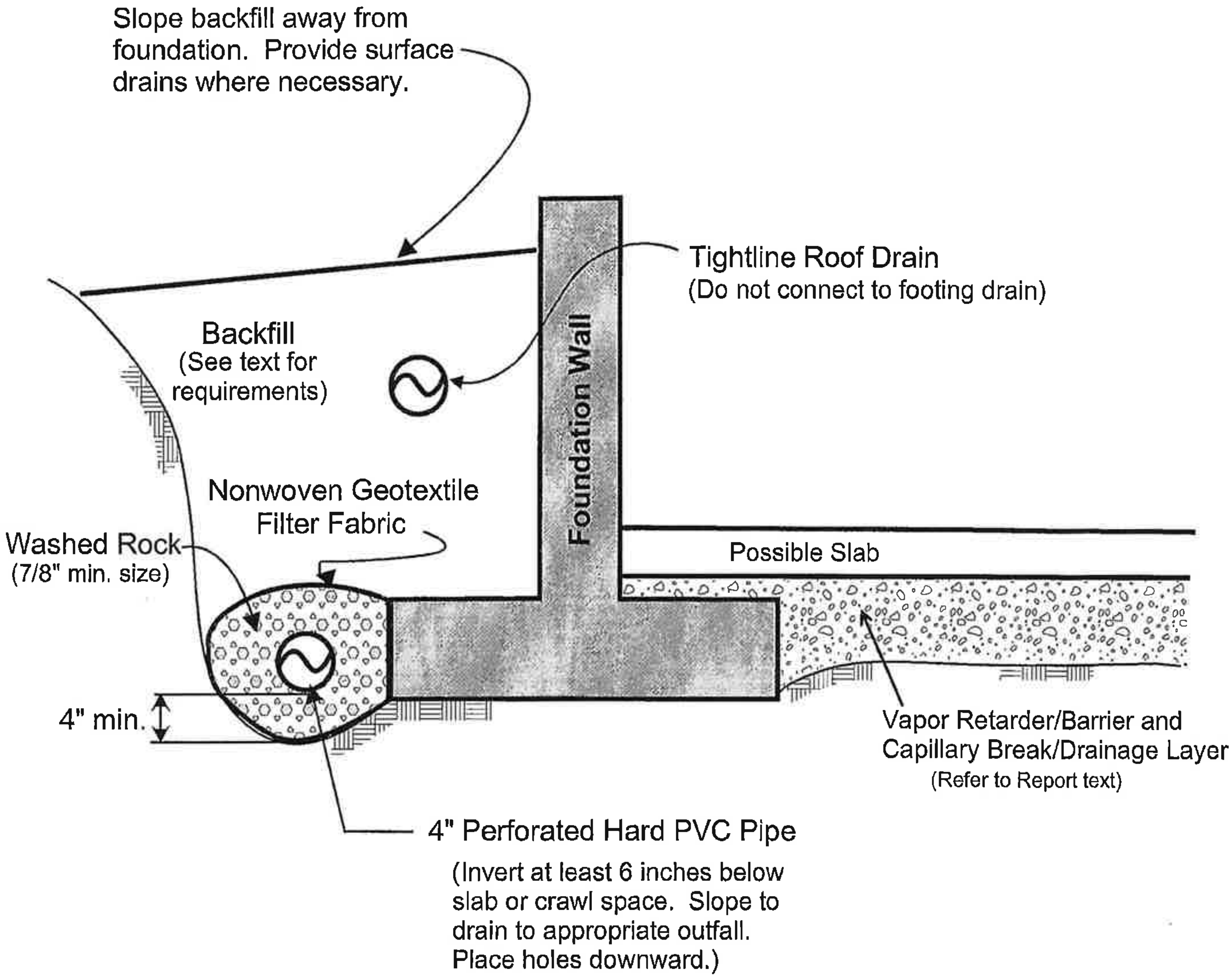
*** Test boring was terminated at 16.5 feet on March 28, 2019.**
*** Groundwater seepage was observed at 2.5 feet during drilling.**

GEOTECH
CONSULTANTS, INC.

TEST BORING LOG

856 West Lake Sammamish Parkway NE
Bellevue, Washington

Job	*Date:*	*Logged by:*	*Plate:*
19090	Apr. 2019	MKM	5



NOTES:

(1) In crawl spaces, provide an outlet drain to prevent buildup of water that bypasses the perimeter footing drains.

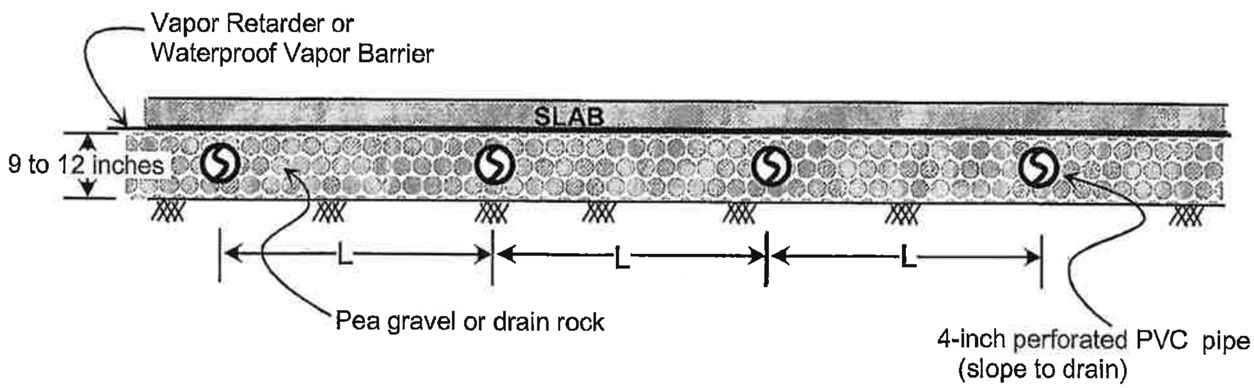
(2) Refer to report text for additional drainage, waterproofing, and slab considerations.

GEOTECH
CONSULTANTS, INC.

FOOTING DRAIN DETAIL

856 West Lake Sammamish Parkway NE
Bellevue, Washington

Job No:	Date:		Plate:
19090	Apr. 2019		6



NOTES:

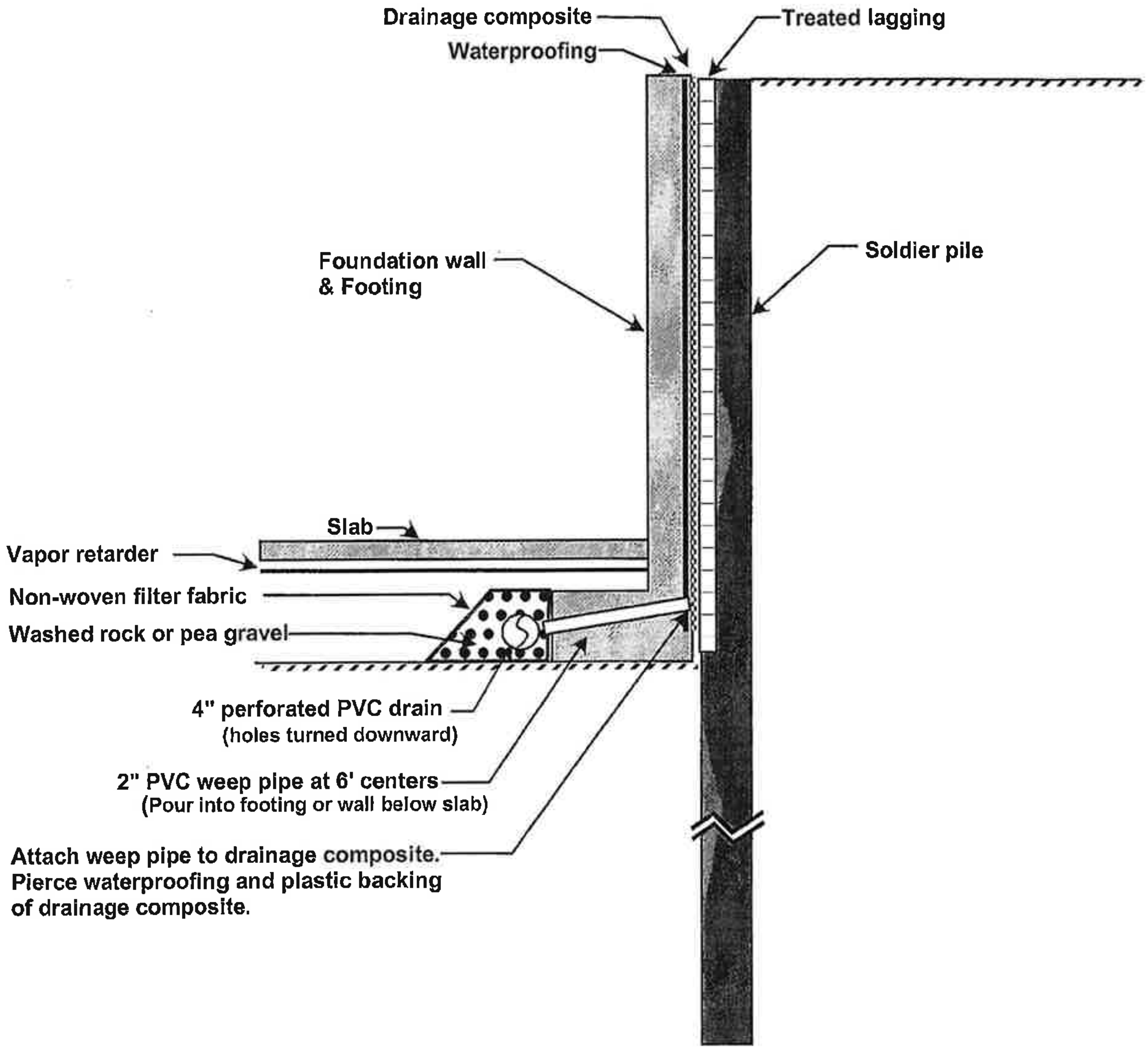
(1) Refer to the report text for additional drainage and waterproofing considerations.
(2) The typical maximum underslab drain separation (L) is 15 to 20 feet.
(3) No filter fabric is necessary beneath the pipes as long as a minimum thickness of 4 inches of rock is maintained beneath the pipes.
(4) The underslab drains and foundation drains should discharge to a suitable outfall.

GEOTECH
CONSULTANTS, INC.

TYPICAL UNDERSLAB DRAINAGE

856 West Lake Sammamish Parkway NE
Bellevue, Washington

Job No:	Date:		Plate:
19090	Apr. 2019		7

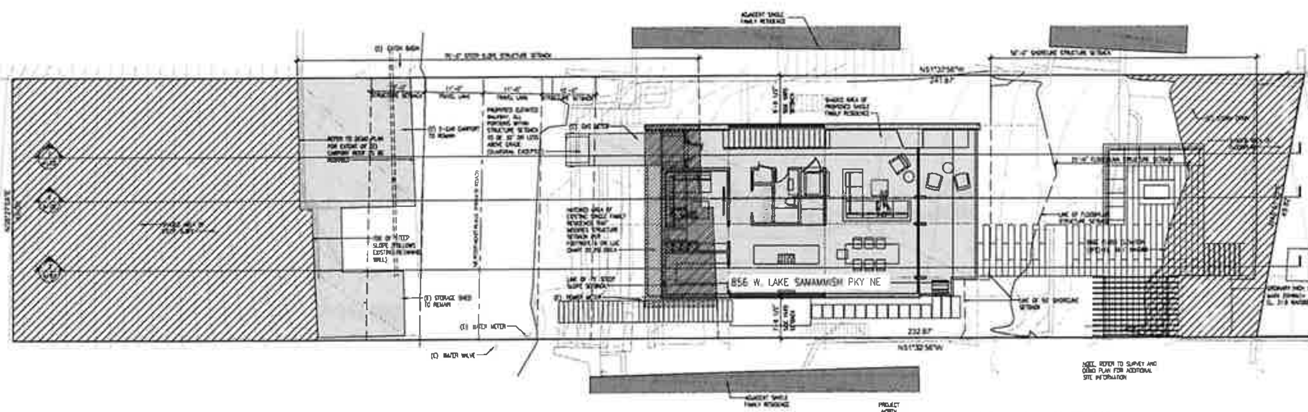


<u>Note</u> - Refer to the report for additional considerations related to drainage and waterproofing.

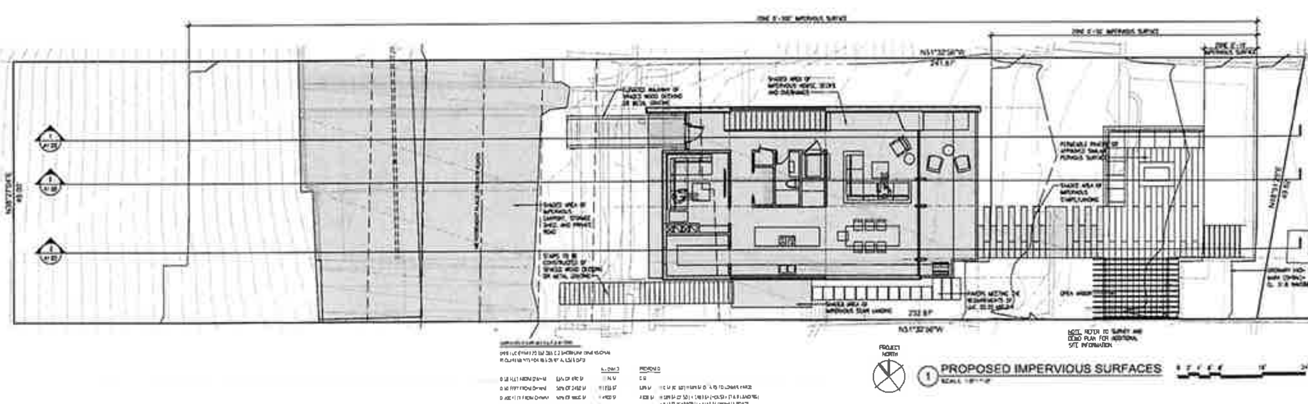
GEOTECH
CONSULTANTS, INC.

SHORING DRAIN DETAIL
856 West Lake Sammamish Parkway NE
Bellevue, Washington

Job No:	Date:		Plate:
19090	Apr. 2019		8



2 PROPOSED LOT COVERAGE



1 PROPOSED IMPERVIOUS SURFACES

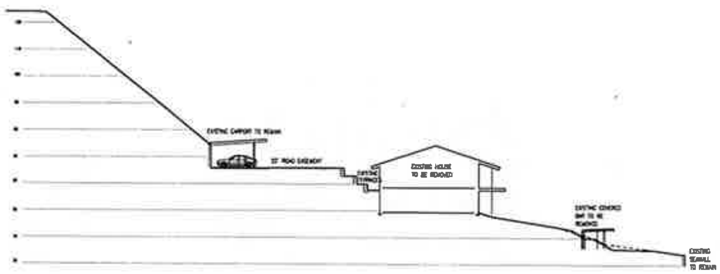
CALUP SUBMITTAL

Shabalin-Liborski Residence

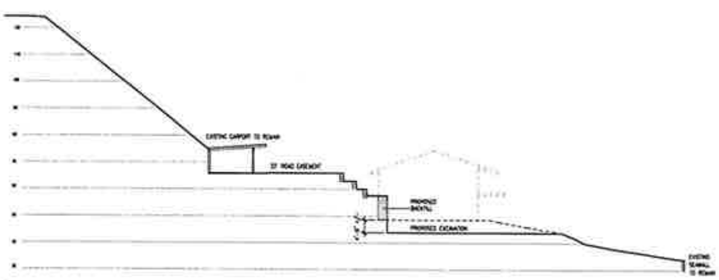
Scale AS NOTED

SITE PLAN

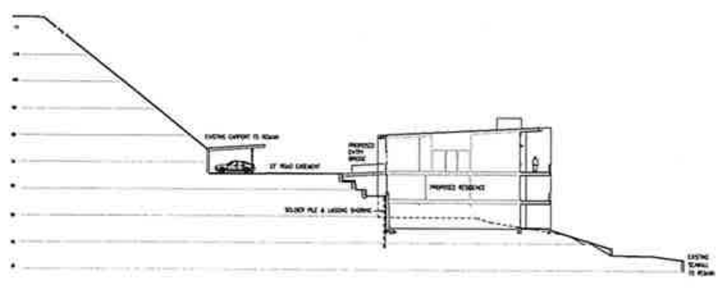
A1.01



③ SITE SECTION SHOWING EXISTING CONDITIONS



② SITE SECTION SHOWING PROPOSED EXCAVATION



① SITE SECTION SHOWING PROPOSED CONSTRUCTION

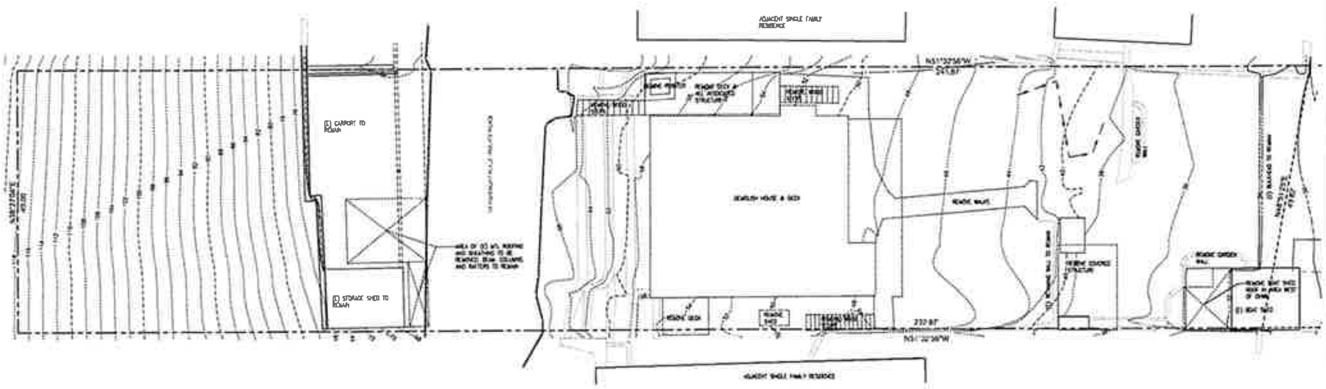
CALUP SUBMITTAL

Shabalin-Liberski Residence

SITE SECTIONS

A1.02

VICINITY MAP



1 DEMOLITION PLAN

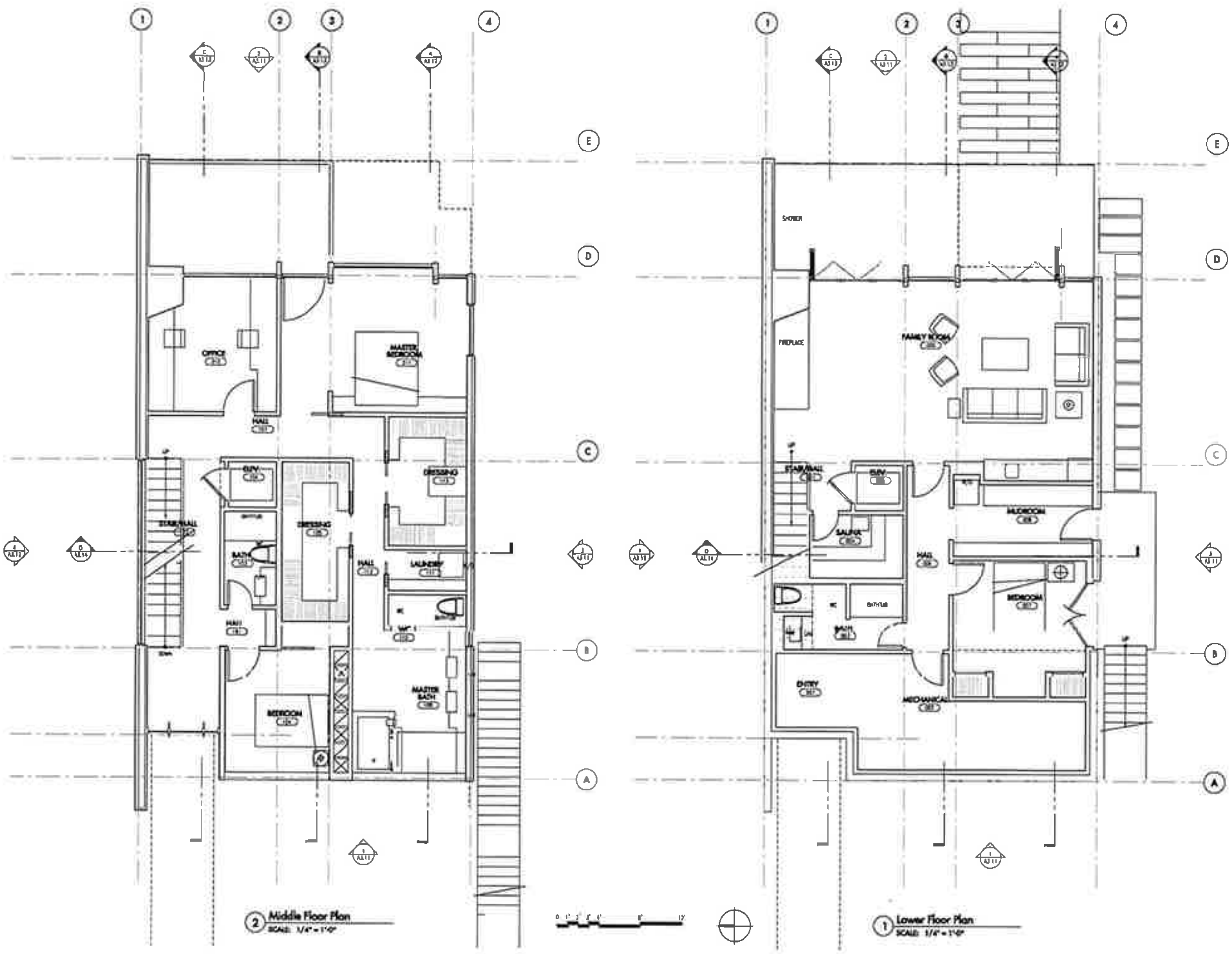
CALUP SUBMITTAL

Shabalin-Liborski Residence

Scale AS NOTED

DEMOLITION PLAN

D1.01



2 Middle Floor Plan
SCALE: 1/4" = 1'-0"

1 Lower Floor Plan
SCALE: 1/4" = 1'-0"

NOT FOR CONSTRUCTION
AUGUST 2019

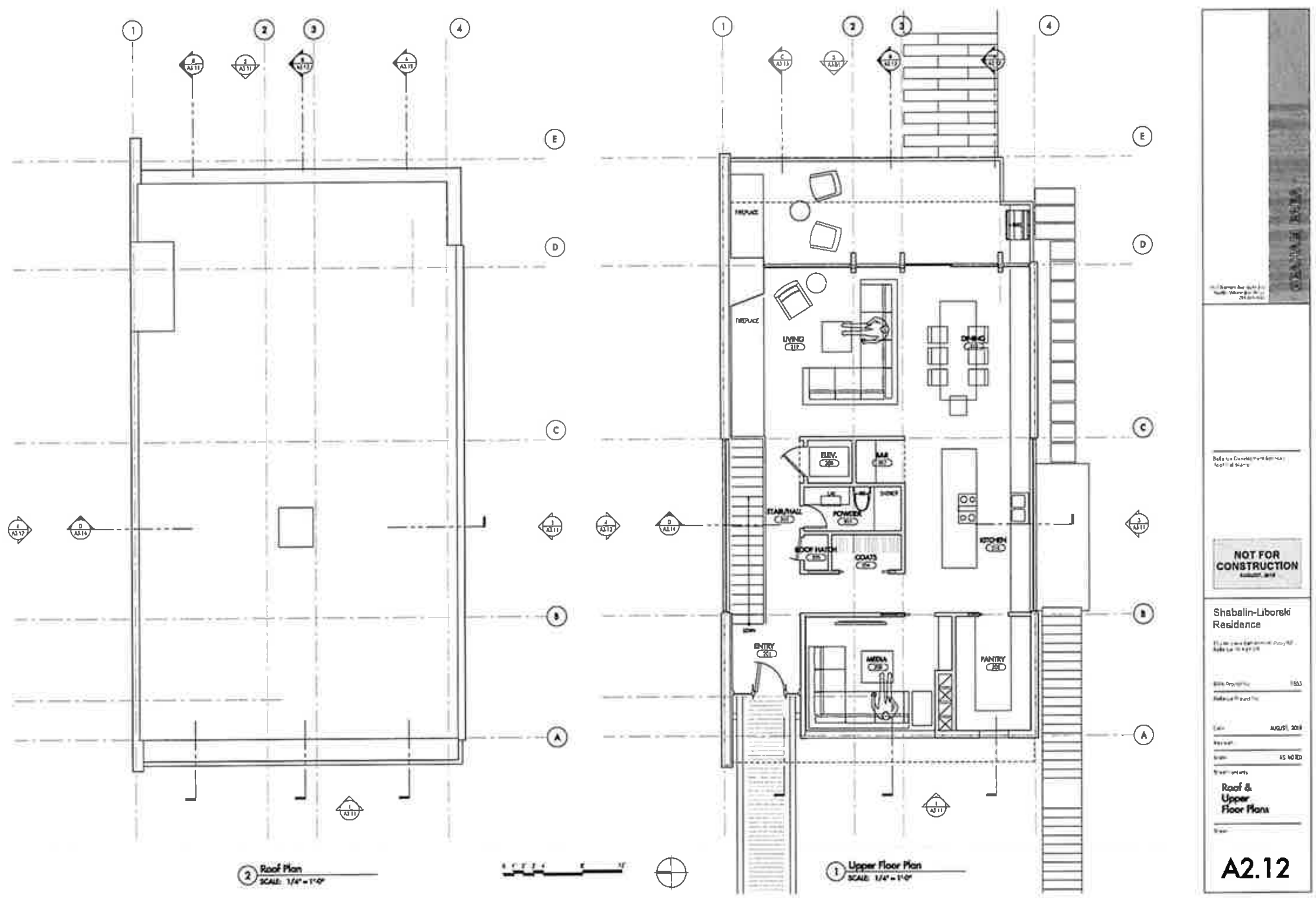
Shabalin-Liborski Residence

Date AUGUST 2019

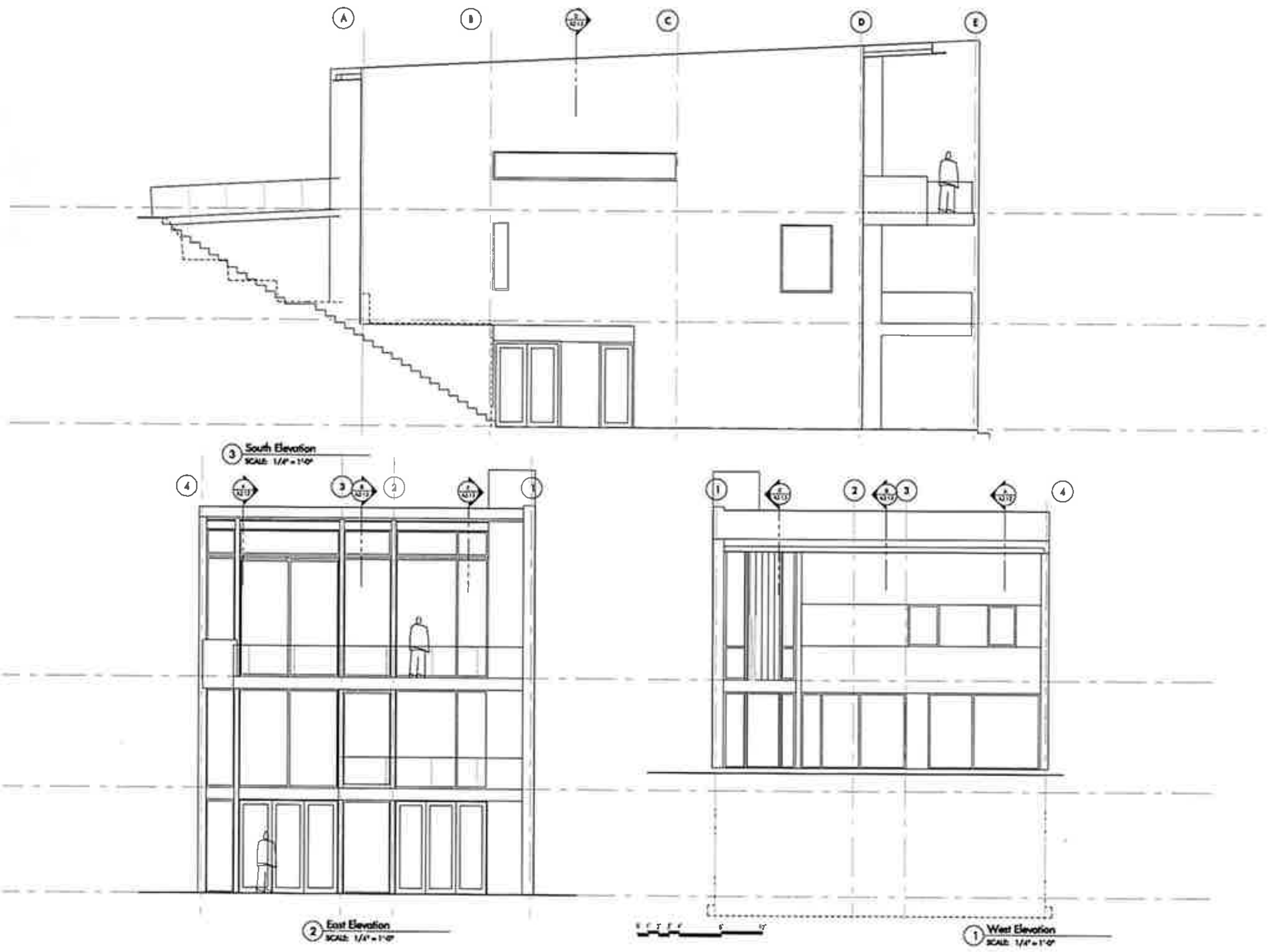
Scale AS NOTED

Lower & Middle Floor Plan

A2.11



Roof Plan
SCALE: 1/4" = 1'-0"
Upper Floor Plan
SCALE: 1/4" = 1'-0"
FIREPLACE
LIVING
DINING
ELEV.
BAR
STAIR/HALL
POWDER
ROOF HATCH
COATS
KITCHEN
ENTRY
MEDIA
PANTRY
NOT FOR
CONSTRUCTION
Shabalin-Liborski
Residence
AUGUST, 2018
AS NOTED
Roof &
Upper
Floor Plans
A2.12



Approval Stamp

NOT FOR CONSTRUCTION

Shabalin-Liborski Residence

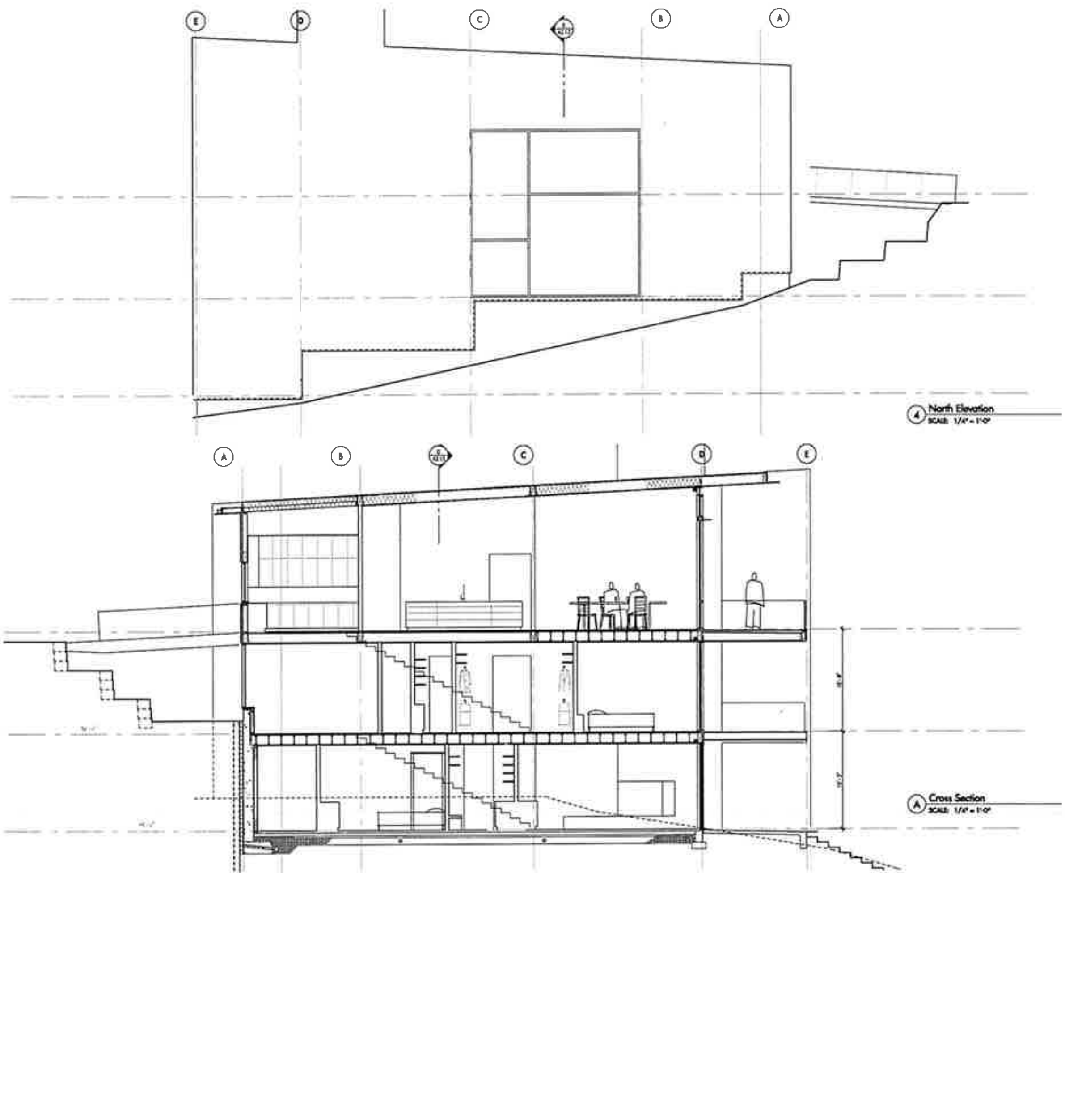
Date AUGUST, 2018

Scale AS NOTED

West, South & East Elevations

Sheet

A3.11



NOT FOR CONSTRUCTION

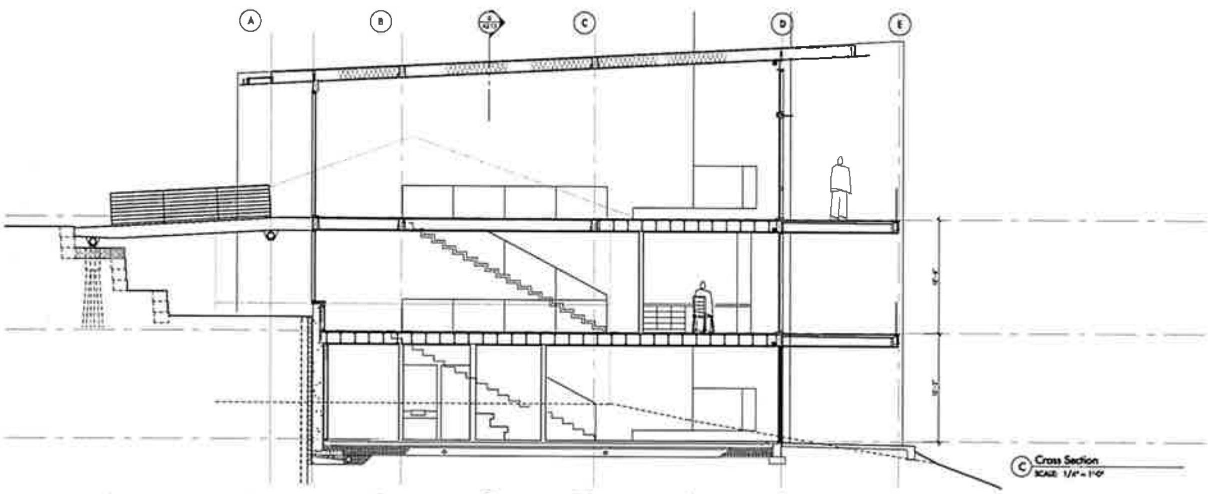
Shabalin-Liborski Residence

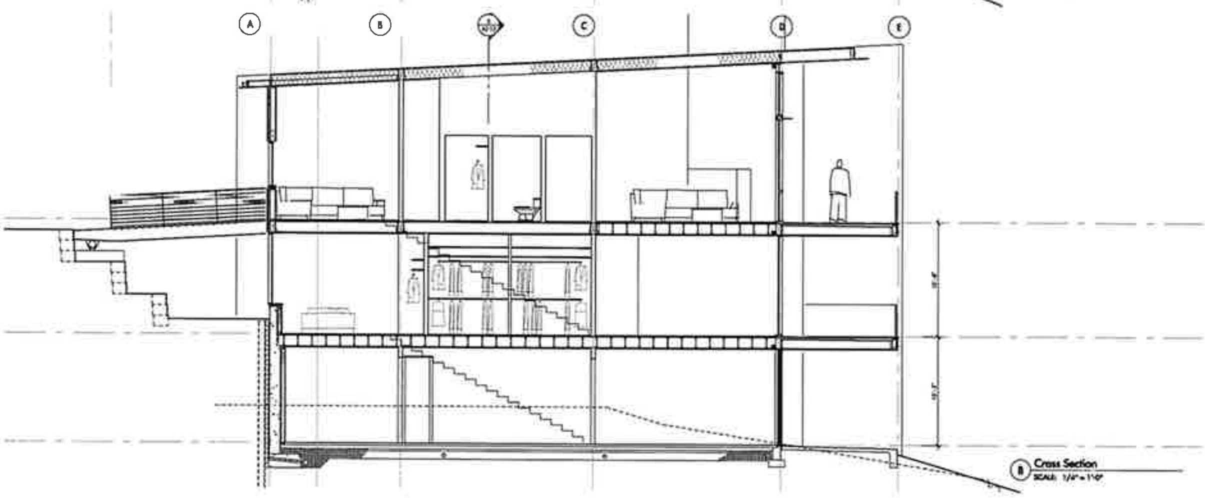
Date AUGUST 2019

Scale AS NOTED

North Elevation & Building Section

A3.12





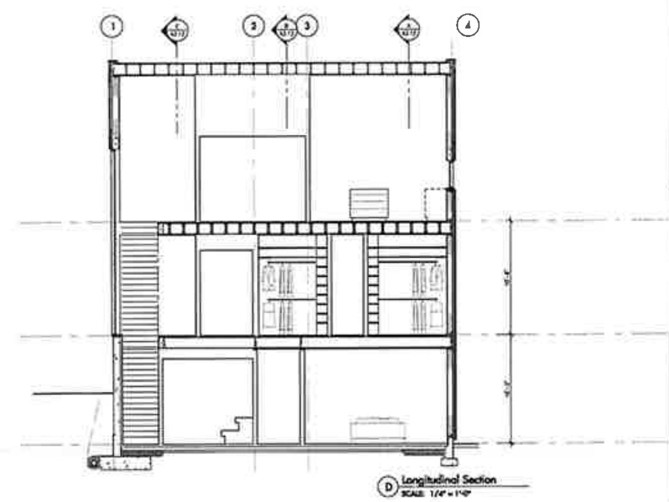
NOT FOR CONSTRUCTION

Shabalin-Liborski Residence

Scale AS NOTED

Building Sections

A3.13



NOT FOR CONSTRUCTION

Shabalin-Liborski Residence

Date
AUGUST

Scale
AS NOTED

Sheet contents

Building Sections

Sheet

A3.14

DECKER
Consulting Engineers

1511 3rd Avenue, Suite 323
Seattle, WA. 98101
Ph. 206-403-0933

PRELIMINARY DRAINAGE MEMO
For Critical Area Land Use Permit
#19-116945 - LO

For:

856 West Lake Sammamish Parkway NE
Parcel # 7430500480

New Shabalin-Liborski Residence

Located in:
Bellevue, Washington

JAY D. DECKER
STATE OF WASHINGTON
31195
REGISTERED PROFESSIONAL ENGINEER

8/30/19

June 26, 2019
Revised August 27, 2019

By:

Decker Consulting Engineers, PLLC
1511 3rd Avenue, Suite 323
Seattle, WA. 98101
Contact: Jay D. Decker, PE
(206) 403-0933
DCE Job No. 2019004

1/14

I. DRAINAGE DESIGN OVERVIEW - Critical Areas Land Use Permit (CALUP - "LO" Permit)

This preliminary drainage Memo is proposed to support the CALUP application. ***The full drainage report will be developed during the building permit "BS" application.***

This project involves the demolition and reconstruction of a new home at 856 West Lake Sammamish Parkway NE. **(See *Figure 1 - Vicinity Map & Infiltration Feasibility Map.*)** The site is located in the "Rosemont" drainage area. **See *Figure 2 - Rosemont Drainage Basin Map.***

Stormwater from the existing and proposed project drains through a tightlined pipe into Lake Sammamish. **(see *Figure 3 - Downstream Drainage Course.*)**

The project site is 11,631 square feet or 0.267 acre. Since the ratio of existing hard surface to total lot area exceeds 35%, this is considered a "redevelopment" project. **(see *Figure 4 - Existing Hard Surface Area.*)**

A new home will be located roughly the same configuration as the old home with a roof area of roughly 2,200 SF - ***See Figure 5 - Proposed Condition Site Area Breakdown.***

The home will be accessed from a permeable (rain can pass through) bridge structure from the road above. There will also be a stoop/patio located on the southwest side of the home. The "New + Replaced Hard Surface Area" will be less than 5,000 SF (roughly 3,000 SF.)

Entering the "New Development" and "Redevelopment" flow charts for determining minimum requirements (***Figures 6a and 6b***) yields the requirement to address minimum requirements 1-5.

The drainage report and plans to be prepared for this project will conform with Appendix D-2 of the 2019 City of Bellevue Surface Water Engineering Standards.

Infiltration not feasible
(RED HATCH)

Figure 1 - Vicinity Map & Infiltration Feasibility Map

Rosemont Area

Lake Washington Watershed (WRIA 8)

3/14

LAND CHARACTERISTICS

Basin Area: 152 Total Acres (City area: 0.7%)

Drainage Jurisdiction(s):

Bellevue: 100.0%	Kirkland: 0.0%
Beaux Arts: 0.0%	Medina: 0.0%
Clyde Hill: 0.0%	Newcastle: 0.0%
Issaquah: 0.0%	Redmond: 0.0%
King County: 0.0%	Renton: 0.0%

Lowest Elevation: 31 Ft

Highest Elevation: 330 Ft

Total Length of Open Channel: 0.0 Miles

Total Length of Storm Drainage Pipes: 3.3 Miles

POPULATION

Basin Population (2016): 926 (0.6% of all Basins)

Basin Population Density: 3,895 People per Square Mile

The population density in Bellevue ranges from 1,344 to 9,851 people per square mile.

LAND USE

LAND USE	Entire Basin	Within Bellevue
Public Right of Way:	17.0 %	17.1 %
Commercial/Office:	0.0 %	0.0 %
Industrial:	0.0 %	0.0 %
Institutional/Government:	0.0 %	0.0 %
Mixed Use/Misc:	4.9 %	4.9 %
Multi-Family	0.0 %	0.0 %
Open Space/Park:	2.5 %	2.5 %
Single Family Residential:	74.6 %	74.5 %
Unknown:	1.0 %	1.0 %

SALMON PRESENT in BASIN

Lake only: Chinook*+, Coho+ , Kokanee+, Sockeye

Rainbow & cutthroat trout (Lake only)

Steelhead (Lake only)

* Listed Federal Endangered Species

+ City Species of Local Importance (Bellevue Land Use Code 20.25H.150A)

LAND COVER

LAND COVER	Entire Basin	Within Bellevue
Impervious:	37.5 %	37.8 %
Tree Canopy:	44.8 %	45.2 %
Impervious in 100 Ft Stream Buffer:	0.0 %	0.0 %
Tree Canopy in 100 Ft Stream Buffer:	0.0%	0.0 %

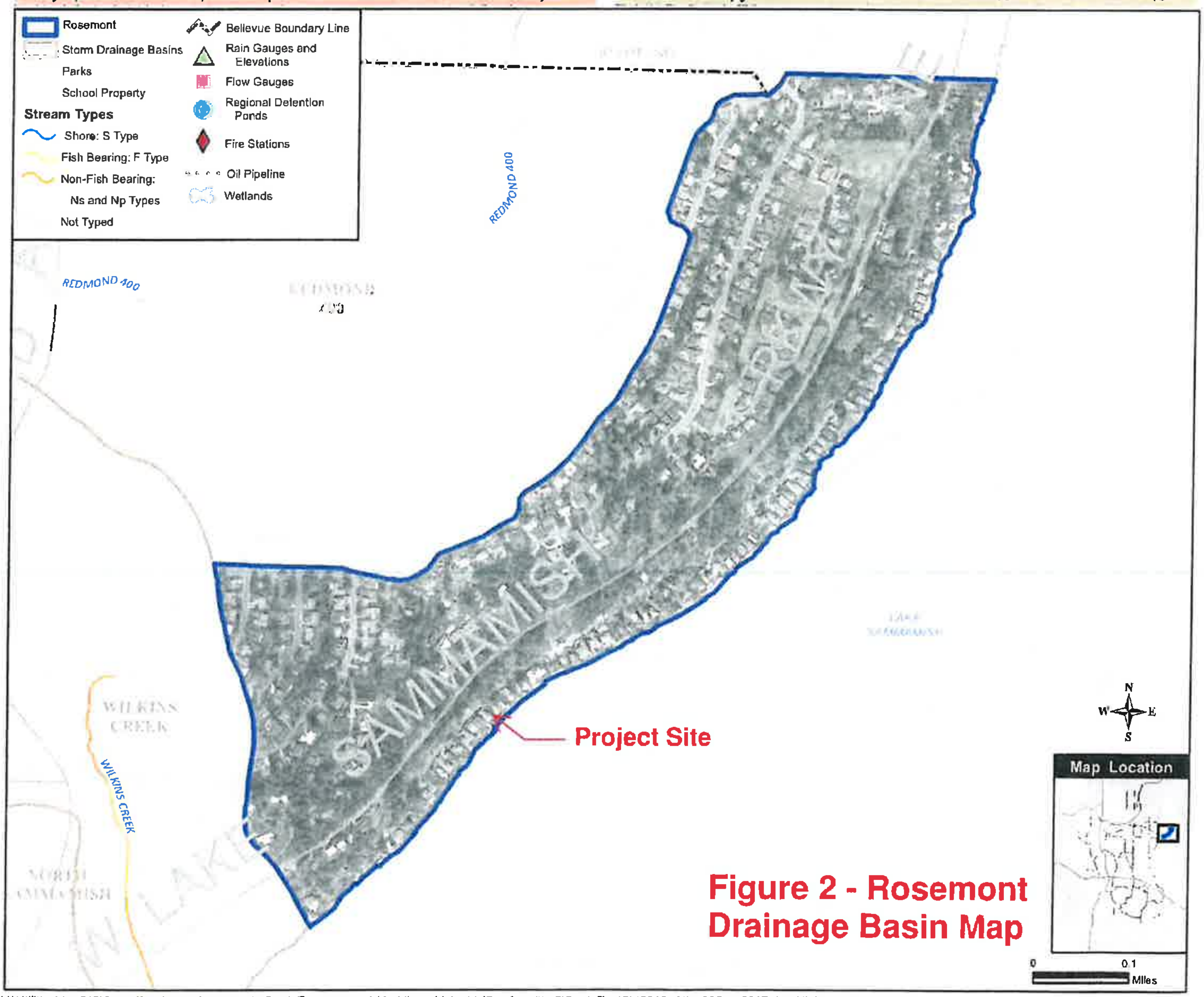


Figure 2 - Rosemont Drainage Basin Map

V:\Utilities\ArcGIS\Storm\BasinFactSheets2016\BasinFactSheets2016_Atlases\AtlasMXDs_Sept2017\BasinFactSht2016_Atlas06Sept2017_LegNW_TextMaplex.mxd

Updated: July 2017

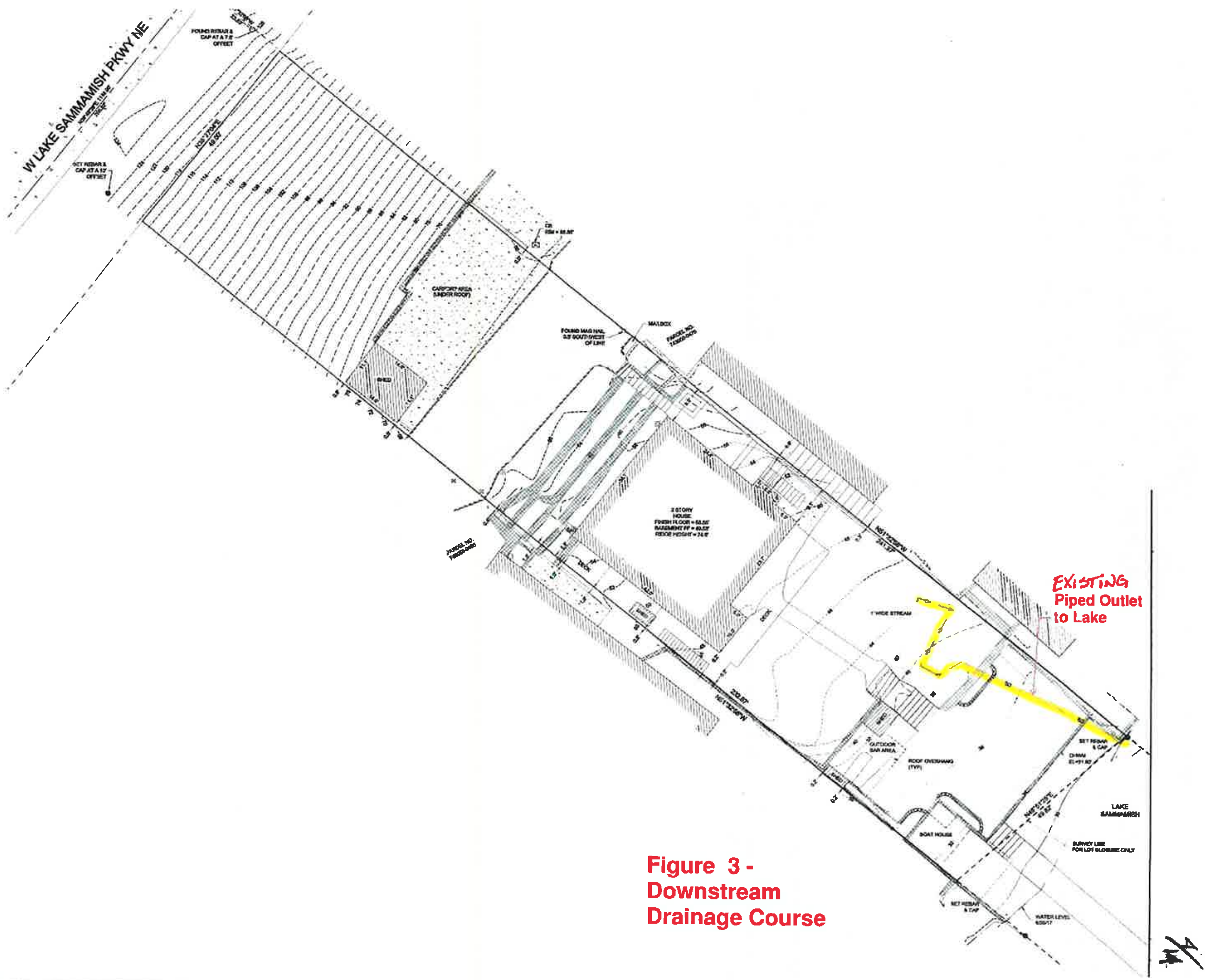


Figure 3 - Downstream Drainage Course

4/14

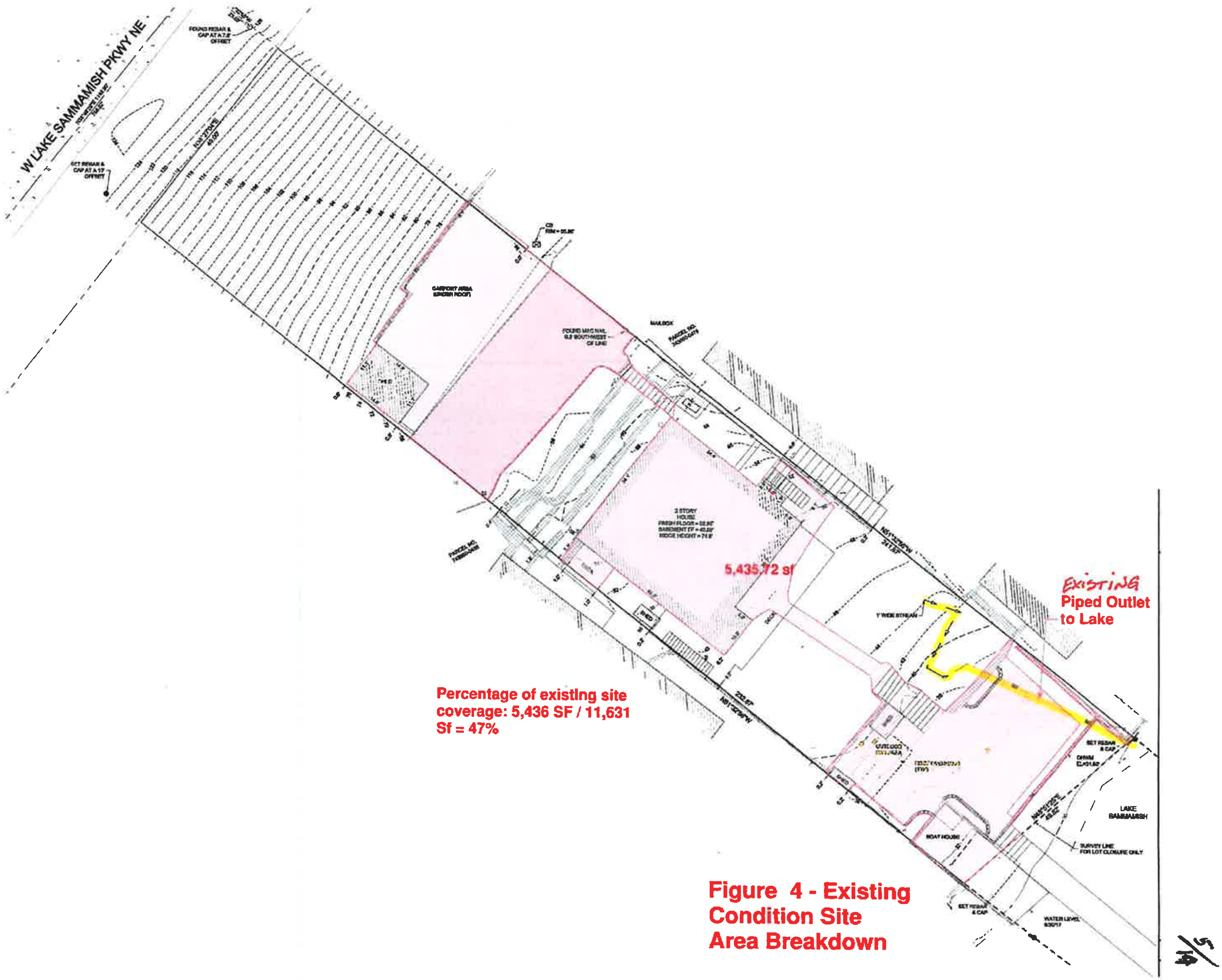


Figure 4 - Existing Condition Site Area Breakdown

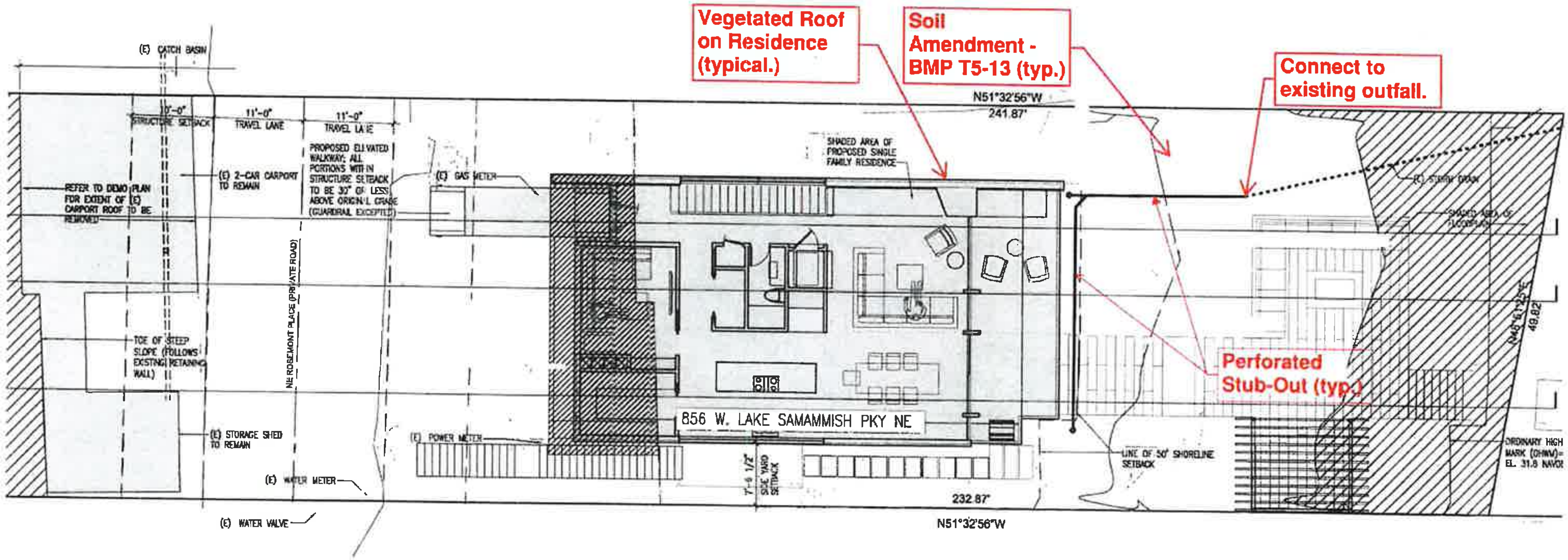


Figure 5 - Proposed Condition Site Area Breakdown

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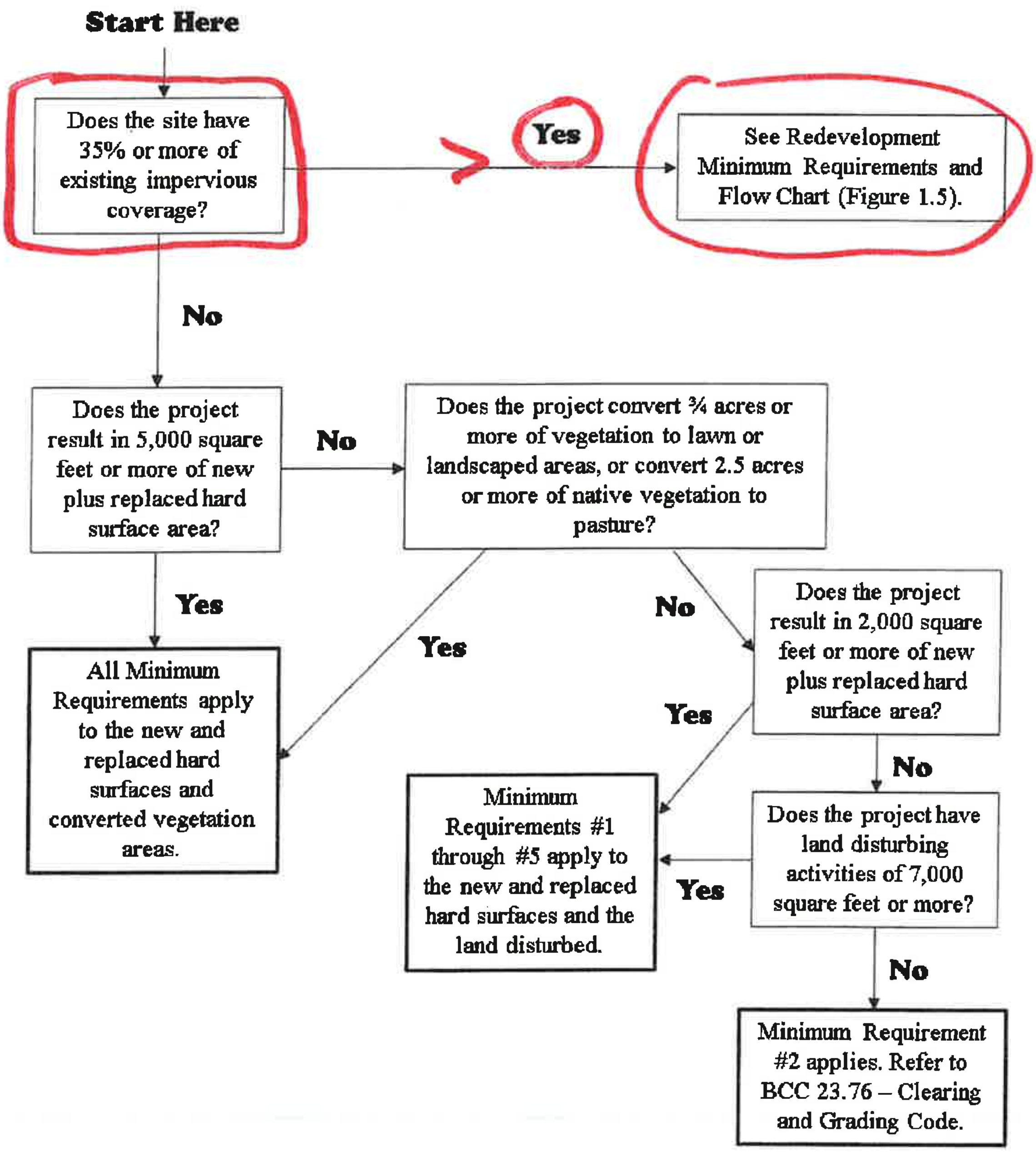

Figure 1.4 – Flow Chart for Determining Minimum Requirements for New Development Projects

Source: Adapted from Figure 2.4.1 of Volume I of the DOE Manual.

FIGURE 62

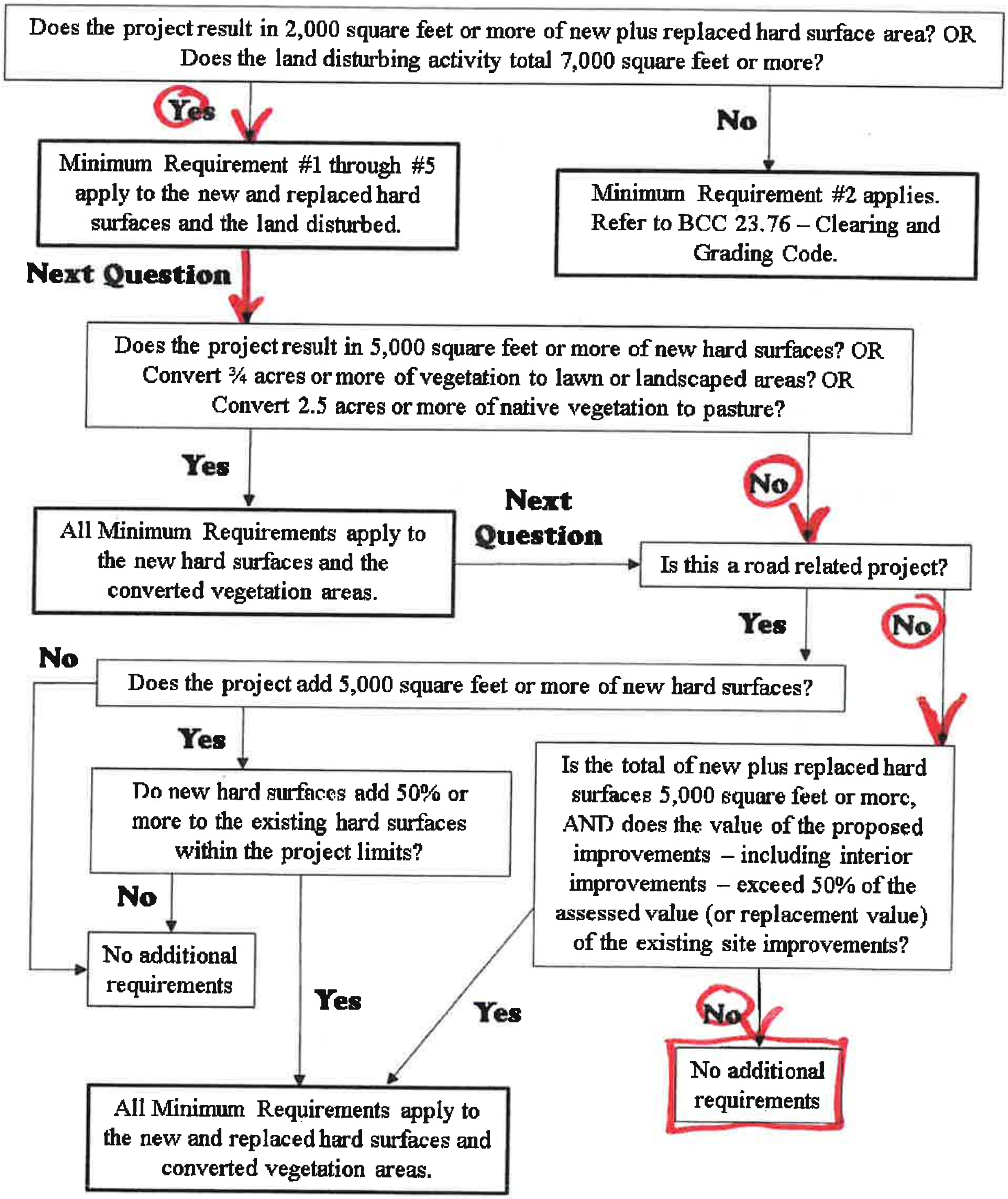

Figure 1.5 – Flow Chart for Determining Minimum Requirements for Redevelopment Projects

Source: Adapted from Figure 2.4.2 of Volume I of the DOE Manual.

FIGURE 6b.

9/14

II. EXISTING CONDITIONS AND MINIMUM REQUIREMENTS ANALYSIS

Existing topography on the site is challenging. The elevation at the West corner of the site is 118 (W Lake Sammamish Parkway NE) and at the east corner of the site (by the lake) elevation 30 for a grade differential of 88 feet.

In the region of work adjacent to the existing home the slope is approximately 20%.

According to SCS data for the area, geotechnical conditions are glacial till and silty soils. The geotechnical report describes three borings advanced on the site. The report confirms that there is between 7 and 13-feet of loose, wet fill soils on the top of the site. Under the fill native silt and very silty sand were encountered in the borings.

Stormwater from this project site flows east down the site to an engineered open channel on the northeast quadrant of the project and then enters a piped conveyance to Lake Sammamish. See ***Figure 3 - Downstream Drainage Course.*** The proposed project will eliminate the engineered open channel and all drainage from the site will be tightlined and connect to the existing drainage system on the site. See ***Figure 5 - Proposed Condition Site Area Breakdown.***

There were no observed drainage problems between the residence and Lake Sammamish.

The existing hard surface on the site is roughly 47% of the existing 11,631 SF parcel (***Figure 4.***) Therefore, this is a "redevelopment" project.

The new + replaced hard surface area is roughly 3,000 SF - ***See Figure 5 - Proposed Condition Site Area Breakdown.*** Entering ***Figures 6a And 6b*** (Flow Charts for Determining Requirements for New and Redevelopment) yields the requirement to address ***Minimum Requirements 1-5.*** These are discussed below:

- **MR-1** - Preparation of Stormwater Site Plans
- A storm drainage plan and drainage report is being prepared for this project in accordance with the 2019 City of Bellevue Surface Water Engineering Standards (and by reference the 2014 DOE Manual.)

- **MR-2** - Construction Stormwater Pollution Prevention Plan
- See Section V. "Construction Stormwater Pollution Prevention-Plan (CSWPPP) Plan" in this report as well as in the civil plans for the project.

- **MR-3** - Source Control of Pollution
- There will be no outdoor storage of fertilizers, pesticides, equipment or materials that will allow pollutants to enter the stormwater system.

- **MR-4** - Preservation of Natural Drainage Systems and Outfalls
- This site requires a "Critical Area Land Use Permit" due to steep slopes. Topography on the site slopes from NW to SE (toward Lake Sammamish.) As shown on ***Figure 4*** -there is approximately 31-feet of drop from the carport area at the top of the site to the bulkhead at Lake Sammamish. There are

provides standards for off-site analyses, including when a downstream analysis is required, the level of analysis that must be performed, and documentation requirements.

D1-04.2(e) Minimum Requirement #5 - On-site Stormwater Management

Projects shall employ On-site Stormwater Management BMPs in accordance with the following projects thresholds, standards, and lists to infiltrate, disperse, and retain stormwater runoff on-site to the extent feasible without causing flooding or erosion impacts.

Project Thresholds

Different compliance paths for meeting Minimum Requirement #5 are available depending on whether the project triggers all nine minimum requirements or whether the project triggers Minimum Requirements #1-5 only. Projects that trigger Minimum Requirements #1-5 only shall either:

- Use On-site Stormwater Management BMPs from List #1 (see List #1 provided below) for all surfaces within each type of surface in List #1; or
- Demonstrate compliance with the LID Performance Standard (described below). Projects selecting this option cannot use Rain Gardens. They may choose to use Bioretention BMPs as described in Chapter D5 of this manual and Chapter 7 of Volume V of the DOE Manual to achieve the LID Performance Standard.

Projects for which all nine minimum requirements apply shall either:

- Demonstrate compliance with the Performance Standard and BMP T5.13; OR
- Use On-site Stormwater Management BMPs from List #2 (see List #2 provided below) for all surfaces within each type of surface in List #2.

LID Performance Standard

Stormwater discharges shall match developed discharge durations to pre-developed durations for the range of pre-developed discharge rates from 8% of the 2-year peak flow to 50% of the 2-year peak flow. Refer to the "Standard Flow Control Requirement" section in Minimum Requirement #7 (D1-04.2(g)) for information about the assignment of the pre- developed condition. Project sites that must also meet Minimum Requirement #7 must match flow durations between 8% of the 2-year flow through the full 50-year flow.

List #1: On-site Stormwater Management BMPs for Projects Triggering Minimum Requirements #1 through #5

Feasibility shall be determined by evaluation against:

1. Design criteria, limitations, and infeasibility criteria identified for each BMP in Chapter D5 and Appendices D2, D9, and D10 of these Standards; and
2. Competing Needs Criteria listed in Chapter 5 of Volume V of the DOE Manual.

FIGURE 7

11/14

For lawn and landscaped areas, roofs, and other hard surfaces, consider the BMPs in the order listed below for that type of surface. Use the first BMP that is considered feasible for each surface. No other On-site Stormwater Management BMP is necessary for that surface.

Lawn and landscaped areas:

1. Post-Construction Soil Quality and Depth in accordance with Chapter D5 of this manual and BMP T5.13 in Chapter 5 of Volume V of the DOE Manual.

Roofs:

1. Full Dispersion in accordance with Chapter D5 of this manual and BMP T5.30 in Chapter 5 of Volume V of the DOE Manual, or Downspout Full Infiltration Systems in accordance with Chapter D5 of this manual and BMP T5.10A in Section 3.1.1 in Chapter 3 of Volume III of the DOE Manual.
2. Rain Gardens in accordance with Chapter D5 of this manual and BMP T5.14A in Chapter 5 of Volume V of the DOE Manual, or Bioretention in accordance with Chapter D5 of this manual and Chapter 7 of Volume V of the DOE Manual. The rain garden or bioretention facility must have a minimum horizontal projected surface area below the overflow which is at least 5% of the area draining to it.
3. Downspout Dispersion Systems in accordance with Chapter D5 of this manual and BMP T5.10B in Section 3.1.2 in Chapter 3 of Volume III of the DOE Manual.
4. Perforated Stub-out Connections in accordance with Chapter D5 of this manual and BMP T5.10C in Section 3.1.3 in Chapter 3 of Volume III of the DOE Manual.

Other Hard Surfaces:

1. Full Dispersion in accordance with Chapter D5 of this manual and BMP T5.30 in Chapter 5 of Volume V of the DOE Manual.
2. Permeable pavement[1] in accordance with Chapter D5 of this manual and BMP T5.15 in Chapter 5 of Volume V of the DOE Manual, or Rain Gardens in accordance with Chapter D5 of this manual and BMP T5.14 in Chapter 5 of Volume V of the DOE Manual, or Bioretention in accordance with Chapter D5 of this manual and Chapter 7 of Volume V of the DOE Manual. The rain garden or bioretention facility must have a minimum horizontal projected surface area below the overflow which is at least 5% of the area draining to it.
3. Sheet Flow Dispersion in accordance with Chapter D5 of this manual and BMP T5.12, or Concentrated Flow Dispersion in accordance with Chapter D5 of this manual and BMP T5.11 in Chapter 5 of Volume V of the DOE Manual.

List #2: On-site Stormwater Management BMPs for Projects Triggering Minimum Requirements #1 through #9

For each surface (lawn and landscaped areas, roofs, and other hard surfaces), consider the BMPs in the order listed for that type of surface. Use the first BMP that is considered feasible. No

[1] This is not a requirement to pave these surfaces. Where pavement is proposed, it must be permeable to the extent feasible unless full dispersion is employed.

three tiered rock walls between the carport (elevation 65) to the main floor level of the existing house at elevation 59 +/-. Although there are no "natural drainage systems" on the site, there is an existing piped drainage system that conveys water from the house to lake (passing through the bulkhead.) This existing piped drainage system will remain in place and reused for the new home.

- **MR-5** - On-Site Stormwater Management
- In accordance with ***Figure 7 - List #1 On-Site Stormwater Management BMPs*** Due to steep slopes, poor soils conditions, and shallow groundwater, infiltration evaluation is not required to be considered (see ***Figure 1***.) ***"Post Construction Soil Quality and Depth (BMP T5-13), "Vegetated Roof" and stormwater "dispersion" are proposed.***

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III. PROPOSED CONDITIONS AND PERMANENT STORMWATER CONTROL PLAN

A. On-Site Stormwater Management BMPs

As noted above in the discussion of **MR-5**, infiltration analysis is not required (see ***Figure 1***) The project is required to address stormwater LID features in "List 1" - See ***Figure 7.***

1.) **Lawn and Landscape Areas:**
a.) ***BMP T5.13*** - ***USED*** - post construction soil amendment is listed and soil mixture (Seattle) Specification is provided on the plans.

2.) **Roofs:**
a.) Evaluation of ***"Full Dispersion" - BMP T5.30*** - ***NOT USED.*** This is not applicable to this site. It is not possible to leave the site in a state of 65% "native vegetation" due to prior extensive development of the site.
b.) Evaluation of Downspout ***'Full Infiltration" - BMP T5.10A - NOT USED.*** Infiltration is not feasible on this site. See ***Figure 1.***
c.) Evaluation of ***"Bioretention Planter Boxes" - BMP T7.30 - NOT USED.*** A "Bioretention Planter Box" was considered and would have been located at the upper (west) side of the lot due to the roof sloping to the west. This option was eliminated due to potential flooding concerns (overflows) and proximity to two occupied levels of the home to the east.
d.) ***"Downspout dispersion systems"*** - ***NOT USED*** - were considered for the roof area but upon review of the geotechnical report, the existing fill soils are a major concern and lack of infiltration capacity would create unpleasant unpassable lawn surfaces during inclement weather.
e.) ***Perforated Stub outs -USED*** - are proposed for the roof area prior to connecting to the existing drainage system into the lake. Furthermore, a ***"Vegetated Roof" - USED - will be incorporated over the entire roof area*** prior to the connection to the perforated stub out. Stormwater Continuous Modeling of the vegetated roof is not required since it is only about 2,200 square feet.

3.) **Other Hard Surfaces:**
- Evaluation of ***"Full Dispersion" - BMP T5.30*** - ***NOT USED.*** This is not applicable to this site. It is not possible to leave the site in a state of 65% "native vegetation."
- Evaluation of ***"Permeable Pavement"*** - ***BMP T5.15 - NOT USED.*** "Permeable pavement" was considered for other hard surfaces. But upon review of the geotechnical report, the existing fill soils are a major concern, relatively shallow groundwater, and lack of infiltration capacity would create unpleasant unpassable lawn surfaces during inclement weather on the downstream side of the permeable surface.
- ***Rain Gardens and Bioretention*** - ***NOT USED*** - were considered for these areas but due to existing fill soils on the site, again, this option was eliminated.
- ***Sheet flow dispersion*** - ***NOT USED*** - was considered for these areas but due to existing fill soils on the site and possibility of fouling lawn areas was eliminated. ***Runoff from these hard surface areas will be captured in traditional catch basins and routed through perforated stub-outs prior to connection to the existing drain passing into Lake Sammamish.***

IV. CONSTRUCTION STORMWATER POLLUTION PREVENTION PLAN (SWP) PLAN

In accordance with City of Bellevue Standards for a single family residence, the ***"Construction Stormwater Pollution Prevention Plan (CSWPPP) Short Form for Small Construction Projects."***