DEVELOPMENT SERVICES DEPARTMENT
ENVIRONMENTAL COORDINATOR
450 110th Ave NE
BELLEVUE, WA 98009-9012

DETERMINATION OF NON-SIGNIFICANCE

PROPONENT: Jim Shannon, Hart Crowser

LOCATION OF PROPOSAL: 9201 SE Shoreland Place

DESCRIPTION OF PROPOSAL: Removal of an existing deck and stairway and addition of 426 square feet to an existing single-family residence on a steep slope critical area.

FILE NUMBERS: 14-140143-LO **PLANNER:** Reilly Pittman

The Environmental Coordinator of the City of Bellevue has determined that this proposal does not have a probable significant adverse impact upon the environment. An Environmental Impact Statement (EIS) is not required under RCW 43.21C.030(2)(C). This decision was made after the Bellevue Environmental Coordinator reviewed the completed environmental checklist and information filed with the Land Use Division of the Development Services Department. This information is available to the public on request.

☐ There is no comment period for this DNS. There is a 14-day appeal period. Only persons who submitted written comments before the DNS was issued may appeal the decision. A written appeal must be filed in the City Clerk's office by 5:00 p.m. on _____.

☒ This DNS is issued after using the optional DNS process in WAC 197-11-355. There is no further comment period on the DNS. There is a 14-day appeal period. Only persons who submitted written comments before the DNS was issued may appeal the decision. A written appeal must be filed in the City Clerk's Office by 5 p.m. on **9/17/2015**

☐ This DNS is issued under WAC 197-11-340(2) and is subject to a 14-day comment period from the date below. Comments must be submitted by 5 p.m. on _____. This DNS is also subject to appeal. A written appeal must be filed in the City Clerk's Office by 5:00 p.m. on _____.

This DNS may be withdrawn at any time if the proposal is modified so as to have significant adverse environmental impacts; if there is significant new information indicating a proposals probable significant adverse environmental impacts (unless a non-exempt license has been issued if the proposal is a private project): or if the DNS was procured by misrepresentation or lack of material disclosure.

Environmental Coordinator

9/3/2015
Date

OTHERS TO RECEIVE THIS DOCUMENT:

☐ State Department of Fish and Wildlife / Stewart.Reinbold@dfw.gov; Christa.Heller@dfw.wa.gov;
☒ State Department of Ecology, Shoreline Planner N.W. Region / Jobu461@ecy.wa.gov; sepaunit@ecy.wa.gov
☐ Army Corps of Engineers Susan.M.Powell@nws02.usace.army.mil
☐ Attorney General ecyolyef@atg.wa.gov
☐ Muckleshoot Indian Tribe Karen.Walter@muckleshoot.nsn.us; Fisheries.fileroom@muckleshoot.nsn.us

City of Bellevue
Development Services Department
Land Use Staff Report

Proposal Name:	9201 SE Shoreland Place Remodel
Proposal Address:	9201 SE Shoreland Pl.
Proposal Description:	Land Use review of a Critical Areas Land Use Permit proposing a 426 square foot house addition in place of an existing deck and other landscape changes within a steep slope critical area on a property adjacent to Lake Washington
File Number:	14-140143-LO
Applicant:	Jim Shannon, Hart Crowser
Decisions Included	Critical Areas Land Use Permit (Process II. 20.30P)
Planner:	Reilly Pittman, Land Use Planner
State Environmental Policy Act Threshold Determination:	**Determination of Non-Significance** Carol V. Helland, Environmental Coordinator Development Services Department
Director's Decision:	**Approval with Conditions** Michael A. Brennan, Director Development Services Department By: _____ Carol V. Helland, Land Use Director

Application Date:	August 22, 2014
Complete Application:	September 12, 2014
Notice of Application Date:	September 18, 2014
Decision Publication Date:	September 3, 2015
Project/SEPA Appeal Deadline:	September 17, 2014

For information on how to appeal a proposal, visit Development Services Center at City Hall or call (425) 452-6800. Comments on State Environmental Policy Act (SEPA) Determinations can be made with or without appealing the proposal within the noted comment period for a SEPA Determination. Appeal of the Decision must be received in the City's Clerk's Office by 5 PM on the date noted for appeal of the decision.

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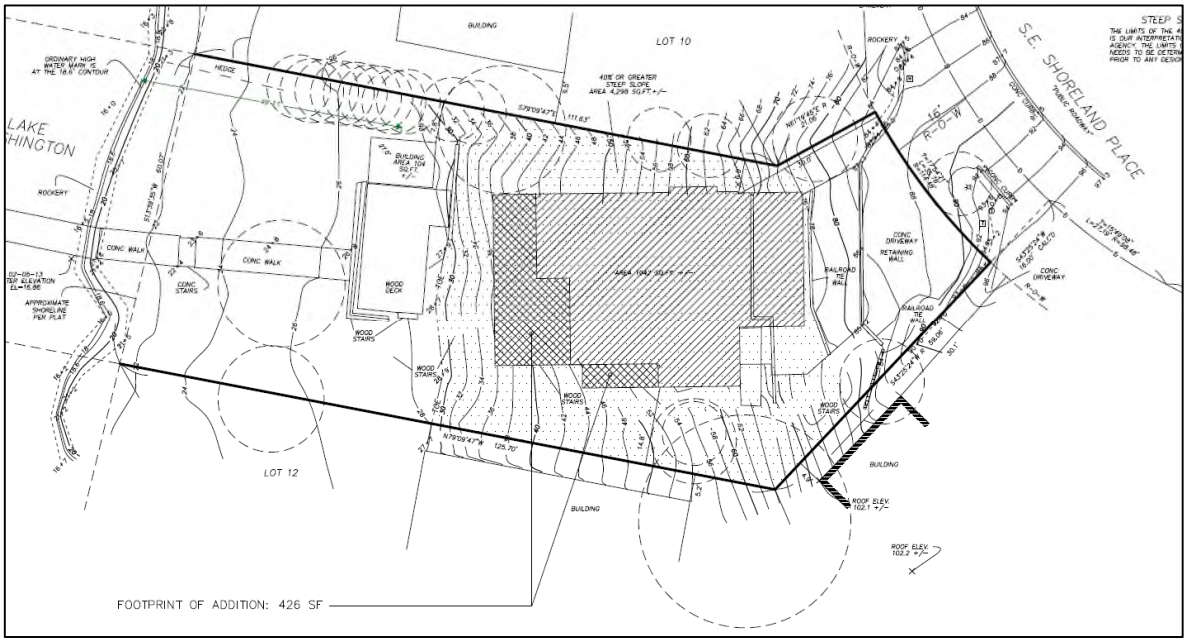
Attachments

1. Site Plan – Enclosed
2. Geotech Report – Enclosed
3. Critical Areas Study, Mitigation and Monitoring Plan – Enclosed
4. SEPA Checklist, Forms and application materials – In File

I. Proposal Description

The project proposes to remove an existing elevated deck and stairway associated with an existing single family residence within a steep slope critical area. The improvements will be replaced with additions to the home that will expand the house footprint by 426 square feet, within the steep slope critical area. Temporary disturbance of the slope around the house is also anticipated from construction and removal of the stair that provides access from the upper property down to the shoreline. Areas of temporary disturbance are proposed to be restored and mitigation is proposed along the shoreline for impacts to the steep slope critical area and removal of a large big leaf maple which is currently within a few feet of the existing house. See figure 1 below for the site plan proposed.

Figure 1

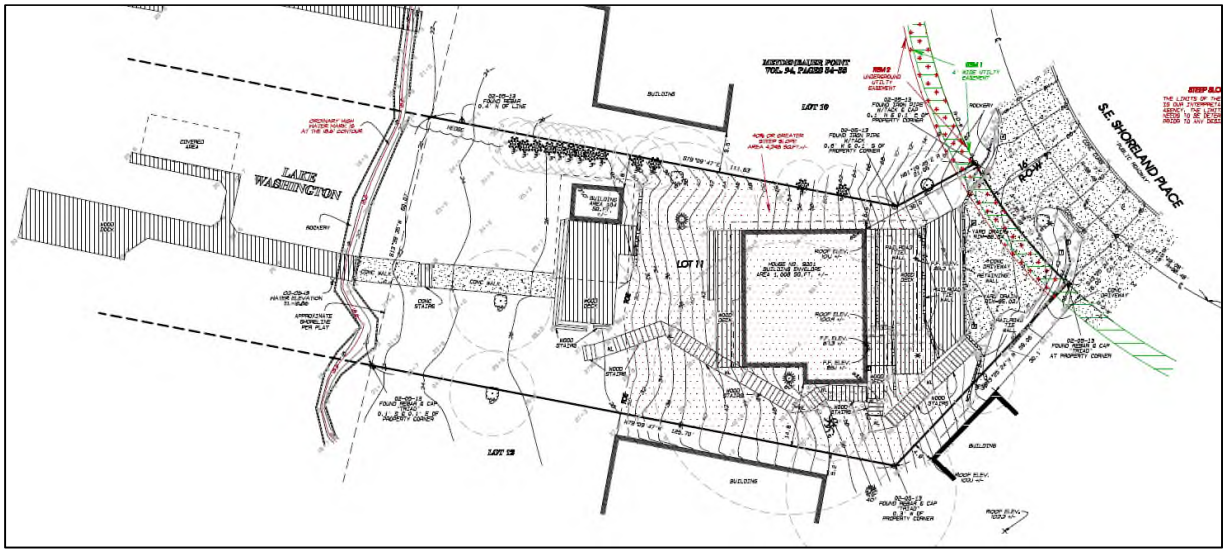


II. Site Description, Zoning, Land Use and Critical Areas

A. Site Description

The project site is located at 9201 SE Shoreland Place the Southwest Bellevue subarea of the City. Other single-family zoned and developed properties are located to the north and south. Shoreland Place is adjacent to the east where the property has a steep driveway access to connect to the road. The concrete driveway turns into an elevated deck that provides parking area and access to the garage attached to the house. Lake Washington is adjacent to the western property boundary. The steep slope is found in the middle portion of the site and surrounds the existing house, which is located on the slope. There is an existing detached structure (cabana) that is proposed to remain which is located below the toe-of-slope, between the lake and the house. See figure 2 for existing site condition.

Figure 2



B. Zoning

The property is zoned R-4, single-family residential which allows the proposed single-family development.

C. Land Use Context

The property has a Comprehensive plan Land Use Designation of SF-H (Single Family High Density). The proposed improvements are consistent with this residential land use.

D. Critical Areas On-Site and Regulations

i. Geologic Hazard Areas

Geologic hazards pose a threat to the health and safety of citizens when commercial, residential, or industrial development is inappropriately sited in areas of significant hazard. Some geologic hazards can be reduced or mitigated by engineering, design, or modified construction practices. When technology cannot reduce risks to acceptable levels, building in geologically hazardous areas is best avoided (WAC 365-190).

Steep slopes may serve several other functions and possess other values for the City and its residents. Several of Bellevue's remaining large blocks of forest are located in steep slope areas, providing habitat for a variety of wildlife species and important linkages between habitat areas in the City. These steep slope areas also act as conduits for groundwater, which drains from hillsides to provide a water source for the City's wetlands and stream systems. Vegetated steep slopes also provide a visual amenity in the City, providing a "green" backdrop for urbanized areas enhancing property values and buffering urban development.

ii. Shorelines

Shorelines provide a variety of functions including shade, temperature control, water purification, woody debris recruitment, channel, bank and beach erosion, sediment delivery, and terrestrial-based food supply (Gregory et al. 1991; Naiman et al. 1993; Spence et al.1996).

Shorelines provide a wide variety of functions related to aquatic and riparian habitat, flood control and water quality, economic resources, and recreation, among others. Each function is a product of physical, chemical, and biological processes at work within the overall landscape. In lakes, these processes take place within an integrated system (ecosystem) of coupled aquatic and riparian habitats (Schindler and Scheuerell 2002). Hence, it is important to have an ecosystem approach which incorporates an understanding of shoreline functions and values.

iii. Critical Areas Overlay District/Critical Area Land Use Permit

A Critical Area Land Use Permit (CALUP) is required as the applicant is requesting to impact steep slope critical areas. In addition to meeting general zoning requirements, the applicant is required to prepare a critical areas report and geotechnical report to show how the project is meeting performance standards for construction in geologically hazardous areas and decision criteria in LUC 20.25H and LUC 20.30P.

III. Consistency with Land Use Code Requirements:

A. Zoning District Dimensional Requirements:

The R-4 zoning dimensional requirements found in LUC 20.20.010 apply to the proposed home construction. The plans submitted generally demonstrate conformance with zoning dimensional standards, however conformance will be verified during building permit review. Specifically the proposed structural lot coverage is proposed to be near the maximum 35 percent coverage allowed. If the proposal is found to exceed the lot coverage the amount of structure proposed on the site may need to be reduced. A survey of the property during construction will likely be required to confirm the site conforms to the limit on structural lot coverage. The house will also be required to meet the limits on height for single family structure, including the limit on maximum façade height which is 40 feet from the lowest point to the highest point visible on any façade. Of primary concern will be the façade facing down slope toward the lake. Any freestanding retaining walls or rockeries are limited in height to 30 inches if they are located in any structure setbacks unless the wall will allow less slope

disturbance.
This Critical Areas Land Use Permit only allows the described modifications to critical areas and does not approve any non-conforming zoning dimensional standards. If the proposed construction is reviewed under the building permit and found to be non-conforming to requirements in LUC 20.20.010 the plans will be required to be changed. Revisions may require additional critical areas permitting if they increase the amount of disturbance or modification of critical areas and buffers. See Conditions of Approval in Section X of this report

B. Critical Areas Requirements LUC 20.25H:
The City of Bellevue Land Use Code Critical Areas Overlay District (LUC 20.25H) establishes performance standards and procedures that apply to development on any site which contains in whole or in part any portion designated as critical area, critical area buffer or structure setback from a critical area or buffer. The project area is within a steep slope critical area and is subject to the performance standards found in LUC 20.25H as specified in the table below

Critical Area	**Geologic Hazard-Steep Slopes**
Performance Standards	20.25H.125 20.25H.145 20.25H.230

i. Consistency with LUC 20.25H.125
Development within a landslide hazard or steep slope critical area or the critical area buffers of such hazards shall incorporate the following additional performance standards in design of the development, as applicable. The requirement for long-term slope stability shall exclude designs that require regular and periodic maintenance to maintain their level of function.

1. **Structures and improvements shall minimize alterations to the natural contour of the slope, and foundations shall be tiered where possible to conform to existing topography;**
   The proposed house expansion is located in areas already disturbed by the deck and stairs. The proposed house foundation under the new additions is tiered.

2. **Structures and improvements shall be located to preserve the most critical portion of the site and its natural landforms and vegetation;**
   The house is already located within the steep slope critical area. The proposed addition is within already disturbed areas where there is no vegetation. The most important critical area on the property is the shoreline which is avoided by the project. A large big leaf maple tree is proposed for removal as it is within feet of the current house and has reached its maximum growth potential. The tree will become a hazard as it declines and removal is proposed to allow a clear zone around the house.

3. **The proposed development shall not result in greater risk or a need for increased buffers on neighboring properties;**
   The project geotechnical engineer has found that the proposal will not result in greater risk or the need for increased buffers on adjacent property. The geotech found the project proposal will increase slope stability.

4. **The use of retaining walls that allow the maintenance of existing natural slope area is preferred over graded artificial slopes where graded slopes would result in increased disturbance as compared to use of retaining wall;**
   The geotech found that graded artificial slopes would result in increased slope disturbance and would decrease slope stability and so they are not proposed. Retaining walls are proposed as part of the house foundation but no freestanding walls are shown on the proposed plans.

5. **Development shall be designed to minimize impervious surfaces within the critical area and critical area buffer;**
   The proposed house addition is the only new impervious surface proposed. All drainage will be collected and dispersed away from the steep slope.

6. **Where change in grade outside the building footprint is necessary, the site retention system should be stepped and regrading should be designed to minimize topographic modification. On slopes in excess of 40 percent, grading for yard area may be disallowed where inconsistent with this criteria;**
   No permanent grading is proposed outside of the building footprint. Temporary disturbance may result from construction.

7. **Building foundation walls shall be utilized as retaining walls rather than rockeries or retaining structures built separately and away from the building wherever feasible. Freestanding retaining devices are only permitted when they cannot be designed as structural elements of the building foundation;**
   Per the geotech report, the building foundation walls will be utilized as retaining walls.

8. **On slopes in excess of 40 percent, use of pole-type construction which conforms to the existing topography is required where feasible. If pole-type construction is not technically feasible, the structure must be tiered to conform to the existing topography and to minimize topographic modification;**
   Micropiles or pin piles will be used for the proposed construction. Tiebacks and soil nails are also proposed to stabilize the slope.

9. **On slopes in excess of 40 percent, piled deck support structures are required where technically feasible for parking or garages over fill-based construction types; and**
   The existing parking area and garage are on a piled deck. No changes are proposed to the parking area.

10. **Areas of new permanent disturbance and all areas of temporary disturbance shall be mitigated and/or restored pursuant to a mitigation and restoration plan meeting the requirements of LUC 20.25H.210.**
    The project proposes to restore all areas of temporary disturbance and provides mitigation planting along the shoreline as described later in this report.

ii. Consistency With LUC 20.25H.230 and LUC 20.25H.145

Modification of steep slopes requires a critical areas report as part of the application for a Critical Areas Land Use Permit. The applicant has obtained the services of a qualified geotechnical engineering consultant to study the site and document the observed conditions. Staff has reviewed the following documents:

- Geotechnical Engineering Report revised May 14, 2015 prepared by Hart Crowser, Inc.

This geotechnical analysis finds that the proposal will "decrease the risk" associated the steep slopes and will "mitigate the hazard level equal to or less than would exist" that if the slopes were unchanged (pg. 30). Per LUC 20.30P.170, approval of projects to locate or modify buffers, setbacks, or the steep slopes critical areas require the proponent to complete a Hold Harmless Agreement with the City. The agreement is required to be completed prior to building permit issuance on a form provided by the City. <u>See Conditions of Approval in Section X of this report.</u>

IV. Public Notice and Comment

Application Date:	August 22, 2014
Public Notice (500 feet):	September 18, 2014
Minimum Comment Period:	October 2, 2014

The Notice of Application for this project was published the City of Bellevue Weekly Permit Bulletin and Seattle Times on September 18, 2014. It was mailed to property owners within 500 feet of the project site. No comments were received.

V. Summary of Technical Reviews

A. Clearing and Grading

The Clearing and Grading Division of the Development Services Department has reviewed the proposed site development for compliance with Clearing and Grading codes and standards. The Clearing and Grading staff found no issues with the proposed development and has approved the application.

VI. State Environmental Policy Act (SEPA)

The environmental review indicates no probability of significant adverse environmental impacts occurring as a result of the proposal. The Environmental Checklist submitted with

the application adequately discloses expected environmental impacts associated with the project. The City codes and requirements, including the Clear and Grade Code, Utility Code, Land Use Code, Noise Ordinance, Building Code and other construction codes are expected to mitigate potential environmental impacts. Therefore, issuance of a Determination of Non-Significance (DNS) is the appropriate threshold determination under the State Environmental Policy Act (SEPA) requirements.

A. Earth, Air, and Water

No dredging, withdrawals, diversions, or discharges are anticipated from the proposed improvements in the steep slope. Erosion and sedimentation control requirements and BMPs will be reviewed by the Clearing and Grading Department as part of a clearing and grading permit.

B. Animals

Lake Washington contains coho (Species of Concern), chinook (Threatened), steelhead (Threatened), and potentially contains bull trout (Threatened). No structures are proposed in the lake, buffer or setback. Vegetation will be planted within the 25-foot shoreline buffer as mitigation for the proposed slope impacts.

C. Plants

Existing ornamental planting, invasive species, and lawn will be impacted by the improvements in the steep slope and shoreline setback. One significant big leaf maple tree will be removed as it is within a few feet of the house, limits access and will become a hazard as this mature tree declines. The resulting planting will establish native trees, shrubs, and ground covers within the shoreline buffer of Lake Washington.

D. Noise

The site is adjacent to single-family residences whose residents are most sensitive to disturbance from noise during evening, late night and weekend hours when they are likely to be at home. Construction noise will be limited by the City's Noise Ordinance (Chapter 9.18 BCC) which regulates construction hours and noise levels. <u>See Section X for a related condition of approval.</u>

VII. Changes to Proposal Due to Staff Review

Staff required revisions to the geotech report, critical areas report, and planting plan.

VIII. Decision Criteria

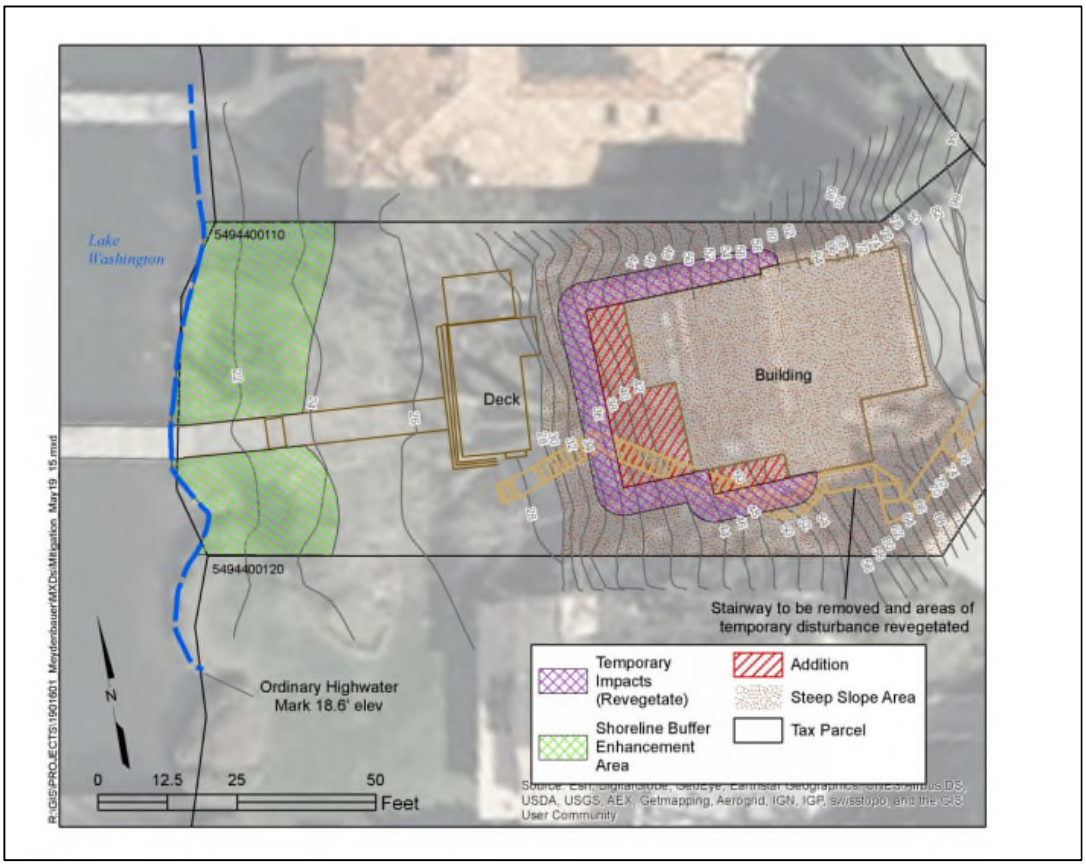
A. 20.25H.255.B Critical Areas Report – Decision Criteria – General

The Director may approve, or approve with modifications, a proposal to reduce the regulated critical area buffer on a site where the applicant demonstrates:

1. **The proposal includes plans for restoration of degraded critical area or critical area buffer functions which demonstrate a net gain in overall critical area or critical area buffer functions;**

The mitigation plan proposes to improve the shoreline by replacing existing lawn and ornamental planting with native planting in exchange for the proposed house addition in the steep slope. On sites where there are more environmentally significant critical areas, such as Lake Washington, mitigation planting can be allowed to improve the more significant critical area, rather than the impacted steep slope in this case. The vegetation provided in the shoreline buffer will enhance the function and value of the buffer above the current condition. See figure 3 below for planting area.

Figure 3



The proposed planting is in the table below and will be located within the 25-foot shoreline buffer and areas of temporary disturbance on the steep slope.

Scientific Name	Common Name	Spacing	Quantity (Shoreline buffer enhancement area)	Quantity (steep slope enhancement area)	Size
Fraxinus latifolia/	Oregon Ash	9´ O.C.	~~6~~ 7	0	2 gallon
Pinus contorta	Shore Pine	9´ O.C.	~~6~~ 7	0	2 gallon
Salix scouleriana	Scouler's Willow	9´ O.C.	~~6~~ 7	0	2 gallon
Acer macrophyllum	Big-leaf maple	9´ O.C.	0	~~3~~ 4	2 gallon
Alnus rubra	Red alder	9´ O.C.	0	~~3~~ 4	2 gallon
Pseudotsuga menziesii	Douglas fir	9´ O.C.	0	~~3~~ 4	2 gallon

HARTCROWSER

19016-01
May 19, 2015

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Scientific Name	Common Name	Spacing	Quantity (Shoreline buffer enhancement area)	Quantity (steep slope enhancement area)	Size
Cornus sericea	Red-osier dogwood	4´ O.C.	16	0	1 gallon
Holodiscus discolor	Oceanspray	4.5´ O.C.	10	~~4~~ 6	1 gallon
Philadelphus lewisii/	Mock orange	4.5´ O.C.	10	~~4~~ 6	1 gallon
Rubus parviflorus	Thimbleberry	3.5´ O.C.	16	~~8~~ 12	1 gallon
Corylus cornuta	Beaked Hazelnut	6' O.C.	0	~~4~~ 6	1 gallon
Symphoricarpos albus	Snowberry	4.5´ O.C.	10	~~6~~ 9	1 gallon
Festuca idahoensis	Idaho fescue	2´ O.C.	16	~~10~~ 15	1 gallon
Arctostaphyllos uva-ursi	Kinnikinnick	2´ O.C.	32	~~14~~ 21	1 gallon

Note:

2. **The proposal includes plans for restoration of degraded critical area or critical area buffer functions which demonstrate a net gain in the most important critical area or critical area buffer functions to the ecosystem in which they exist;**
   The steep slope on this site is disturbed by existing improvements, landscaping, and invasive species. The proposal includes the planting of several trees, some of which will be located in close proximity to the lake. These trees will provide opportunity for future habitat usage as perch trees for avian species that are frequently found near the lake and will provide opportunity for debris input. Given the conditions of the site and adjacent properties the improvement of shoreline vegetation providing habitat on this site is the most important function which can be restored on the site.

3. **The proposal includes a net gain in stormwater quality function by the critical**

area buffer or by elements of the development proposal outside of the reduced regulated critical area buffer;
Stormwater quality will be improved by the planting along the shoreline.

4. **Adequate resources to ensure completion of any required restoration, mitigation and monitoring efforts;**
The mitigation planting will be maintained and monitored for a period of at least five years as described in critical areas study. A maintenance surety will be required based on an estimate of the installed cost of planting which includes labor, materials, contingency, and monitoring for 5 years. The maintenance surety will be released after the five-year monitoring, assuming restoration has been successful. See Conditions of Approval in Section X of this report.

5. **The modifications and performance standards included in the proposal are not detrimental to the functions and values of critical area and critical area buffers off-site; and**
The modifications in this proposal are not detrimental to the functions and values of the slopes on or off-site and are not inconsistent with what other adjacent properties already have constructed.

6. **The resulting development is compatible with other uses and development in the same land use district.**
The proposed improvements are compatible with residential uses.

B. 20.30P.140 Critical Area Land Use Permit Decision Criteria – Decision Criteria
The Director may approve, or approve with modifications an application for a Critical Area Land Use Permit if:

1. **The proposal obtains all other permits required by the Land Use Code;**
The applicant must obtain a building permit and any other development permits. See Conditions of Approval in Section X of this report.

2. **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;**
The proposed improvements are located in already disturbed locations or are replacing existing improvements.

3. **The proposal incorporates the performance standards of Part 20.25H to the maximum extent applicable, and ;**
As discussed in Section III of this report, the applicable performance standards of LUC Section 20.25H are being met.

4. **The proposal will be served by adequate public facilities including street, fire protection, and utilities; and;**
The site is served by adequate public facilities which are not impacted.

5. **The proposal includes a mitigation or restoration plan consistent with the requirements of LUC Section 20.25H.210; and**
   The mitigation planting is proposed to be consistent with the City's planting templates for steep slopes. The planting and conditions in this staff report make the project consistent with LUC 20.25H.210.

6. **The proposal complies with other applicable requirements of this code.**
   As discussed in this report, the proposal complies with all other applicable requirements of the Land Use Code.

IX. Conclusion and Decision

After conducting the various administrative reviews associated with this proposal, including SEPA, Land Use Code consistency, City Code and Standard compliance reviews, the Director of the Development Services Department does hereby **approve with conditions** the 426 square-foot addition on the exiting house and temporary disturbance located within a steep slope critical area. **Approval of this Critical Areas Land Use Permit does not constitute a permit for construction. A building permit, clear and grade permit, and/or utility permit is required and all plans are subject to review for compliance with applicable City of Bellevue codes and standards.**

Note- Expiration of Approval: In accordance with LUC 20.30P.150 a Critical Areas Land Use Permit automatically expires and is void if the applicant fails to file for a building permit or other necessary development permits within one year of the effective date of the approval.

X. Conditions of Approval

The applicant shall comply with all applicable Bellevue City Codes and Ordinances including but not limited to:

Applicable Ordinances	Contact Person
Clearing and Grading Code- BCC 23.76	Savina Uzunow, 425-452-7860
Land Use Code- BCC Title 20	Reilly Pittman, 425-452-4350
Noise Control- BCC 9.18	Reilly Pittman, 425-452-2973

The following conditions are imposed under the Bellevue City Code or SEPA authority referenced:

1. **Building Permit:** Approval of this Critical Areas Land Use Permit does not constitute an approval of a development permit. Application for a building permit or other required permits must be submitted and approved. Plans submitted as part of either permit application shall be consistent with the activity permitted under this approval.

   Authority: Land Use Code 20.30P.140
   Reviewer: Reilly Pittman, Development Services Department

2. **Conformance with LUC 20.20.010:** This approval does not guarantee the project for conformance with zoning requirements in LUC 20.20. Additional critical areas permitting may be required if any changes required by zoning will increase impacts to critical areas.

   Authority: Land Use Code 20.20.010; Land Use Code 20.30P.140
   Reviewer: Reilly Pittman, Development Services Department

3. **Height Limits:** The proposed changes to the house are required to be less than 35 feet in height measured from the average existing grade. No façade shall exceed 40 feet from where the building wall meets existing grade to the highest point visible on each façade.

   Authority: Land Use Code 20.20.010; Land Use Code 20.25E.080
   Reviewer: Reilly Pittman, Development Services Department

4. **Maintenance and Monitoring:** The submitted monitoring plan with goals, objectives, performance standards, and schedule (Attachment 3) is required to be carried out for 5 years. The yearly monitoring reports should be mailed to:

   Environmental Planning Manager
   Development Services Department
   City of Bellevue
   PO Box 90012
   Bellevue, WA 98009-9012

   Authority: Land Use Code 20.30P.140
   Reviewer: Reilly Pittman, Development Services Department

5. **Cost Estimate:** A cost estimate of the cost to carry out maintenance and monitoring for 5 years is required prior to building permit issuance. This cost estimate will be used to determine the amount of the maintenance surety required. If planting installation will be delayed an installation surety and cost estimate for planting will be required.

   Authority: Land Use Code 20.25H.255
   Reviewer: Reilly Pittman, Development Services Department

6. **Maintenance Surety:** The maintenance surety is required to be held until completion of the 5-year monitoring. Release of this surety is contingent upon successful monitoring based on the established goals and objectives. The amount of the surety must cover 100 percent of the cost for maintenance and monitoring for 5 years. Land Use inspection of the planting after 5-years is required to release the surety.

   Authority: Land Use Code 20.30P.140
   Reviewer: Reilly Pittman, Development Services Department

7. **Land Use Inspection:** Following installation of planting the applicant shall contact Land

Use staff to inspect the planting area and release the installation surety. Inspection by Land Use staff is required to release the maintenance surety at the end of five years of monitoring. Staff will need to find that the plants are in a healthy and growing condition and the mitigation plan is successful per the established performance standards in the monitoring plan.

Authority: Land Use Code 20.30P.140
Reviewer: Reilly Pittman, Development Services Department

8. **Hold Harmless Agreement:** The applicant shall submit a hold harmless agreement in a form approved by the City Attorney which releases the City from liability for any damage arising from the location of improvements within a critical area buffer in accordance with LUC 20.30P.170. The hold harmless agreement is required to be recorded with King County prior to building permit issuance. Staff will provide the applicant with the hold harmless form.

   Authority: Land Use Code 20.30P.170
   Reviewer: Reilly Pittman, Development Services Department

9. **Noise Control:** Noise related to construction is exempt from the provisions of BCC 9.18 between the hours of 7 am to 6 pm Monday through Friday and 9 am to 6 pm on Saturdays, except for Federal holidays and as further defined by the Bellevue City Code. Noise emanating from construction is prohibited on Sundays or legal holidays unless expanded hours of operation are specifically authorized in advance. Requests for construction hour extension must be done in advance with submittal of a construction noise expanded exempt hours permit.

   Authority: Bellevue City Code 9.18
   Reviewer: Reilly Pittman, Development Services Department

GRAPHIC SCALE 1"=10'

0 10 20 30

MEYDENBAUER POINT
VOL. 94, PAGES 54-55

LOT 10

BUILDING

ITEM 1
4' WIDE UTILTY
EASEMENT

ROCKERY

S.E. SHORELAND PLACE
"PUBLIC ROADWAY"

16' R-O-W

CONC CURB

CONC DRIVEWAY

RETAINING WALL

RAILROAD TIE WALL

CONC DRIVEWAY

R-O-W

S43°25'24"W 16.00' CALC'D

Δ=15°49'08" L=27.19' R=98.48'

S62°23'44"E (30.75' PLAT 94/54-55) 50.75' CALC'D

S43°25'24"W R 59.06'

30.1'

N61°19'45"E R 21.06

S79°09'47"E 111.63'

N79°09'47"W 125.70'

40% OR GREATER
STEEP SLOPE
AREA 4,298 SQ.FT.+/-

27.0' BUILDING
AREA 104
SQ.FT.
+/-

BUILDING ENVELOPE
AREA 1,008 SQ.FT. +/-

WOOD DECK

WOOD STAIRS

WOOD STAIRS

WOOD STAIRS

WOOD STAIRS

TOE

TREE
TO BE REMOVED

BUILDING

BUILDING

ROOF ELEV.
102.1 +/-

ROOF ELEV.
102.2 +/-

LOT 12

LAKE
WASHINGTON

ORDINARY HIGH
WATER MARK IS
AT THE 18.6' CONTOUR

HEDGE

ROCKERY

S13°39'35"W 60.07'

CONC WALK

CONC WALK

CONC
STAIRS

02-05-13
WATER ELEVATION
EL=16.86

APPROXIMATE
SHORELINE
PER PLAT

BASIS OF BEARINGS
N27°38'16"E
(247.19')

02-05-13
FOUND CONC
MON IN CASE
BRASS DISC
DOWN 0.9'

STEEP SLOPE/BUFFER DISCLAIMER

THE LIMITS OF THE 40 % AS SHOWN ON THIS DRAWING IS OUR INTERPRETATION WHICH MAY DIFFER FROM THAT OF THE REVIEWING AGENCY. THE LIMITS OF THE 40% SLOPES AND ASSOCIATED SETBACKS NEEDS TO BE DETERMINED BY THE RESPECTIVE REVIEWING AGENCY, PRIOR TO ANY DESIGN AND OR CONSTRUCTION TAKING PLACE.

EXISTING HOUSE FOOTPRINT = 1008 SF
FOOTPRINT OF ADDITION: 426 SF
TOTAL PROPOSED HOUSE FOOTPRINT: 1434 SF
NEW DRIVEWAY = 250 SF
EXISTING CABANA = 104 SF
NEW DECKS = 160 SF
TOTAL COVERAGE = 1948 SF

GROSS LOT AREA = 9225 SF; CRITICAL SLOPE AREA = 4298 SF
CRITICAL SLOPE AREA – EXISTING HOUSE FOOTPRINT = 3290 SF

RESULTING NET LOT AREA = 9225 SF – 3290 SF = 5935 SF
MAX. LOT COVERAGE = 35%; 5935 X 0.35 = 2077 SF
PROPOSED LOT COVERAGE: 1948 / 5935 = 32%

FAR= 0.22

1 PROPOSED SITE PLAN
SCALE: 1/8" = 1'-0"

N

MEYDENBAUER RESIDENCE

9201 S.E. SHORELAND PLACE
BELLEVUE, WA 98004

Chin-Ley/Reche Associates
1516 Melrose Ave, Suite 200
Seattle, Washington 98122
Telephone: 206.938.1501
Fax: 206.935.3449

DATE	DESCRIPTION
09.16.13.	PREAPPLICATION
~~01.29.15~~	~~C.U.P. DESIGN~~
05.07.15.	C.A.R. REVISION.

PROJECT NO.

DATE
05.07.15

DRAWING TITLE
PROPOSED
SITE PLAN

DRAWING NUMBER
PA2.0

Geotechnical Engineering Design Study

9201 SE Shoreland Place

Bellevue, Washington

Prepared for

Everyoung Service, LLC

~~**July 1, 2014**~~

~~**February 11, 2015**~~

May 14, 2015 –

Response to City Comments Round 2

19016-00

HARTCROWSER

HARTCROWSER

www.hartcrowser.com

Geotechnical Engineering Design Study

9201 SE Shoreland Place

Bellevue, Washington

Prepared for

Everyoung Service, LLC

~~July 1, 2014~~
~~February 11, 2015~~
May 12, 2015 –
Response to City Comments Round 2
19016-00

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Geotechnical Engineering Design Study

9201 SE Shoreland Place

Bellevue, Washington

1.0 INTRODUCTION

This report presents our geotechnical engineering design recommendations and fulfills the Critical Areas Report requirements for the Landslide Hazard and Steep Slope Critical Areas for the proposed modifications to 9201 SE Shoreland Place Single Family Residence (SFR) in Bellevue, Washington. A Vicinity Map showing the site location is presented on Figure 1.

This report is divided into several sections. Following Sections 1 and 2, which describe the organization and purpose of this report, our principal geotechnical engineering design recommendations are organized as follows:

- Site and Project Description;
- Subsurface Conditions;
- Seismic Considerations;
- Geotechnical Conclusions and Recommendations;
- Construction Considerations; and
- Recommended Additional Geotechnical Services.

Tables are presented in the text; figures follow the main text along with two attachments and three appendices. Attachments 1 and 2 present our recommendations for tieback testing and shoring monitoring, respectively. Appendices A and B present field exploration methods and logs, and laboratory testing methods and results, respectively. Appendix C presents historical subsurface data collected at the adjacent property to the north of the site.

2.0 PURPOSE, SCOPE, AND USE OF THIS REPORT

The purpose of our work is to provide geotechnical engineering design recommendations and meet the Critical Areas Report requirements for the Landslide Hazard and Steep Slope Critical Areas for the proposed modifications to 9201 SE Shoreland Place SFR in Bellevue, Washington. In this report, we present our findings, conclusions, and recommendations for:

- Subsurface conditions;
- Seismic considerations;
- Temporary shoring;
- Building foundations;
- Structural fill;
- Construction monitoring; and
- Recommendations for additional services.

Our scope of work for this project included:

- Advancing three hollow-stem auger borings;
- Performing slope reconnaissance;
- Testing soil samples in our laboratory;
- Completing geotechnical engineering analyses; and
- Producing this geotechnical engineering design report.

We completed this work in general accordance with our proposal dated March 25, 2014, and authorized on March 27, 2014. This report is for the exclusive use of Everyoung Service, LLC, and its design consultants for specific application to the subject project and site. We completed this work in accordance with generally accepted geotechnical engineering practices for the nature and conditions of the work completed in the same or similar localities, at the time the work was performed. We make no other warranty, express or implied.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Description

The project site is located on a steep slope on the eastern shore of Lake Washington in the Meydenbauer neighborhood of Bellevue, Washington. The site is currently occupied by a two- to three-story, wood-frame, single-family house supported by three rows of wooden post foundations on steeply sloping ground. Figure 2 shows the footprint of the existing structure as well as the ground surface topographic contours.

Site grades in the project area slope steeply to the west toward Lake Washington. Site elevations vary from about elevation 88 feet at the intersection of the driveway and SE Shoreland Place to about 20 feet at the west edge of the site along the shoreline. The steep slope portion of the site is about 50 feet tall with an average slope of about 67 percent (or 34 degrees) and covers an area of about 4,300 square feet.

Note that elevations in this report are referenced to the North American Vertical Datum of 1988 (NAVD88) and were estimated from the site survey by GeoDimensions dated March 1, 2013.

With the exception of the sewer line, utilities servicing the residence appear to be underground from the SE Shoreland Place right of way. The sewer line runs approximately parallel to the Lake Washington shoreline approximately 35 feet west of the toe of the slope. The side sewer servicing the house extends below the house near the west and runs downslope along the south property line to the sewer main.

3.2 Project Description

Current development plans call for renovating the existing SFR with an addition that includes two levels below part of the existing structure and extending over the slope on the downhill side of the house. The footprints of both the existing structure (1516 square feet) and proposed addition (426

square feet) are shown on Figure 2, and the existing and proposed structures are shown in section view on Figure 3. The renovation will include grading and installing new retaining walls and foundations to allow the house to be built on the terraced ground.

4.0 SLOPE RECONNAISSANCE AND SUBSURFACE CONDITIONS

We based our interpretation of subsurface conditions on data obtained from our field exploration and laboratory testing program for the site, as well as from existing borings completed previously by others at the property to the north.

Subsurface conditions at the site are shown in generalized subsurface cross section provided on Figure 3. The location of the cross section is shown on Figure 2. Exploration logs in Appendices A and C should be consulted for a more detailed presentation of subsurface conditions.

Our field exploration and testing program consisted of:

- A slope reconnaissance site visit and
- Three geotechnical borings designated HC-1 through HC-3.

4.1 Slope Reconnaissance

We performed a slope reconnaissance to investigate for signs of slope instability, soil erosion, or groundwater seepage. The slope was generally vegetated with grass, brush, and trees. Maple trees and remaining tree trunks appeared to show slight curvature for trees estimated to be over 50 years old, indicating some slow, near-surface soil creep had occurred. No evidence of significant slope instability or groundwater seepage was apparent during our site visit. Soils were exposed and unvegetated below the post-supported house and deck, and there was evidence of only minor surficial erosion.

The slope is protected from wave erosion by a rock bulkhead on the Lake Washington shoreline approximately 65 feet west of the toe of the slope, and the lake level is about 10 feet below the toe of the slope. Lake Washington water levels are controlled by the US Army Corps of Engineers at the Hiram M. Chittenden Locks in Seattle; lake levels were dropped about 9 feet from historical water levels when the locks were built in 1916 (see Figure 3).

4.2 Generalized Subsurface Soil Conditions

In general, subsurface conditions consist of a limited amount of possible fill and medium stiff to stiff sandy silt over very stiff to hard sandy silt and dense silty sand over very stiff to hard clay. The native silty sand and sandy silt are interpreted to be advance outwash, based on available geologic maps, but could also include transitional glaciolacustrine deposits; the underlying clay is interpreted to be glaciolacustrine. The native soils are glacially overridden.

The site soil units observed in borings are described below from the ground surface downward.

Fill and Medium Stiff to Stiff Sandy Silt. This soil unit was encountered in borings HC- 1 and HC-2 from the ground surface to depths of about 4 to 9 feet and generally consists of medium stiff to stiff sandy silt. In HC-1, this unit is likely fill from site grading during construction of the driveway. In HC-2 below the existing structure, this unit appears to include possible fill over weathered native soils and potential colluvium from historical erosion of the slope above. Scattered organic materials (roots and wood debris) encountered between about 5 to 9 feet deep likely indicate this was the slope surface prior to original construction of the house. In HC-3, this unit is very limited in thickness as the first sample appeared native and transitional to the underlying Very Stiff to Hard Clay.

Hard Sandy Silt and Dense Silty Sand. This soil unit generally consists of very stiff to hard, slightly sandy to very sandy silt and dense very silty sand, which underlies the Fill and Medium Stiff to Stiff Sandy Silt. The majority of the slope is composed of this unit, and its thickness varied from about 54 feet thick at the top of the slope to absent near the bottom of the slope.

Hard Clay. Below the Hard Sandy Silt and Dense Silty Sand is a unit generally consisting of very stiff to hard clay. This unit appeared to have sheared texture and scattered slickensides generally ranging from about 40 to 65 degrees from horizontal. The sheared texture and slickensides are often observed in glacially overridden, glaciolacustrine deposits in the Bellevue/Seattle area. These features are often cause by stress relief that occurred during glacier melting and retreat. The Hard Clay is generally below the base of the existing slope, and the sheared texture and slickensides do not appear to be associated with historical instability of the existing slope.

Please note that the explorations performed for this study reveal subsurface conditions only at discrete locations across the project site and actual conditions in other areas could vary. Furthermore, the nature and extent of any variations would not become evident until additional explorations are performed or until construction begins. If significant variations are observed at that time, we may need to modify our conclusions and recommendations to reflect actual site conditions.

4.3 Groundwater Conditions

In general, soils encountered were moist throughout the height of the slope and even below the elevation of Lake Washington, which ranges from about 16.5 to 18.5 feet, as controlled by the US Army Corps of Engineers. Some wet soil zones were encountered in HC-1 below 35 feet deep, indicating the potential for perched groundwater, but there was no measurable groundwater at the time of drilling.

Water levels were measured at the times and under conditions stated on the boring logs or in the text. Fluctuations in the groundwater conditions may be caused by variations in rainfall, temperature, season, lake level, and other factors.

5.0 GEOLOGIC HAZARD AREAS

The site is mapped as an erosion-sensitive area according to King County, but is not mapped as a landslide hazard area by either King County or Washington State Department of Natural Resources. The site slope is designated as a Steep Slope Critical Area per the City of Bellevue Critical Areas Ordinance – Designation of Critical Area and Buffers (Geologic Hazard Areas) [LUC 20.25H.120], because it is taller than 10 feet, steeper than 40 percent, and covers an area greater than 1,000 square feet.

~~We assume a reasonable use exception will be applied to the site [LUC 20.25H.190] and~~ The proposed project will require an approval of modification in a geologic hazard area [LUC 20.25H.145]. The geotechnical analyses and recommendations provided in this report are based on ~~the Reasonable Use Exception – Performance Standards [LUC 20.25.205], including~~ Performance Standards – Landslide Hazards and Steep Slopes [LUC 20.25H.125], see the following section for a summary of how the proposed project addresses these performance standards. This geotechnical report serves to meet the critical areas report requirements specific to the steep slope critical area, including requirements identified in Critical Areas Report – Additional Provisions for Landslide Hazard Areas and Steep Slopes [LUC 20.25H.140] and the City of Bellevue Geotechnical Report and Stability Analysis Requirements.

In addition to the steep slope, we understand that several other critical area issues will have to be addressed as part of the City of Bellevue permitting process including shoreline, habitat, and steep slope vegetation/retention changes. A separate critical areas report will be submitted to address these environmental issues.

5.1 Performance Standards – Landslide Hazards and Steep Slopes [LUC 20.25H.125]

In this section, we provide a summary of how the proposed project will address the referenced performance standards when designed and constructed in accordance with the geotechnical recommendations provided in the following sections.

The long-term stability of the slope will be improved from its existing condition based on the proposed design, which will include engineered tiebacks or soil nails and foundation walls to increase the slope stability factor of safety to meet code standards.

A. The structures limit grading of the existing slope with a tiered lowest level for the proposed project. Some cuts are necessary for the proposed project but will be limited to about six feet deep and remove looser surficial soils. The foundation walls proposed to support these cuts will be engineered for the appropriate lateral earth pressures, as well as to improve the overall global slope stability to meet code standards. The limited cuts for foundation walls will allow for tiebacks or soil nails to be installed, which are most efficient for providing lateral restraint to stabilize the slope.

B. The proposed addition is primarily below the existing house with some expansion to the west to better conform to the existing topography. Soil that will be removed is looser surficial soil, which is currently exposed or bare soil below the existing structure. Disturbance to vegetation will be minimized during construction and any areas temporarily disturbed beyond the footprint of the proposed structure will be revegetated.

C. The proposed project will not result in greater risk or a need for increased buffers on neighboring properties. As discussed previously, the proposed project will increase site slope stability to meet code standards.

D. If graded artificial slopes were to be used for the proposed project, they would result in increased disturbance and potentially decreased slope stability. Thus, graded artificial slopes will not be used. Retaining walls will be used for necessary grade changes within the footprint of the building and for stabilizing the slope.

E. Additional impervious surfaces will be limited to those associated with the proposed limited expansion of the structure to the west. Stormwater collected from existing and future impervious surfaces will be collected and discharged downslope of the landslide hazard and steep slope area, reducing potential surficial erosion and instability.

F. Grade changes are not proposed outside of the building footprint and temporary disturbance to the steep slope will be minimized. No grading for a yard area is proposed.

G. The building foundation walls will be used as retaining walls.

H. Slopes in excess of 40 percent are present at the site. Deep foundations (i.e., pole-type construction) in the form of mircopiles or pin piles will be used. Shallow foundations will only be considered where very competent soils will be encountered and as part of a foundation wall system designed to increase slope stability to meet the code standards. The limited cuts for foundation walls will allow for tiebacks or soil nails to be installed, which are more efficient for stabilizing the slope than relying on pole-type foundations alone.

I. No additional parking or garage areas are proposed, and no change to the existing parking area is proposed.

J. Areas of new permanent disturbance and temporary disturbance will be mitigated and/or restored as described in the project critical areas report, submitted separately.

6.0 SEISMIC CONSIDERATIONS

In this section, we describe the seismic setting at the project site, provide recommendations to develop the code-based design response spectrum, and discuss seismically induced geotechnical hazards.

6.1 Seismic Setting

The seismicity of western Washington is dominated by the Cascadia Subduction Zone, where the offshore Juan de Fuca plate subducts beneath the continental North American plate. Three main types of earthquakes are typically associated with subduction zone environments: crustal, intraplate, and interplate earthquakes. Seismic records in the Puget Sound area clearly indicate a distinct shallow zone of crustal seismicity, the Seattle Fault, which may have surficial expressions and can extend to depths of 25 to 30 km. A deeper zone is associated with the subducting Juan de Fuca plate and produces intraplate earthquakes at depths of 40 to 70 km beneath the Puget Sound region (e.g., the 1949, 1965, and 2001 earthquakes) and interplate earthquakes at shallow depths near the Washington coast (e.g., the 1700 earthquake with an approximate magnitude of 9.0).

6.2 Seismic Design Criteria

6.2.1 Structures

According to the Bellevue City Code, which adopts the 2012 International Residential Code (IRC), the Seismic Design Category is D2.

If structures will be designed according to the 2012 International Building Code (IBC), the structural engineer uses a design response spectrum to seismically analyze the proposed structure. Two methods may be used to obtain the design response spectrum: (1) a code-based design method based on generic soil properties to develop the spectrum at the ground surface, or (2) a site-specific method that develops the response spectrum based on site-specific soil properties calculated at the depth of interest to the structural engineer. For this project, we assume the design team will use the code-based response spectrum, if necessary. The basis of design for the 2012 IBC is two-thirds of the hazard associated with an earthquake with 2 percent probability of exceedance in 50 years, which corresponds to an average return period of 2,475 years. We obtained the seismic design parameter from the United States Geologic Survey 2008 National Seismic Hazard Maps for Latitude 47.6079 and Longitude –122.2150. The parameters for seismic design in accordance with this code are:

- Maximum Considered Earthquake Spectral Response Acceleration at Short Periods, S_S = 1.328 g; and
- Maximum Considered Earthquake Spectral Response Acceleration at 1-Second Period, S_1 = 0.511 g.

The above parameters are for a Site Class B (soft rock) and need to be adjusted for the site-specific soil profile in accordance with the 2012 IBC. Based on our explorations, the site soil profile class is Site Class D.

6.2.2 Landslides

According to the City of Bellevue Geotechnical Report and Stability Analysis Requirements pseudo-static acceleration factor must be based on a peak ground acceleration (PGA) with a 10 percent probability of exceedance in 50 years, which corresponds to an average return period of 475 years.

The corresponding mapped PGA for the site is 0.283 g, which assumes a Site Class B soil profile. For a Site Class D profile, the site coefficient F_{PGA} is 1.234, which results in a design PGA of 0.349 g. Note this design PGA for landslide evaluation is approximately the same value as determined from the 2012 IBC design response spectrum.

6.3 Seismically Induced Geotechnical Hazards

Potential seismically induced geotechnical hazards include surface rupture, liquefaction and subsidence, lateral spreading, and landslides. Our review of these hazards is based on our soil explorations, regional experience, and our knowledge of local seismicity.

6.3.1 Surface Rupture

The northernmost splay of the Seattle Fault is about 1.3 miles south of the site. There is a remote potential for surface rupture at the site from a new splay of the Seattle Fault; however, this hazard is very low based on the Seattle Fault's 3,000-year recurrence interval, the large number of possible locations for surface rupture, and the chance that the fault would not produce surface rupture in this segment of the fault.

6.3.2 Liquefaction and Subsidence

Our borings and past explorations at and near this site encountered wet zones but did not encounter groundwater. Based on the elevation of Lake Washington, regional groundwater is anticipated to be within the Hard Clay or within the lower portions of the Hard Sandy Silt and Dense Silty Sand. Because of the fines content and density of the native soils, the risk of liquefaction, seismically induced settlement, or significant ground deformation caused by liquefaction from the design earthquake is low.

6.3.3 Lateral Spreading

Lateral spreading is typically associated with lateral movement on sloping ground caused by liquefaction or a reduction of shear strength of soils within or under the slope. Lateral spreading can affect a structure by increasing the lateral force exerted on the subsurface walls or within the pile foundations. Given the low liquefaction hazard, the risk of lateral spreading is considered low.

6.3.4 Landslides

Based on the site location and topography, the landslide hazard is considered moderate, and seismic landslide hazards are further evaluated in the following section. There appear to be no reported or observed signs of slope instability due to historical earthquakes.

7.0 GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

This section of the report presents our conclusions and recommendations for the geotechnical aspects of design and construction on the project site. We have developed our recommendations based on our current understanding of the project and the subsurface conditions revealed by our explorations and explorations performed by others nearby. If the nature or location of the facilities is different than

we have assumed, we should be notified so we can review, change, and/or confirm our recommendations.

7.1 General Considerations

The proposed development includes downward expansion of the existing structure. There is limited access below the existing structure due to the slope of the ground and the existing wood post foundations. The existing structure will remain and will be supported by new foundations and columns, replacing the existing wood post foundations below the footprint of the proposed addition.

Most of the site is in a steep slope critical area. Permanent tiebacks or soil nails are the most efficient option for mitigating any potential landslide hazard at the site. With the slope stabilized, a combination of shallow foundations and deep foundations may be used to support vertical building loads, depending on soil conditions encountered. Deep foundations may also be used for slope stabilization, provided they are embedded sufficiently below potential critical failure surfaces and have sufficient lateral capacity.

7.2 Slope Stability

To evaluate the stability of the slope, we performed limit equilibrium stability analysis using the computer program SLOPE/W Version 8.11 (Geo-Slope International 2013). The Morgenstern-Price method for slope stability analysis was used to search for rotational circular surface failure mechanisms.

Stability analyses were performed for existing and proposed conditions based on our field observations, laboratory testing, and estimated soil properties to evaluate slope stability factors of safety. Slope stability factors of safety represent the magnitude of forces acting to stabilize the slope (e.g., soil strength) divided by magnitude of forces acting to destabilize the slope (e.g., soil weight).

According to the City of Bellevue Geotechnical Report and Stability Analysis Requirements, the minimum required factors of safety for static and seismic analysis of permanent slopes are 1.5 and 1.15, respectively.

Seismic stability evaluations use a pseudostatic horizontal acceleration coefficient of one-half the design peak ground acceleration (PGA). As discussed in Section 6.2.2, the design PGA is 0.349 g. Based on this PGA, a pseudostatic horizontal acceleration coefficient of 0.175 is used in the seismic slope stability analysis.

Subsurface properties are based on information from the borings performed by Hart Crowser and provided in Appendix A. The soil parameters used in our analysis are shown on the slope stability figures as part of this report.

The slope stability analyses focused on deeper, global stability (i.e., deeper than 10 feet) for the site rather than potential shallow surficial instability, which may result in minor sloughing/erosion and generally can be managed through drainage considerations and maintaining vegetation on slopes.

Our field explorations and slope reconnaissance were performed in mid-April at the end of the typical wet season in the Puget Sound area. January through April 2014 rainfall data are summarized in Table 1 for the NWS Seattle Weather Forecast Office location, which is about 6 miles north-northeast of the site across Lake Washington. This data shows the Seattle-Bellevue area received above-average rainfall in February through April, and a record-setting amount of rain fell in March. Based on the amount of rainfall received during the time leading up to our fieldwork, we interpret the observed conditions during our slope reconnaissance and drilling as representative of wet season conditions.

Table 1 – January Through April 2014 Rainfall Data – NWS Seattle Weather Forecast Office

Month	Total Rainfall in Inches	+/- Relative to Average Rainfall in Inches
January	4.02	-0.79
February	5.13	+1.82
March	8.42	+4.91
April	3.45	+0.68

7.2.1 Existing conditions

Static and seismic stability were checked as a baseline for the existing slope based on observed conditions. Static and seismic slope stability for existing conditions are presented on Figures 4 and 5, respectively. Note the existing post foundations embedded in the slope were conservatively ignored in these analyses.

Slope stability factors of safety for existing conditions are summarized in Table 2. For the both the static and seismic case, the factors of safety indicate the slope is stable under static conditions and marginally unstable under seismic conditions. For both cases, the factors of safety are below the current code minimums.

Table 2 – Existing Conditions Factors of Safety

Static Conditions		Seismic Conditions	
Analysis	Code Minimum	Analysis	Code Minimum
1.25	1.50	0.96	1.15

7.2.2 Proposed Development

Static and seismic stability were evaluated for the proposed development based on observed conditions. Static and seismic slope stability for proposed conditions are presented on Figures 6 and 7, respectively. The building load was provided by the structural engineer and modeled as a 250 psf surcharge on the slope as shown on Figures 6 and 7. We conservatively assumed all the building load will be applied to the slope in our analysis, ignoring beneficial effects of any deep foundations. In order to increase factors of safety to meet the minimum code requirements for both the static and seismic conditions, three horizontal point loads were modeled as shown on Figures 6 and 7, to model the horizontal reaction from proposed tiebacks anchored to the basement walls. The required

horizontal load per row of tiebacks is 5.0 kips per foot, or a total horizontal load of 15 kips per foot, which is governed by seismic stability. Slope stability factors of safety for existing conditions are summarized in Table 3.

Table 3 – Proposed Conditions Factors of Safety*

Static Conditions		Seismic Conditions	
Analysis	Code Minimum	Analysis	Code Minimum
1.67	1.50	1.16	1.15

*Includes 5 kips per foot horizontal load per row of tiebacks from the three rows shown on Figures 6 and 7.

The horizontal load per row of tiebacks required to achieve the code minimum factors of safety appears reasonable for tiebacks at typical spacing and reasonable bond lengths; however, this will need to be confirmed by the structural engineer.

The global slope stability analyses assume the basement wall is stable. The structural design of the wall should use the lateral earth pressures recommended later in this section.

7.3 Site Preparation and Grading

Site preparation for the footprint addition and structure renovations may involve demolishing some of the existing wood post foundations, removing driveway pavement, removing existing retaining walls, and removing other obstructions that may interfere with new construction. We recommend cutting the wood post foundations near grade and leaving them in place after the structure loads have been transferred to the new foundations, where applicable. If the wood post foundations must be removed from the ground, we recommend backfilling the void with controlled density fill.

We do not recommend disturbing existing vegetation on the steep slope portion of the site; if vegetation must be removed, remove as little as possible. If trees need to be removed to accommodate construction, we recommend minimizing disturbance to the root system. For example, trees may be cut and stumps ground, but the trees or stumps should not be pulled from the ground, which may disturb the near-surface soils.

Removal of visible organic material (sod, humus, roots, and/or other plant material), debris, and other unsuitable material is recommended from subgrade areas where asphalt or concrete will be placed. We recommend all site grading, paving, and any utility trenching be conducted during relatively dry weather.

We recommend limiting site grading to only that necessary to construct the proposed addition. We recommend the natural topography surrounding the site be maintained to the extent feasible.

It may be necessary to relocate or abandon some utilities. These utility lines will likely be excavated in fill materials. Abandoned underground utilities should be removed or completely grouted. The ends of remaining abandoned utility lines should be sealed to prevent soil or water from entering the pipe.

Soft or loose backfill should be removed, and excavations should be backfilled with structural fill. Coordination with the utility owners is generally required to address existing utilities.

7.4 Excavation Shoring and Support of Existing Structures

The proposed addition requires a cuts of about 6 feet deep into the slope to accommodate the lowest proposed level of the addition, which has a finish floor elevation of about 47.33 feet. This cut will require properly designed temporary or permanent shoring to provide lateral support for safety and stability of the adjacent buildings and utilities and allow for construction of subgrade walls and installation of foundations for the proposed addition.

7.4.1 General Considerations

It is our opinion that the limited project excavation could be supported using a combination of slope cuts, cantilevered soldier piles, and braces, tiebacks, or soil nails. Due to limited overhead access at below the proposed west-facing basement walls, installation of soldier piles does not appear feasible, and some combination of shotcrete or cast-in-place walls may be needed for temporary support or implemented as part of the permanent basement wall. For the north and south basement walls, access may allow for installing soldier piles and lagging for temporary shoring.

The temporary shoring should be designed by a professional structural engineer registered in the State of Washington. We also recommend that we be given the opportunity to review the proposed shoring design before construction.

This report is not meant to provide specific criteria for the contractor's construction means and methods. It should be the responsibility of the shoring contractor to verify actual ground conditions at the site and determine the construction methods and procedures needed for the installation of an appropriate shoring system.

7.4.2 Cantilevered and Braced or Tieback Excavation Support

The geotechnical criteria for the design of an excavation supported by soldier piles and braces or tieback include lateral soil pressures and frictional resistance for tiebacks. Figures 8, 9, and 10 illustrate and outline these recommended parameters for cantilevered shoring and braced or tieback shoring, respectively.

Lateral Pressures

Lateral earth pressures for the shoring design depend on the type of shoring and its ability to deform. If the top of the shoring is allowed to deform on the order of 0.001 to 0.002 times the shoring height, and if no settlement-sensitive structures or utilities are within the zone of deformation, the shoring may be designed using active earth pressures. If settlement-sensitive structures or utilities exist within the potential zone of deformation, or where the shoring system is too stiff to allow sufficient lateral movement to develop an active condition, at-rest earth pressures should be used to design the shoring.

Temporary and Permanent Shoring. Cuts may be supported by a cantilevered soldier pile shoring wall or temporary shoring with single or multiple levels of braces or tiebacks at the site, depending on access limitations and lateral soil loads. For cantilevered soldier pile wall design, lateral earth pressures are approximated using a triangular pressure distribution using equivalent fluid unit weights based on the soil properties and ground slopes. For braced or tieback wall design, lateral earth pressures can be approximated by a composite, trapezoidal, lateral earth pressure distribution. The distributions presented on Figures 8 and 9 are appropriate for cantilevered or single and multilevel brace/tieback supports, respectively. The pressure distribution changes, depending on the slope behind and perpendicular to the wall, and Figures 8 and 9 present varying lateral earth pressures depending on the slopes. For walls with no slope perpendicular to the shoring wall (i.e., at the north and south basement walls and the walls below proposed floor slabs) we recommend using the lateral earth pressures for $\beta = 0°$. For the upper-most, west-facing basement wall we recommend using the lateral earth pressures for $\beta = 34°$.

Important. The lateral earth pressures presented herein are based on level ground or sloping conditions behind the walls as identified and dewatered conditions such that hydrostatic pressure does not act on the walls above the base of the excavation. For design calculations, we recommend that at least 2 feet be added to the proposed excavation depth to allow for possible surface pressures near the excavation such as light vehicles and small material stockpiles. Surcharge pressures resulting from heavier loads such as buildings, footings, heavy equipment, or large material stockpiles, should be calculated using Figure 11. These additional loads would be added to the calculated loads for the shoring walls.

Wall Design

Walls must be designed to carry the bending stresses due to soil and/or tieback loads and the vertical load resulting from down-angle tieback anchors and underpinned footings. The bending stresses can be calculated using the earth pressure diagrams presented on Figures 8 and 9.

For walls with soldier piles, they must be embedded deeply enough to resist these vertical loads and to provide kickout resistance for the portion of the wall below the lowest support. General soldier pile design information is presented on Figures 8 and 9, as discussed above. For soldier pile design, we recommend:

- Use Figures 8 and 9 for allowable pile end-bearing and skin friction design values.
- Embed soldier piles at least 10 feet below the bottom of the excavation to provide firm support for the soldier piles and avoid excessive settlement when tieback loads are applied.
- Design soldier piles for bending using a uniform loading equivalent to 80 percent of the design values and analyze for shear using total load.
- For design against kickout, compute the lateral resistance based on the passive pressures presented on Figures 8 and 9, acting over twice the diameter of the concreted soldier pile section or the pile spacing, whichever is less.

These recommendations are based on proper installation of the soldier piles as discussed below.

Soldier Pile Installation

Conditions such as caving soil and groundwater can loosen soil at the bottom of the soldier pile borehole and reduce the bearing capacity of the zone of disturbed soil. Tieback destressing and shoring failure could occur if bearing capacity is inadequate and soldier piles settle under the vertical component of the inclined tieback load. We recommend that a Hart Crowser representative closely monitor soldier pile installation for these conditions so that construction methods can be adjusted accordingly.

For soldier pile installation, we recommend:

- The contractor should be prepared to case the soldier piles. The need for casing should be determined in the field at the time of installation.
- The contractor should be prepared to tremie concrete from the bottom of the hole to displace groundwater or drilling mud used to maintain an open hole.
- The contractor should be prepared to excavate the soldier piles in a manner that prevents "heave" or "boiling" of the bottom of the soldier pile excavation. It may be possible to over-drill the borehole and backfill the bottom of the borehole with structural concrete bearing on undisturbed soil.
- Drilling mud should not be used unless reviewed and approved by the geotechnical and structural engineer.

Soldier pile shoring construction may be difficult if cobbles or loose sand and gravel are encountered in the excavation. If these conditions are encountered, substantial raveling of the soil could occur. The contractor should be prepared to place lagging in short vertical increments and should be prepared to backfill voids caused by ground loss behind the shoring system.

Lagging

Ground loss between soldier piles is prevented using lagging. The most common form of lagging is timber planks. The lagging is attached to the soldier pile and, because of soil arching and the ability of the lagging to deflect, is designed for some fraction of the applied pressure on the wall.

Timber Lagging Design Recommendations. Design the lagging based on the FHWA (1999) recommendations. Timber lagging design will depend on the final spacing of the soldier piles and the depth of the excavation. We provide our recommendations assuming a typical soldier pile spacing of 8 feet on center or less and excavation depths up to about 15 feet.

The Hard Sandy Silt and Dense Silty Sand unit and Hard Clay unit, as previously defined, are classified as "competent soil" per the FHWA (1999) design table, and the Fill and Medium Stiff to Stiff Sandy Silt is classified as "difficult soil." Table 4 summarizes our lagging thickness recommendations.

Table 4 – Minimum Lagging Thickness Recommendations*

Excavation Wall	Minimum Lagging Thickness in Inches
Upper-Most, West-Facing	4
All Others	3

*Assuming typical soldier pile spacing of 8 feet on center or less and excavation depths up to about 15 feet.

Once the shoring designer has laid out the final shoring design and lagging requirements, we should review this layout to suggest appropriate modifications, if necessary.

Construction Recommendations. Prompt and careful installation of lagging, particularly in areas of seepage and loose soil (especially organic soil or peat), is important to maintain the integrity of the excavation. Proper lagging installation should be the responsibility of the shoring contractor to prevent soil failure, sloughing, and ground loss, and to provide safe working conditions.

Backfill voids greater than 1 inch using sand, pea gravel, or a porous slurry. Backfill the void spaces progressively as the excavation deepens. The backfill must not allow hydrostatic pressure buildup behind the wall. Drainage behind the wall must be maintained or hydrostatic water pressure should be added to the recommended lateral earth pressures.

Tieback Anchor Design

We anticipate that tieback anchors may be used for external lateral support of the north, west, and south shoring walls. For tieback anchor design, we recommend:

- Install the bond zone of tieback anchors into the Hard Sandy Silt and Dense Silty Sand or Hard Clay. This may require inclining tiebacks at a steeper angle than recommended on Figure 9.
- Locate the anchored portions of the tiebacks behind the no-load zone as shown on Figures 9 and 10.
- Use allowable friction design values as presented on Figure 9.
- Locate anchors at least three tieback diameters apart.
- Pump structural grout into the anchor zone either using a grout hose or tremie hose placed to the bottom of the anchor.
- Grout and backfill drilled installations immediately after drilling, and do not leave holes open overnight. This will help prevent collapse of the holes, loss of ground, and surface subsidence.
- Take care not to “mine out” large cavities in granular soils if drilling with a continuous-flight auger.
- Maintain continuous cutting return if using a pneumatic drill so that air pressure does not damage nearby utility vaults, corridors, or subgrade slabs.

- Plot and compare anchor lengths with any underground support elements of adjacent structures.
- Design temporary and permanent tiebacks to resist corrosion for the service life of the anchor.

The shoring contractor should note any existing facilities adjacent to the project site including buried utilities and foundations, as these may affect the location and length of the anchor holes. The anchor holes should be installed in a manner that will minimize ground loss and will not disturb previously installed anchors. During tieback drilling, wet or saturated zones may be encountered and caving or "blow-in" could occur. Drilling with a casing would reduce the potential for these conditions and ground loss.

We recommend that selection of the materials and the installation technique be left to the shoring contractor. The selected tieback anchor installation method must be subject to field verifications with performance and proof testing as discussed in Attachment 1.

Hart Crowser should review the design for anchor locations, capacities, and related criteria prior to implementation.

The tentative anchor pullout resistances as presented on Figure 9, include a factor of safety of at least 2.0. This factor of safety provides for a reasonable additional load capacity should anchor installation cause an unforeseen increase in unit soil load. Some field changes in anchor length may be necessary due to variability in soil conditions encountered in the field.

Tieback Anchor Testing

The tiebacks will be tested to confirm the anchor design values and to verify that a suitable installation is achieved. The procedure for performance and proof testing is presented in Attachment 1 and summarized below.

For testing of tieback anchors, we recommend:

- Require the shoring contractor to complete successful 200 percent performance tests on a minimum of four tiebacks—specifically, two tiebacks for each different soil type and construction method. Hart Crowser should review and approve the final number of verification tests and their locations once the shoring plans are finalized. Contract documents should be prepared so that additional verification tests could be performed on a unit price basis should differing site conditions be encountered.
- For anchors installed for the 200 percent verification test, the specifications should include components to prevent friction between the grout column and the soil in the no-load zone.
- In addition to performance testing, each production anchor should be proof tested up to 133 percent of the design load to verify total movement and creep requirements.
- Following proof loading, we recommend locking off each tieback anchor to 80 to 100 percent of the design load.

Deflections

Based on the assumed loading conditions and the applied loads, we expect the cantilevered and braced/tieback shoring systems to deflect an average of about 1 inch or less into the excavation for designs based on active conditions. Some areas may experience more deflection, and some areas may deflect away from the excavation during tieback stressing. Such deflection can be caused by construction practices.

The geotechnical and structural engineer should review any deflections more than 0.5 inch in an attempt to identify the cause of the deflection. Remedial actions would be recommended, if necessary.

7.4.3 Soil Nail Excavation Support

It is our opinion that the excavations could also be supported using a soil nail shoring wall system. A soil nail shoring system typically may be more expedient and more economical than using tieback anchors.

We also recommend Hart Crowser discuss the conditions with the potential soil nail subcontractor before proceeding with the shoring wall design.

Soil nailing is a method of slope stabilization whereby excavated slopes are strengthened by the insertion and grouting in place of small-diameter steel rods. Soil nailing typically requires completing the excavation in about 4-foot vertical lifts, and installing the nails in a grid pattern as the excavation proceeds downward. The exposed soil would be covered by about 4 to 8 inches of shotcrete that is reinforced with a lightweight steel mesh for temporary shoring. An alternative to a temporary shotcrete wall is to construct a permanent wall, top-down, as the excavation is completed. The process of excavating one lift at a time and installing a row of nails is repeated until the entire excavation is completed.

Soil nails do not retain soil in the same manner that tiebacks retain a shoring wall. Soil nails are grouted through their full length and, in most cases, only a small portion of the anchor load is transferred to the wall face. Tiebacks, on the other hand, are not grouted through their full length, and are designed to transfer loads to the wall face.

Soil Nail Design

The actual bond between the grout and the soil is an important design parameter, and can vary with the method of installation. The soil nail system should be designed to performance specifications, and the designer should be able to demonstrate that:

- No failure surface exists through or outside the nails with a factor of safety less than 1.35 against sliding.
- The nails are not stressed in excess of 80 percent of their yield stress.

- The mobilized bond stress is less than half the ultimate adhesion between the grout and the soil. Ultimate adhesion is determined by the soil shear strength and must be justified by both pullout testing before nail installation and by limited production nail testing.

- Design methods are in accordance with FHWA "Manual for Design & Construction Monitoring of Soil Nail Walls" (1996).

For cost estimating purposes, typical nail spacing for this type of project can be assumed to be about 5 feet center-to-center in a grid pattern, and an average nail length is equal to about 0.8 times the height of the excavation.

Soil nail design should consider slopes above the excavation, surface loading from traffic and site equipment, and loads from adjacent structures.

Soil Nail Wall Drainage

Weep hole toe drains are typically installed at the base of the shotcrete face to ensure that the wall is drained. Temporary wall drainage for these site conditions typically consists of vertical geocomposite drain strips, about 16 inches wide, that extend the full wall height at the soil-shotcrete interface. This will help ensure drainage of potential perched water from the face of the excavation or surface water from the slope. Water from the vertical geocomposite drains and rainfall must be directed away from the toe of the wall. The permanent drainage system can be accommodated between the temporary shotcrete face and the permanent wall.

Soil Nail Wall Construction

Construction sequencing is especially important in soil nail construction. Soil nail wall systems are designed so that the excavation must proceed in staged lifts (a lift is a single row of nails). We make the followings recommendations concerning vertical cuts:

- Conduct testing for each material type to demonstrate the unsupported face will be stable over the required "stand-up" time;

- Ensure that all surface water is controlled during construction;

- Excavate the initial cut a few feet below the first row of nails; and

- Limit excavation height to the minimum amount necessary for practical and timely application of shotcrete, typically no more than an unsupported height of about 4 feet. In caving ground, provide an initial stabilizing layer of shotcrete (flashcoat) and/or steel-reinforced flashcoat as soon as possible; in firm ground the nails may be installed first.

We make the following recommendations concerning installation:

- Allow no excavation section to be open more than 24 hours.

- Drill holes may be accomplished by rotary methods with air flush, dry auger, and cased methods (for less stable ground). Drill the soil nail holes using equipment and techniques that will minimize caving and loss of ground. Ensure that the hole is clean of material.
- No holes should remain open overnight.
- Pump structural grout into the hole through the auger (wet bar installation method) or through a tremie tube extended to the bottom of the hole.
- Grout the hole as soon as possible after drilling to prevent caving.
- Nails should be reinforced steel bars without couplers, splices, or welds, and should be installed with centralizers.
- Soil nail lengths will depend on soil conditions (which will need to be verified by a testing program).
- The shoring contractor should particularly note the presence of existing facilities adjacent to the project site, including buried utilities and foundations, as these may affect the location or extent of the anchor holes.

It is the responsibility of the contractor to verify actual ground conditions at the site and to determine construction methods and procedures for installing a suitable shoring system. Cobbles, boulders, or debris may be encountered and could impact construction.

Reinforced Shotcrete Wall Construction

A structural engineer should design reinforcement and shotcrete for permanent, below-grade walls. For shotcrete wall construction, we make the following recommendations:

- Prior to production, shotcrete application test panels should be applied by each nozzleman under field conditions at the site, and the results cored and examined for defects;
- Preparations for shotcrete typically includes installation of drainage material, installation of soil nails, and placement of approved reinforcement; and
- If sloughing occurs, shorten the time a cut is left open, reduce the height of the cut, use a stabilizing berm, place a flashcoat of shotcrete, or place or complete the cut in sections or stages.

Soil Nail Testing

Soil nails should be tested to confirm the design friction value and to verify that suitable installation has been achieved. Nails that are performance tested should be installed with a bond breaker sleeve or casing within the no-load zone of the nail. The casing or sleeve should be removed and the nail hole fully grouted after testing. A reaction plate or system must be provided by the contractor, and is subject to the designer's approval.

A minimum of two successful 200 percent verification tests should be completed to confirm adhesion values used in design. Verification tests should be completed before installation of production nails in that particular soil type. We recommend that Hart Crowser select the test locations based on observation of the soil conditions as the excavation is accomplished. A minimum of 5 percent of the soil nails should be proof loaded to test the total movement and creep of the nail.

Shoring Deflection

In theory, a soil nail system should deflect more than a pre-stressed shoring system since the nails are not pre-stressed. However, observations of soil nail wall deflections in the Puget Sound area indicate that, if constructed in favorable soil conditions, deflection of the two systems are similar.

7.5 Foundations

We recommend using a combination of shallow and deep foundations for support of the proposed addition and existing structure, depending on the soil conditions encountered as described in the following subsections.

We assume the slope will be stabilized to meet the code minimum factors of safety using permanent tiebacks or soil nails, and deep foundation support will be used only to support structure loads.

Any proposed foundation options should consider the methods selected for excavation support to minimize mobilization of equipment and potential slope disturbance. The proposed foundation option and installation method need to consider site access as well as soil conditions. Other foundation options may be considered and evaluated as design progresses and after a contractor is selected, depending on site access and availability of construction material and equipment.

In order to accommodate the excavation for the proposed addition, the new foundations may need to be installed to transfer the structure load before significantly advancing the excavation below or around the existing wood post foundations.

7.5.1 Deep Foundations

We recommend pin piles or micropiles for foundation support where the Fill and Medium Stiff to Stiff Sandy Silt is encountered below the base of the structure.

7.5.1.1 Pin Piles

We recommend driving a minimum 2-inch-diameter (2.375-inch OD, Schedule 80) steel pipe to refusal using a pneumatic or hydraulic hammer for this project. We recommend an allowable vertical compressive load capacity of 6 kips (3 tons) for 2-inch-diameter pin piles. We recommend an allowable vertical compressive load capacity of 12 kips (6 tons) for 3-inch diameter pin piles. These allowable capacities include a factor of safety of 2.

The refusal criteria are generally defined as the time required to achieve 1 inch of penetration for a range of hammer weights and a given pile diameter, as shown in Table 5.

Table 5 - Refusal Criteria for 2-Inch-Diameter Pin Pile Installation

Hammer			Seconds Per Inch	
Type	Model	Weight in Pounds	2-inch Pipe	3-inch Pipe
Pneumatic	Jack Hammer	90	60	N/A
Hydraulic	TB-100	135	30	N/A
Hydraulic	TB-125	400	8	N/A
Hydraulic	TB-225	650	N/A	12
Hydraulic	TB-325	850	N/A	10
Hydraulic	TB-425	1100	N/A	6

We recommend that pin piles be galvanized for corrosion protection.

We recommend that a representative from Hart Crowser be on site during pile installation to monitor penetration rates and confirm refusal.

7.5.1.2 Micropiles

Micropiles consist of small-diameter (usually 6 to 12 inches) drilled and grouted replacement (non-displacement) piles that are typically reinforced. A micropile is installed by drilling a borehole, placing reinforcement, and grouting from the bottom up. A drilling method suited to local conditions may be selected from a number of options. For example, rotary percussive or rotary duplex techniques may be used to penetrate obstructions.

Micropiles can withstand relatively significant axial loads and moderate lateral loads, and may be considered a substitute for conventional driven piles or drilled shafts. An advantage of micropiles is that they can be installed where access is restricted and in most soil types and ground conditions.

Because of its small diameter, the end-bearing resistance of a micropile is minor compared to the grout-to-ground bond resistance along its shaft, and is typically neglected. The soil conditions and installation procedure strongly influence the grout-to-ground bond strength. In general, micropiles are classified into four types (A to D) depending on the construction details (FHWA 2005). For this analysis, we have assumed that the micropiles will be constructed using pressure grouting through the casing as the casing is withdrawn slowly and incrementally, which corresponds to the Type B classification (FHWA 2005).

Vertical Capacity of Micropiles. We recommend an allowable adhesion of 2 kips per square foot for preliminary design of micropile length for vertical capacity. This allowable adhesion includes a factor of safety of 2.

We make the following additional recommendations:

- Contract the micropiles as design-build to allow the contractor to optimize the installation method. Prospective contractors may be required to provide information about their proposed methods of drilling and grouting to evaluate their applicability and suitability to site conditions.
- Require two verification tests (to 200 percent of the design load) on a non-production, "sacrificial" micropile following standard procedures and criteria (FHWA 2005) before construction of the production piles. These tests are intended to ensure that the design capacities can be achieved with the soil and construction equipment used by the contractor at the site. A minimum of two tests are recommended to test the variability of soil conditions at the pile rows.
- Require proof tests (to 160 percent of the design load) on 5 percent of production piles and a minimum of one pile per row following standard procedures and criteria (FHWA 2005).
- If the foundation layout requires micropile spacing closer than six pile diameters, adjust capacity for group effects (FHWA 2005).

Micropile Installation. It is important to select an experienced micropile contractor. The completed pile is below the ground surface and cannot be observed during construction, so judgment and experience must be used to determine the acceptability of the pile. We recommend close monitoring of installation procedures such as installation sequence, casing withdrawal rate, grouting pressure, and quantity of grout used per pile. Variations from the established pattern, such as low grout pressure, excessive settlement of grout in a completed pile, etc., would make the pile susceptible to rejection.

We make the following recommendations for micropile installation:

- Micropile installation should be observed by Hart Crowser to evaluate the contractor's operation and to collect and interpret the installation data.
- Disposal of excess soil that will be generated during micropile installation should be considered if environmental issues exist at the site.
- Cobbles and large boulders should be anticipated during drilling, and appropriate drilling methods should be chosen accordingly.
- Two micropiles should not be installed within five pile diameters of each other (center-to-center), in a single 12-hour period to prevent interconnection of grout between piles.
- Micropiles in soft silt or any loose sand materials should be cased to avoid excessive grout take.
- The casing should be withdrawn from the hole at a slow rate so that pressure on the grout column is maintained.

- Drill fluids and cuttings should be contained and not allowed to flow down the surface of the slope.

Foundation Settlement. We estimate that total post-construction settlement of pin pile- or micropile-supported structural elements will not exceed 1 inch. Differential settlement between adjacent piles could approach one-half of the actual total settlement. We recommend Hart Crowser review the final foundation plan to confirm the estimated settlement after final structural loads and layouts have been established.

7.5.2 Shallow Foundations

Shallow foundations may be used to support structure loads where the Hard Sandy Silt and Dense Silty Sand unit is present at the bottom of foundation elevation.

We make the following recommendations for design of shallow footings:

- A maximum net allowable bearing pressure of 6 kips per square foot (ksf) is recommended for design of footings bearing on native Hard Sandy Silt and Dense Silty Sand.
- Isolated and strip footings should have a minimum width of 24 and 18 inches, respectively. Place the base of all footings at least 18 inches below the lowest adjacent grade for consideration of frost protection.
- Allowable bearing pressure may be increased by up to one-third for loads of short duration (e.g., wind or seismic loads).
- Use an allowable coefficient of friction against sliding equal to 0.3 for footings poured neat against the native bearing soil. This value includes a factor of safety of at least 1.5.
- Use an allowable passive equivalent fluid unit weight of 300 pcf. This value includes a factor of safety of at least 1.5.
- Compact all exposed subgrades to a firm, non-yielding condition.
- It may be necessary to locally overexcavate beneath individual footings to expose the native bearing soil. Localized overexcavation may be needed if loose or soft material is encountered below the base of the footing. If overexcavation beneath footings is necessary, backfill the overexcavated zone with structural fill or lean concrete. Overexcavation should extend outward and downward from the outer edges of the footing to the top of the native material no steeper than 1H:2V.
- Depth of footings should also ensure that they are founded outside of an imaginary 1H:1V plane projected upward from the nearest bottom edge of adjacent footings or utility trenches.
- A qualified geotechnical representative should be on site to assess and document the suitability of the subgrade during construction, prior to placement of footings or concrete.

Settlement Estimates

For foundations designed and constructed as described above, we estimate that the total static settlement of individual footings will be less than approximately 1 inch. We estimate that differential settlement between adjacent footings will be about one-half of the total settlement. Considering the nature of the Hard Sandy Silt and Dense Silty Sand encountered in our explorations, we anticipate that the static settlement will essentially occur as loads are applied.

The foundation settlement estimated herein assumes that careful preparation and protection of the exposed subgrade will occur before concrete placement. Any loosening of the materials during construction or the presence of loose material beneath footings could result in larger settlement than those estimated herein. It is very important that all foundation excavations be cleaned of loose or disturbed soil prior to placing any concrete and that there be no standing water in any foundation excavation.

7.6 Floor Slabs

We anticipate that the floor slabs will be constructed as a slab-on-grade over a capillary break layer. We recommend overexcavating up to 8 inches of soft, loose, or wet material, if encountered below slab-on-grade floor slabs, and replacing with compacted structural fill.

Alternatively, a structural slab tied to the pile caps may be considered where deep foundation are recommended, in order to limit cracking of the slab or potential differential settlement. A structural slab would also prevent the need to remove, re-compact, or otherwise improve subgrade conditions where loose or soft soils may be present in the Fill and Medium Stiff to Stiff Sandy Silt unit.

In addition to any structural fill, the floor slab should be underlain directly by a capillary break/drainage layer at least 4 inches thick. This layer should consist of clean, well-graded coarse sand and gravel with a fines content (soil finer than the No. 200 sieve based on the minus 3/4-inch fraction of the material) of less than 3 percent by weight. This layer serves as a capillary break and drainage layer and is intended to reduce the potential buildup of hydrostatic pressure beneath the slab and to provide permanent control of groundwater beneath the floor slab and behind the perimeter walls, (see Section 7.7). For slab-on-grade floors, we recommend:

- Compact the drainage layer to the criteria of structural fill described in Section 7.8.
- A modulus of subgrade reaction of 75 pci is appropriate for design of floor slabs on the compacted/well-prepared subgrade in the Fill and Medium Stiff to Stiff Sandy Silt unit or on structural fill. A modulus of subgrade reaction of 200 pci is appropriate for design of floor slabs on the Hard Sandy Silt and Dense Silty Sand. This assumes that the construction is accomplished as described above and that the capillary break layer is underlain by a well-prepared subgrade.
- Determine sliding friction between the slab and subgrade using an allowable coefficient of friction of 0.30.

- Submit any soil that is to be considered as capillary break or drainage material to Hart Crowser for gradational analysis.

- Note that if the bottom of the excavation is soft, wet, and disturbed, the contractor should be prepared to place a temporary working surface. This surface cannot count as part of the 4-inch drainage layer.

7.7 Lateral Earth Pressures on Permanent Subgrade Walls

Permanent walls constructed against temporary shoring should be designed for the same active (or at-rest) pressures used to design the shoring system.

For basement or retaining walls backfilled on one side only, the structural engineer can estimate the lateral load and resistance on the walls using an equivalent fluid to represent the soil. We make the following recommendations for walls with backfill material placed as structural fill.

- For a yielding wall (active horizontal soil pressure coefficient) with level backfill, the equivalent fluid density of the soil is 34 pcf. We define a yielding wall as one where the top moves away from the retained soil, when loaded, at least 0.1 percent of its height.

- For a non-yielding wall (at-rest horizontal soil pressure coefficient) with level backfill, the equivalent fluid weight is 53 pcf.

- Place 18 inches of free-draining, well-graded sand and gravel (less than 3 percent fines based on the minus 3/4-inch fraction) against backfilled walls to prevent hydrostatic pressure buildup.

Resistance to sliding for backfilled walls may be provided by friction under the wall and passive pressure in front of the embedded part of the wall or from the lateral resistance of piles and passive pressures on pile caps. For level ground we recommend an allowable passive pressure of 300 pcf in the Hard Sandy Silt and Dense Silty Sand and 260 pcf in the Fill and Medium Stiff to Stiff Sandy Silt. LPILE parameters can be provided or we can perform lateral analysis of pile supported walls, as design progresses.

Note that the equivalent fluid density does not include the effects of any surface loads or hydrostatic loads.

7.7.1 Lateral Earth Pressures Due to Seismic Pressures

For seismic loading conditions, the basement walls between floor slabs can be designed for 80 percent of the load and resistance factor design (LRFD) seismic load combination based on the relative stiffness of the floor slabs. Lateral earth pressures based on the 2012 IBC design earthquake for active and at-rest conditions can be assumed as uniform pressures in pounds per square foot as shown in Table 6.

Table 6 – Seismic Lateral Earth Pressures

Basement Wall	Active in psf	At-Rest in psf
Upper-Most, West-Facing	17H*	34H
All Others	7H	14H

*H is the height of the wall in feet.

The design earthquake and 2012 IBC design parameters are discussed in Section 6.0.

7.8 Permanent Drainage

Groundwater was not encountered in our borings, although wet zones were observed in one boring at (HC-1) about 35 to 45 feet bgs (elevation 52 to 42 feet). We have assumed regional groundwater is at elevation 18.5 feet based on the Lake Washington Ordinary High Water Level. Groundwater levels can fluctuate and potential perched groundwater may occur, posing a potential issue that can be readily handled by following the recommendations below.

7.8.1 Foundation and Perimeter Wall Drainage

We recommend the following for permanent drainage of the perimeter walls and slabs around the structure.

- Miradrain-type composite panels should be laid flush on the shoring wall (on the outside of the permanent wall) at a typical spacing of one panel between soldier piles or incorporated into the construction of permanent shoring or top-down construction options. The panels should be connected to a collector pipe that runs along the base of the below-grade walls, at an elevation lower than the bottom of the floor slab. This will allow water collected outside the wall to be tight-lined beneath the slab and into the central drainage sump.

- Perimeter drains should be installed near the base of the perimeter walls. The perimeter drains should be a minimum 2-inch-diameter perforated pipe and should be surrounded by 4 inches of drainage material. All drainage pipes should be sloped to drain.

- All slabs should be underlain directly by a capillary break/drainage layer at least 4 inches thick that is hydraulically connected to the perimeter footing or wall drains. This layer should consist of well-graded, free-draining sand and gravel with less than 3 percent fines. This layer is intended to reduce the potential buildup of hydrostatic pressure beneath the slab and to provide a hydraulic connection to the perimeter footing or wall drains.

- Wall and perimeter drainage pipes should be connected to a central underslab sump complete with an appropriate pump or collected by gravity drainage and tightlined to the base of the slope for discharge.

The groundwater inflow is expected to be minimal (less than 5 gpm).

7.8.2 Backfilled Walls

Walls with soil backfilled on only one side will require drainage or must be designed for full hydrostatic pressure. We recommend:

- Backfill immediately behind the wall with a minimum thickness of 18 inches of well-graded, free-draining sand or sand and gravel.
- Install drains behind any backfilled subgrade walls. The drains, with cleanouts, should consist of minimum 4-inch-diameter perforated pipe and placed on a bed of, and surrounded by, 6 inches of free-draining (less than 3 percent fines based on the minus 3/4-inch fraction), well-graded sand or sand and gravel. The drains should be sloped to carry the water to a sump or other suitable discharge, as discussed in Section 7.7.1.
- Drains can also consist of Miradrain-type composite panels laid flush on the outside of the permanent wall and connected to a collector pipe that runs along the base of the wall, at an elevation below the bottom of the floor slab. This will allow water collected outside the wall to be tight-lined beneath the slab and into the central drainage sump or discharge location.
- The backfill should be continuous and envelop the drainage pipe behind the wall.

7.8.3 Site Drainage and Infiltration

Final grades should be sloped to carry surface water runoff away from structures to prevent water from infiltrating near the foundation walls, where feasible; grading should be minimized in the steep slope area. Roof drainage and new pavement drainage should not be tied into the subdrain system or discharged onto the site; all drainage should be tight-lined to the base of the slope for discharge.

We do not recommend infiltrating water above or on the slope, and the soils below the base and away from the slope to not appear suitable for infiltration.

7.9 Structural Fill

Backfill placed within the building area or below paved areas should be considered structural fill. We provide the following recommendations:

- For imported soil to be used as structural fill, we recommend using a clean, well-graded sand or sand and gravel with less than 5 percent by weight passing the No. 200 mesh sieve based on the minus 3/4-inch fraction. Compaction of material containing more than about 5 percent fine material may be difficult if the material is wet or becomes wet during rainy weather.
- Place and compact structural fill in lifts with a loose thickness no greater than 10 inches. If small, hand-operated compaction equipment is used to compact structural fill, the lifts should not exceed 8 inches in loose thickness.

- Compact all structural fill to a minimum of 95 percent of the modified Proctor maximum dry density as determined by ASTM D 1557 test procedure.
- Control the moisture content of the fill to within 2 percent of the optimum moisture. Optimum moisture is the moisture content corresponding to the maximum modified Proctor dry density.
- In wet subgrade areas, clean material with a gravel content (material coarser than a US No. 4 sieve) of at least 30 to 35 percent may be necessary.

Before fill control can begin, the compaction characteristics must be determined from representative samples of the structural and drainage fill. Samples should be obtained and submitted to Hart Crowser for testing as soon as possible. A study of compaction characteristics should include determination of the maximum dry density, gradation, and optimum and natural moisture content of the fill at the time of placement.

7.9.1 Use of On-Site Soil as Structural Fill

The suitability of excavated site soil for compacted structural fill depends on the gradation and moisture content of the soil when it is placed. As the amount of fines (that portion passing the No. 200 sieve) increases, the soil becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve. Soil containing more than about 5 percent fines cannot be consistently compacted to a dense, non-yielding condition when the water content is greater than about 2 percent above or below optimum. Reusable soil must also be free of organic and other unsuitable material.

Our explorations indicate that most site soils to be excavated contain more than 5 percent fines, and many soils were greater than 50 percent fines. We do not recommend on-site soils be used as structural fill due to their composition and gradation. Nonetheless, these soils can be used for non-structural purposes such as in landscaped areas.

7.10 Utilities

Our understanding is existing utilities will generally be maintained, but some additional utilities will be needed to support the proposed addition (e.g., additional sewer lateral).

Proposed utilities may be supported on-grade, provided all unsuitable material has been removed directly below the trench, which may require utility trench subgrade improvements. We recommend the following:

- Provide at least 1 foot of dense granular soil beneath all utilities. This densely compacted soil should not include debris or organic material, and may consist of:
  - In-place granular soil that is naturally dense;
  - In-place granular natural or fill soil that is initially loose to medium dense, but is then compacted in-place to a dense condition;
  - Imported structural fill that is placed and compacted to a dense condition; or

- A combination of these.

- If there is standing water in the trench, pump water to prepare the trench in the dry.

- Bedding and pipe-zone backfill should be consistent with the type and class of pipe used, and method of installation. Backfill the remainder of the trench in a manner similar to that described herein for structural fill.

- Follow our recommendations in the Section 8.2 for utility trenches and excavations.

- Cluster utilities to minimize utility excavations through steep slope areas.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 Dewatering and Drainage

Based on our understanding of the project, the proposed addition excavation will be located above the groundwater table. However, we do anticipate limited seepage into the excavation from zones of perched groundwater could occur. We expect that trenches and sumps will be adequate for any necessary dewatering during construction of the project.

We recommend exposed soil be protected from rain and surface water during construction. Construction related surface water and drainage should be managed to minimize infiltration, surface water run-off, and erosion. We recommend all construction related drainage be tight-lined to the bottom of the slope.

8.2 Temporary Open Cuts

The stability and safety of cut slopes depend on a number of factors, including:

- The type and density of the soil;
- The presence and amount of any seepage;
- The depth of cut;
- The proximity of the cut to any surcharge loads near the top of the cut such as stockpiled material, traffic loads, structures, etc., and the magnitude of these surcharges;
- The duration of the open excavation; and
- The care and methods used by the contractor.

The Occupational Safety and Health Administration (OSHA) and Washington State Department of Labor and Industries – Division of Occupational Safety and Health (DOSH) classify the Fill and Medium Stiff to Stiff Sandy Silt as a Type C soil and the Hard Sandy Silt and Dense Sandy and Hard Clay as a Type B soils. We make the following recommendations for open cuts in Type B and C soils.

- Maintain the maximum allowable slope for excavations less than 20 feet deep, which is 1.5H:1V (Type C) and 1H:1V (Type B).

- Protect the slope from erosion by using plastic sheeting.
- Limit the maximum duration of the open excavation to the shortest time possible.
- Place no surcharge loads (equipment, materials, etc.) within 10 feet of the top of the slope.

Because of the variables involved, actual slope angles required for stability in temporary cut areas can only be estimated before construction. We recommend that stability of the temporary slopes used for construction be the responsibility of the contractor, since the contractor is in control of the construction operation and is continuously at the site to observe the nature and condition of the subsurface. All excavations should be made in accordance with all local, state, and federal safety requirements.

Important. We recommend close monitoring of the temporary cuts during excavation. Flatter slopes may be necessary if unstable conditions are observed. Equipment that will provide the farthest possible clearance from the crest of the excavation will be required.

8.3 Shoring Monitoring Program

The intent of the shoring monitoring program is to provide early warning of the potential need for remedial measures if the shoring does not perform as anticipated. Attachment 2 discusses the recommended shoring monitoring program for this site.

9.0 GEOLOGIC HAZARD AREA BUFFERS AND SETBACKS

Based on our evaluation, and provided the proposed development is designed and constructed in accordance with our recommendations provided in this report, the proposed development:

- Would decrease the risk associated with landslide hazards at the site and mitigate the hazard level equal to or less than would exist if the provisions of LUC 20.25H were not modified;
- Would not increase the hazard to adjacent properties with similar steep slopes over the existing condition or result in a need for increased buffers on neighboring properties; and
- Can be designed safely under anticipated conditions.

Provided our recommendations are followed, we recommend that the geologic hazard area buffers and setbacks be eliminated and the proposed development within the steep slope critical area be permitted.

10.0 RECOMMENDED ADDITIONAL GEOTECHNICAL SERVICES

10.1 Continuing Design and Consultation Services

Recommendations in this report should be reviewed and modified as the project progresses. We will be available to discuss these issues with the design team as the design develops and as needed.

We recommend that Hart Crowser be given the opportunity to review geotechnical aspects of the final design plans and specifications to confirm that our recommendations were properly understood and implemented in the design contract documents. We need to verify our recommendations for foundation settlement once the structure load data are available.

10.2 Construction Services

During the construction phase of the project, we recommend that Hart Crowser review contractor submittals and provide a representative to observe:

- Installation of temporary shoring or permanent slope stabilization measures;
- Pile foundation installation;
- Installation of the permanent drainage system;
- Utility installation;
- Placement and testing of compacted material; and
- Other geotechnical engineering considerations that may arise during the course of construction.

The purpose of our observations is to verify compliance with design concepts and recommendations and to allow design changes or evaluation of appropriate construction methods in the event that subsurface conditions differ from those anticipated prior to the start of construction.

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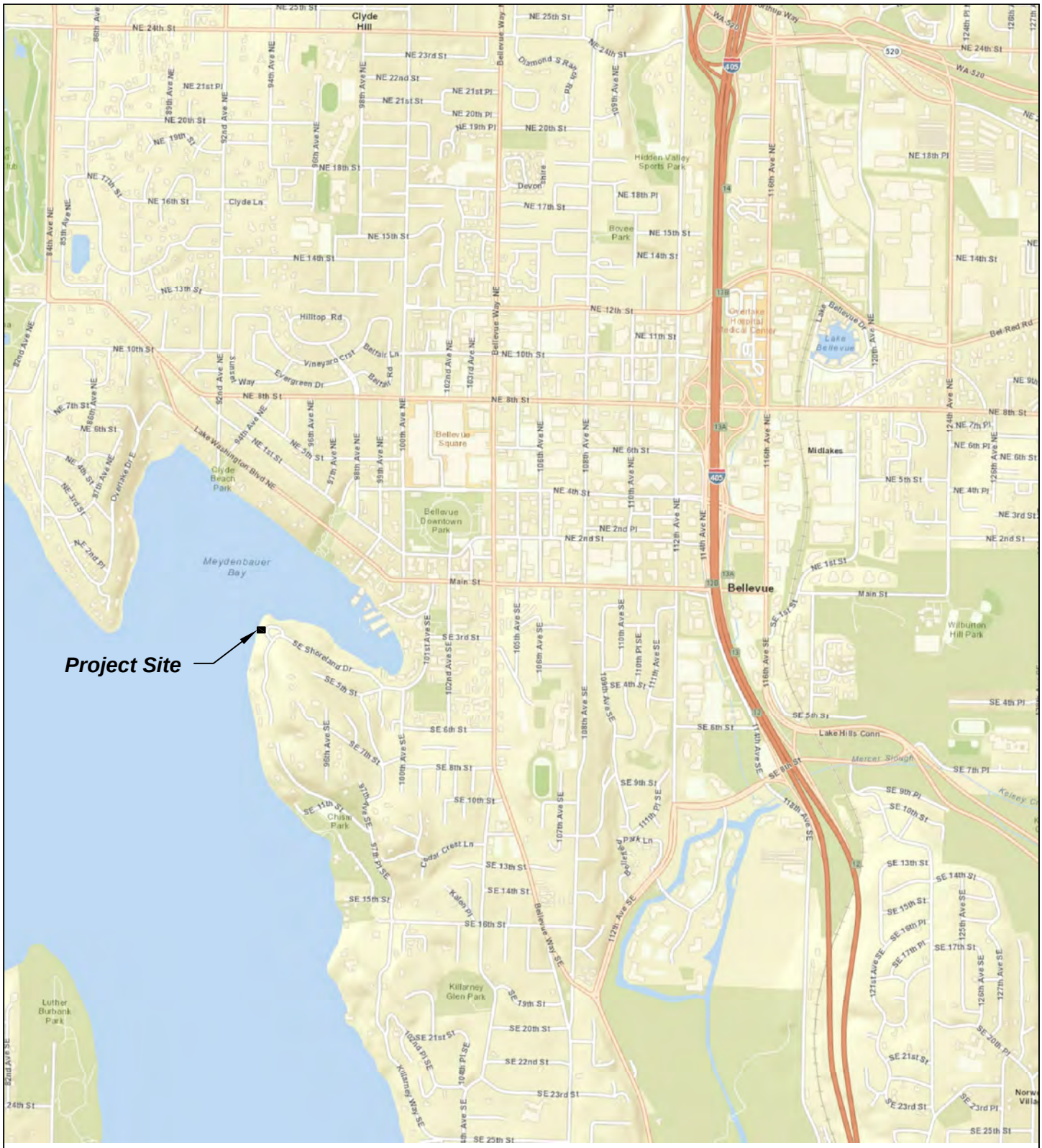

Source: Base map prepared from ArcGIS Online, 2014.

0 2000 4000

Scale in Feet

N

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Vicinity Map

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Figure

1

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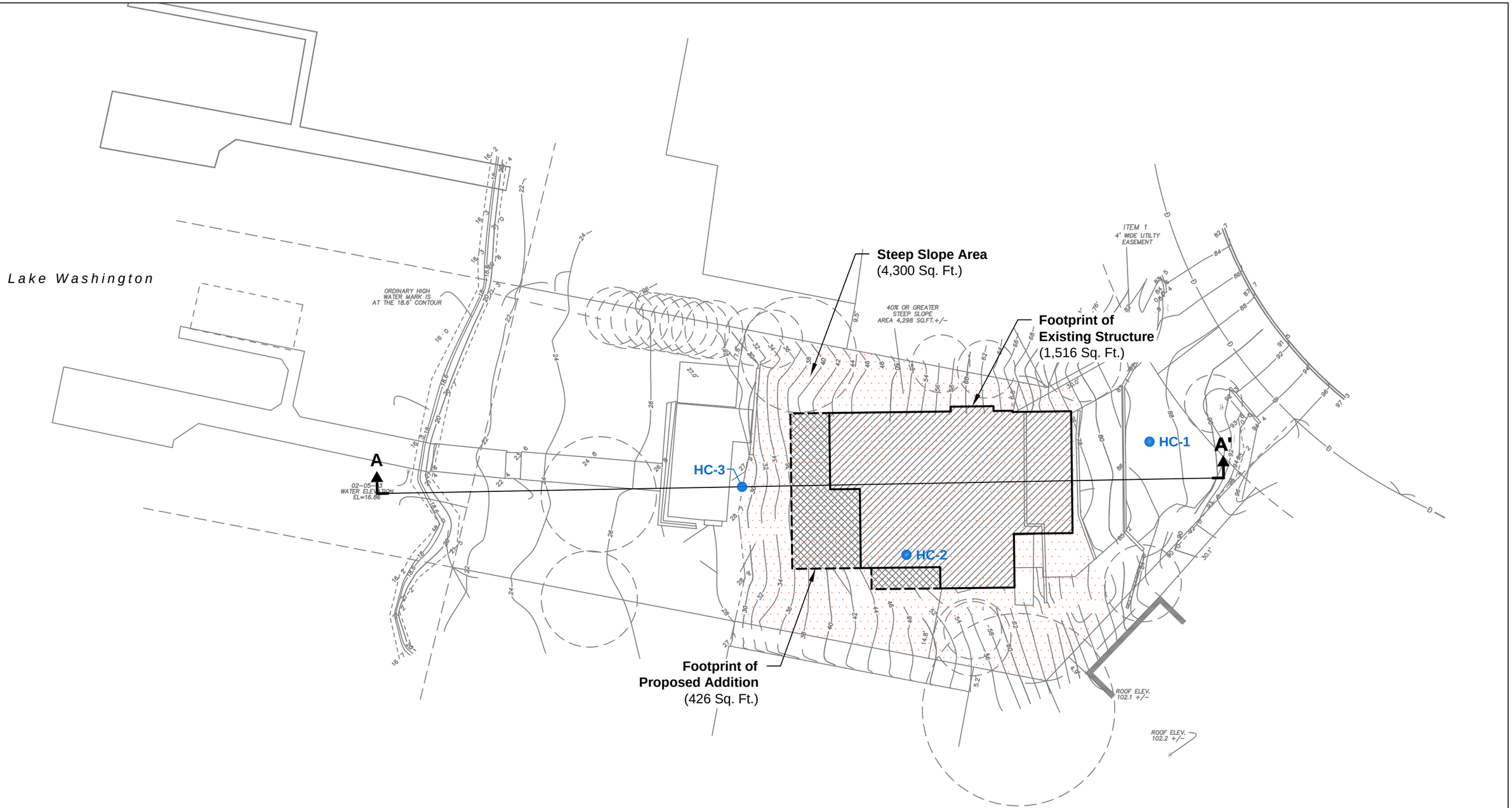

HC-1 ● Boring Location and Number

A A' Approximate Cross Section Location and Designation

Steep Slope Area

0 20 40
Scale in Feet

N

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Site and Exploration Plan

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Figure
2

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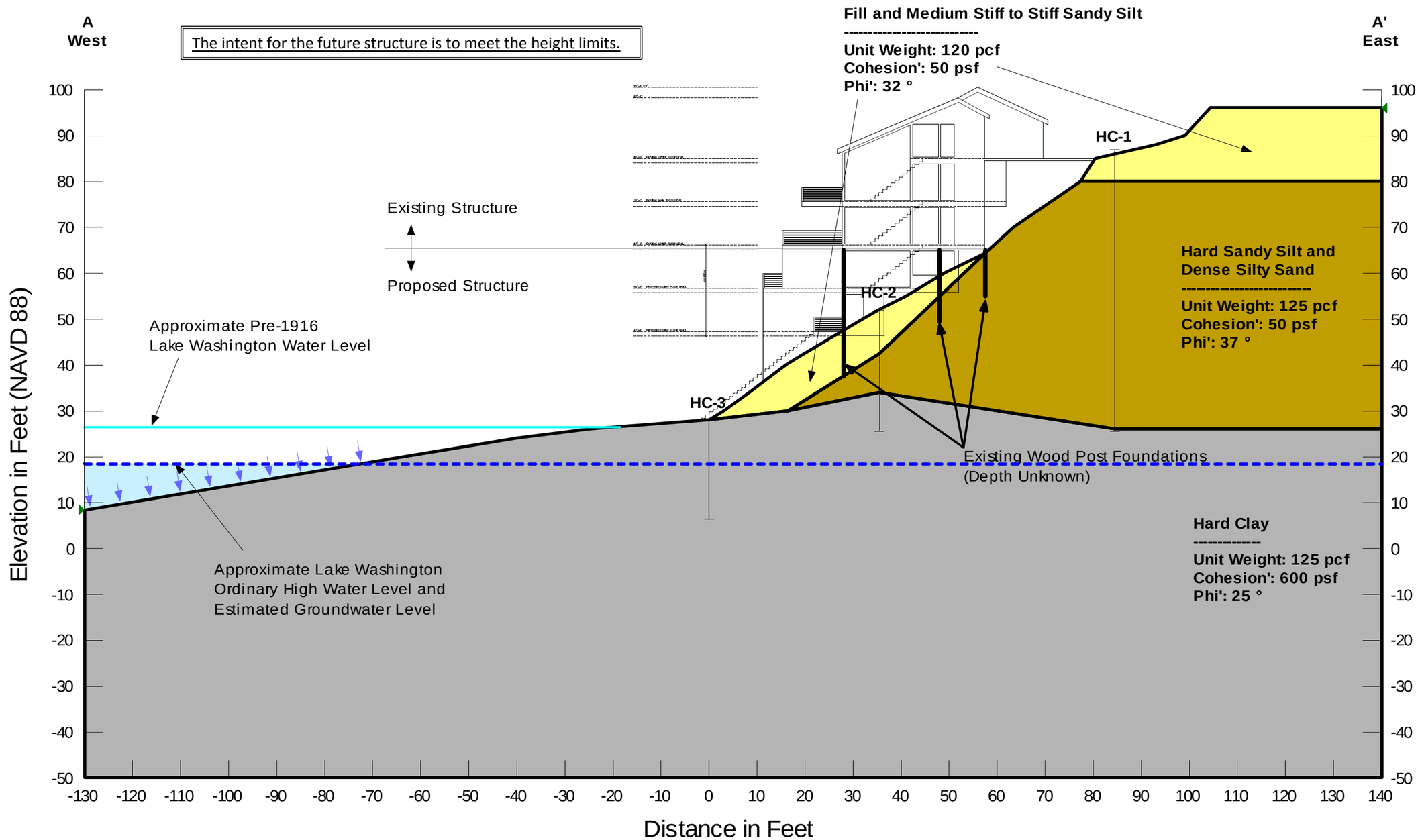

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Generalized Suburface Cross Section A-A'

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Figure
3

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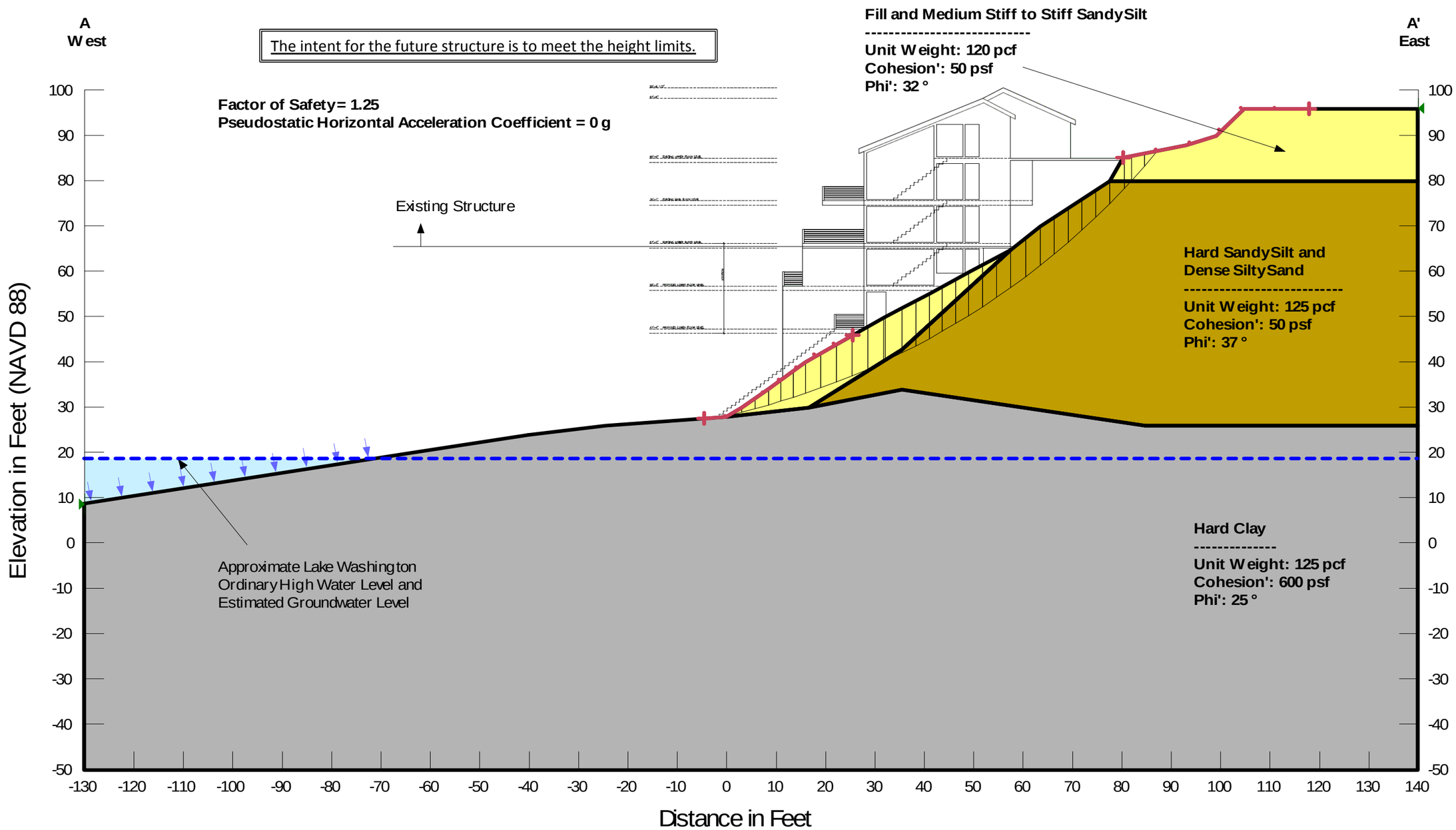

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Static Slope Stability - Cross Section A-A' - Existing Conditions

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Figure
4

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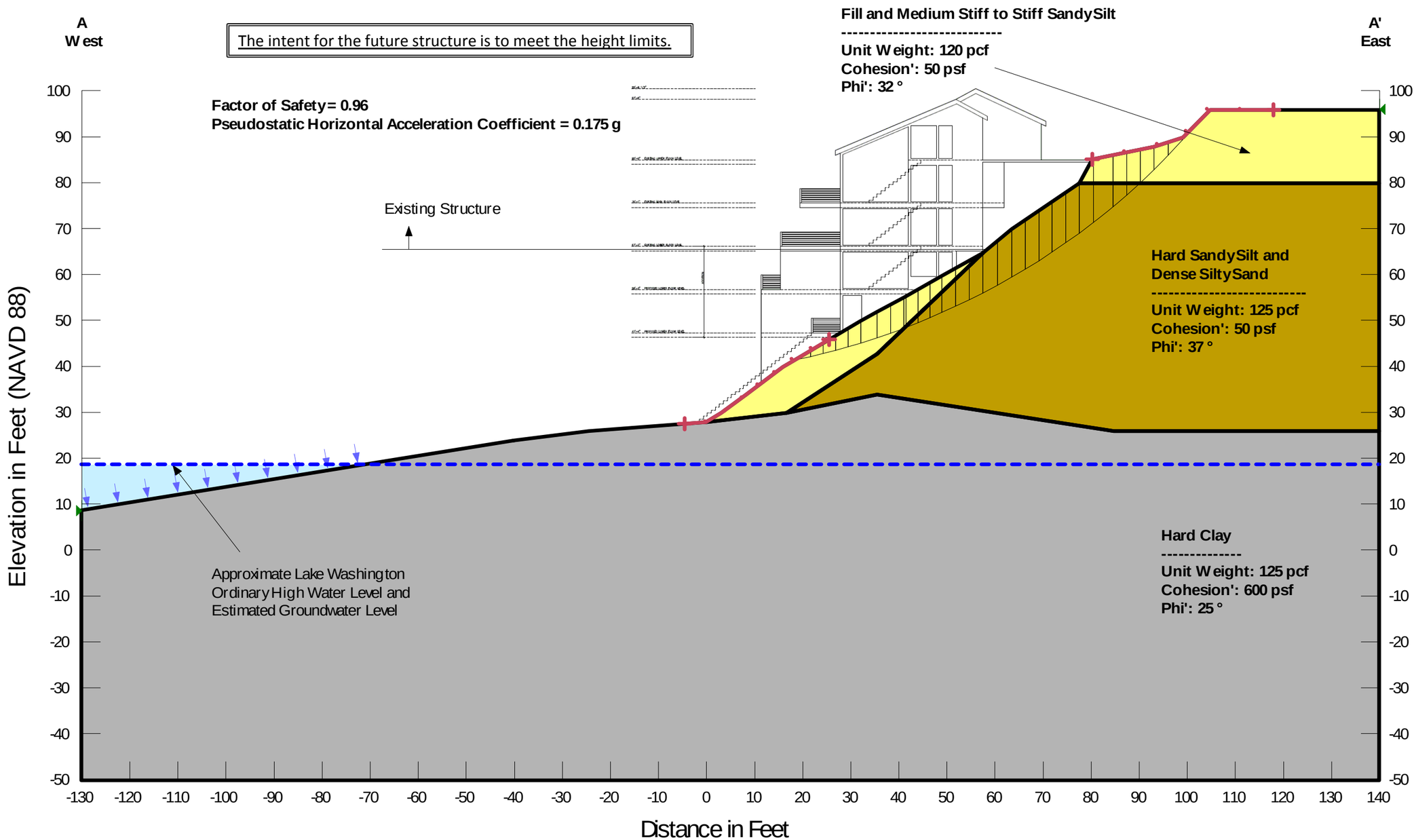

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Seismic Slope Stability - Cross Section A-A' - Existing Conditions

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Figure 5

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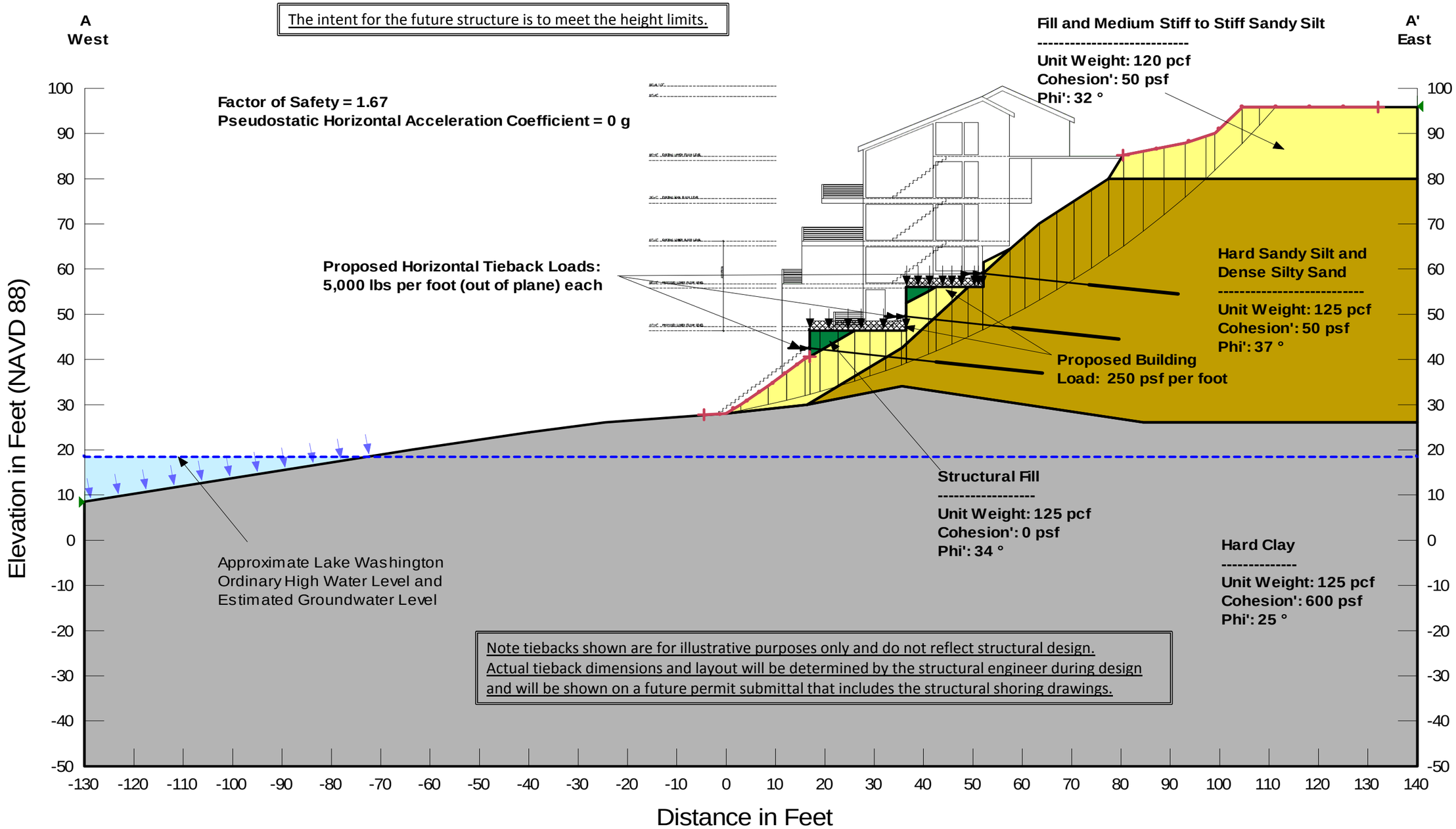

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Static Slope Stability - Cross Section A-A' - Proposed Development

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Figure
6

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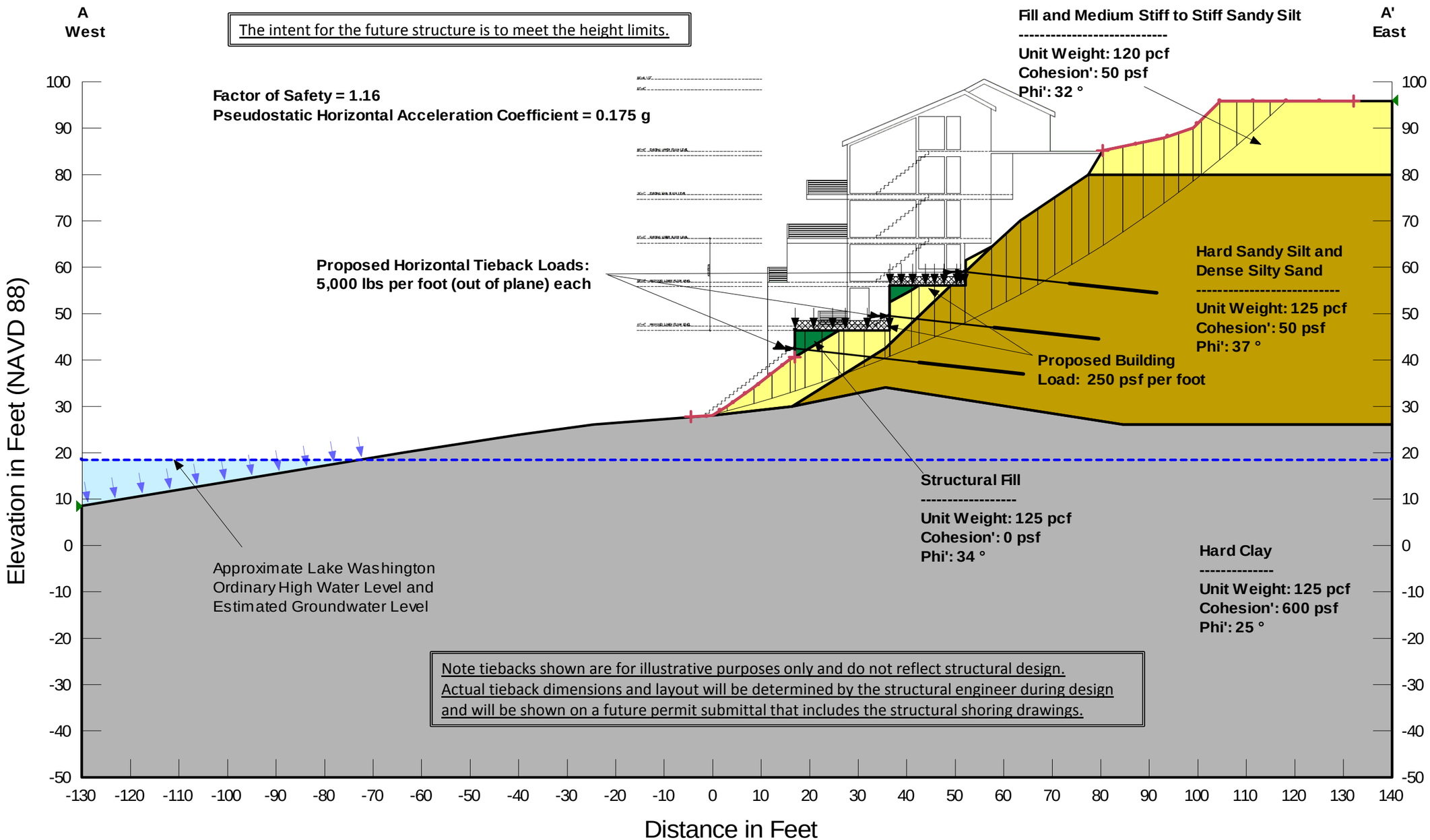

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Seismic Slope Stability - Cross Section A-A' - Proposed Development

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Figure 7

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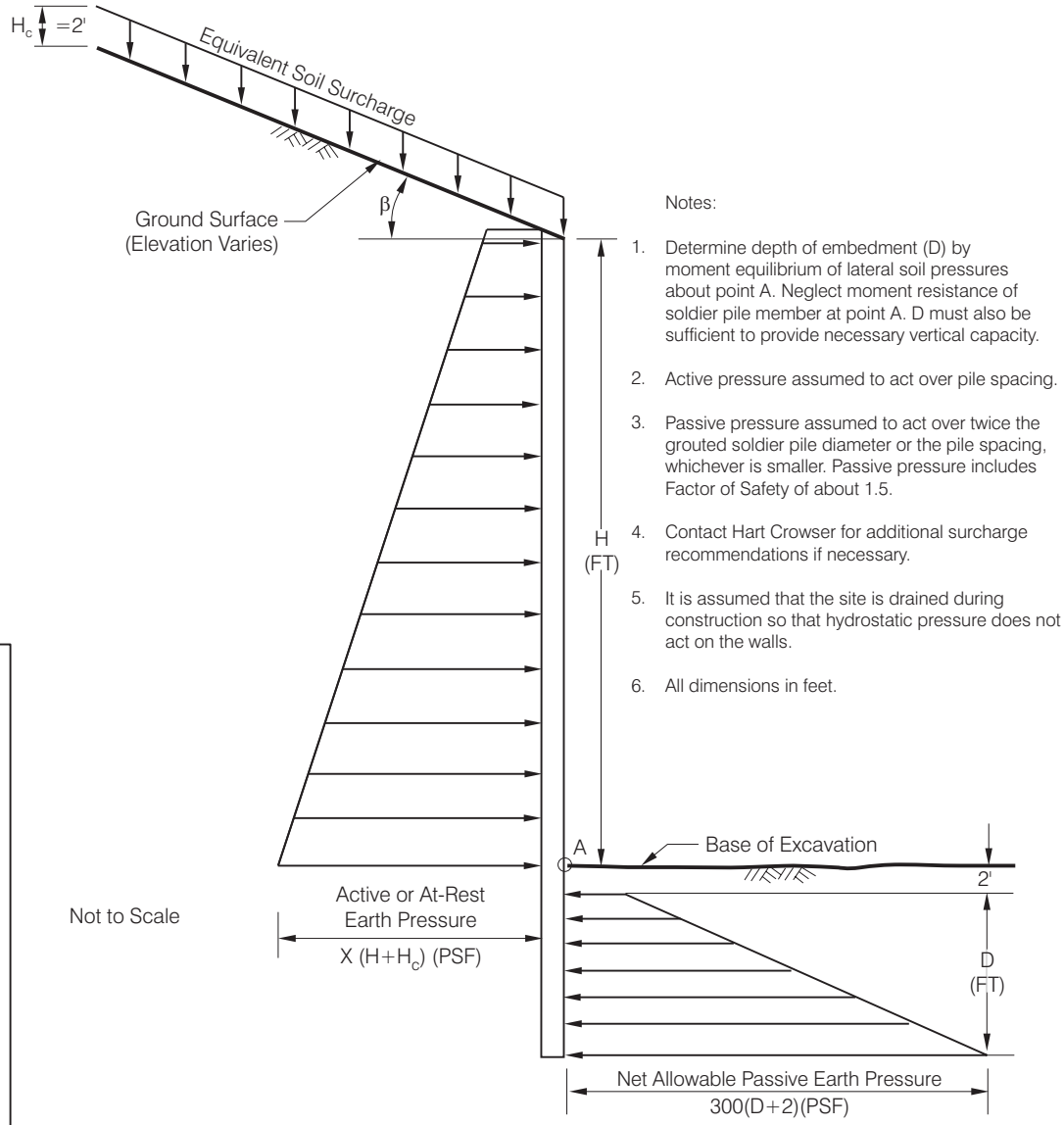
A. Lateral Soil Pressure



Not to Scale

Notes:

1. Determine depth of embedment (D) by moment equilibrium of lateral soil pressures about point A. Neglect moment resistance of soldier pile member at point A. D must also be sufficient to provide necessary vertical capacity.
2. Active pressure assumed to act over pile spacing.
3. Passive pressure assumed to act over twice the grouted soldier pile diameter or the pile spacing, whichever is smaller. Passive pressure includes Factor of Safety of about 1.5.
4. Contact Hart Crowser for additional surcharge recommendations if necessary.
5. It is assumed that the site is drained during construction so that hydrostatic pressure does not act on the walls.
6. All dimensions in feet.

Recommended Minimum Embedment Depth (D) of 10.0 Feet

Recommended Values of X		
β	X, Active	X, At Rest
0°	34	53
15°	36	58
20°	39	62
25°	42	68
30°	48	77
34°	56	89
35°	59	95
37°	80	128

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Design of Cantilever Soldier Pile Shoring

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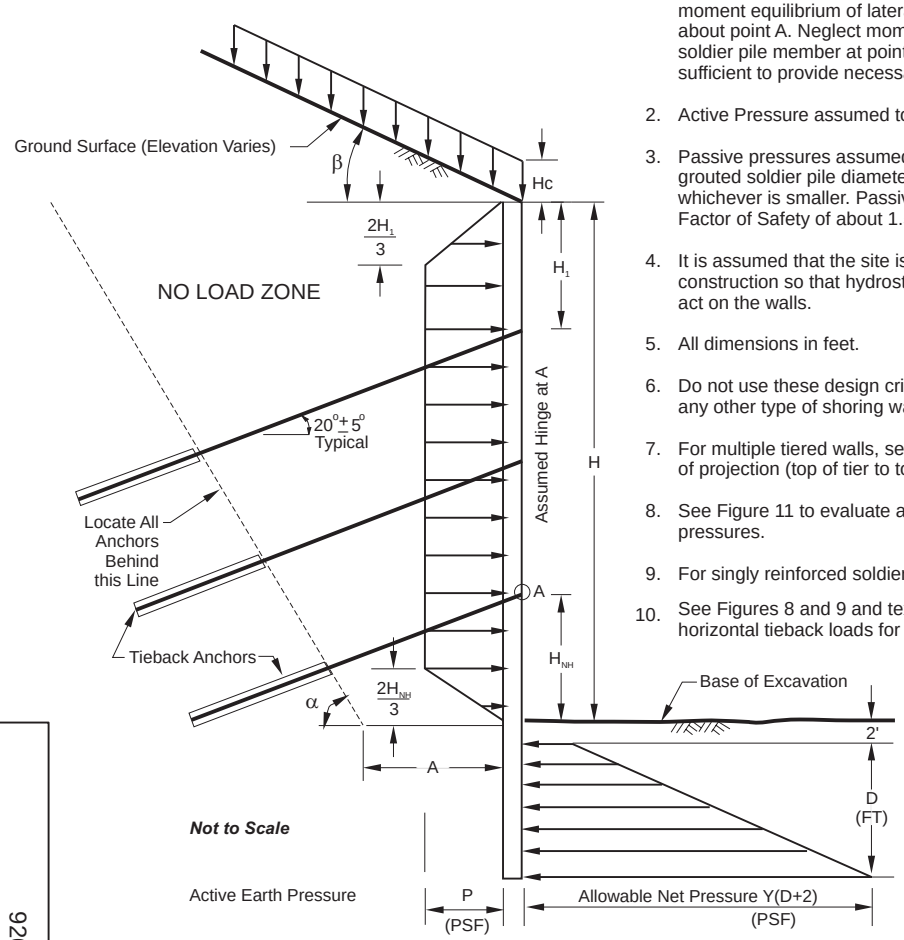
Figure 8

A. General Soil Pressures - No Load Zone



Notes:
1. Determine depth of embedment (D) by moment equilibrium of lateral soil pressures about point A. Neglect moment resistance of soldier pile member at point A. D must also be sufficient to provide necessary vertical capacity.
2. Active Pressure assumed to act over pile spacing.
3. Passive pressures assumed to act over twice the grouted soldier pile diameter or the pile spacing, whichever is smaller. Passive Pressures include Factor of Safety of about 1.5.
4. It is assumed that the site is drained during construction so that hydrostatic pressures do not act on the walls.
5. All dimensions in feet.
6. Do not use these design criteria for design of any other type of shoring wall.
7. For multiple tiered walls, select the steepest angle of projection (top of tier to top of tier).
8. See Figure 11 to evaluate additional surcharge pressures.
9. For singly reinforced soldier piles, $H_{NH} = H - H_1$
10. See Figures 8 and 9 and text regarding minimum horizontal tieback loads for slope stabilization.

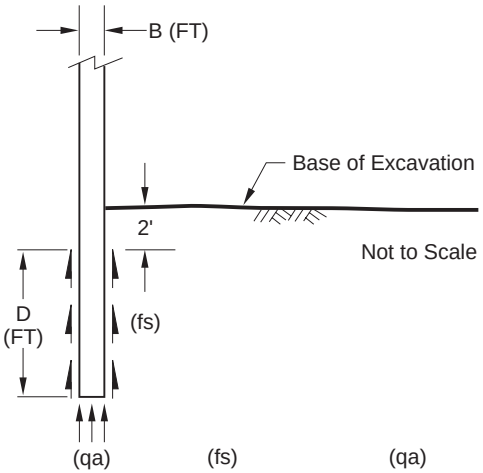
Recommended Values of X		
β	X, Active	X, At Rest
0°	22	34
15°	24	38
20°	25	40
25°	28	44
30°	31	50
34°	36	58
35°	39	62
37°	52	83

Recommended Values of Y in PCF	
Hard Sandy SILT and Dense Silty SAND (Above GWT)	300

$$P = \frac{X(H+Hc)^2}{H-1/3H_{NH}-1/3H_1}$$

Excavation Wall	α	A
North and South	60°	H/4
West-Facing	See Figure 10	

B. Vertical Capacity of Soldier Pile



	(fs) Allowable Friction	(qa) Allowable End Bearing
Native	1.0 KSF	$\frac{4D}{B} \leq$ 30KSF

Recommended Minimum Embedment Depth
10 Feet Into Bearing Layer

C. Tentative Anchor Pullout Resistance

	Allowable (Adhesion)
Hard Sandy SILT and Dense Silty SAND	2.0 KSF

Verify with Load Test to 200% of Design Stress Level.
See Text.

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Design of Braced/Tieback Shoring

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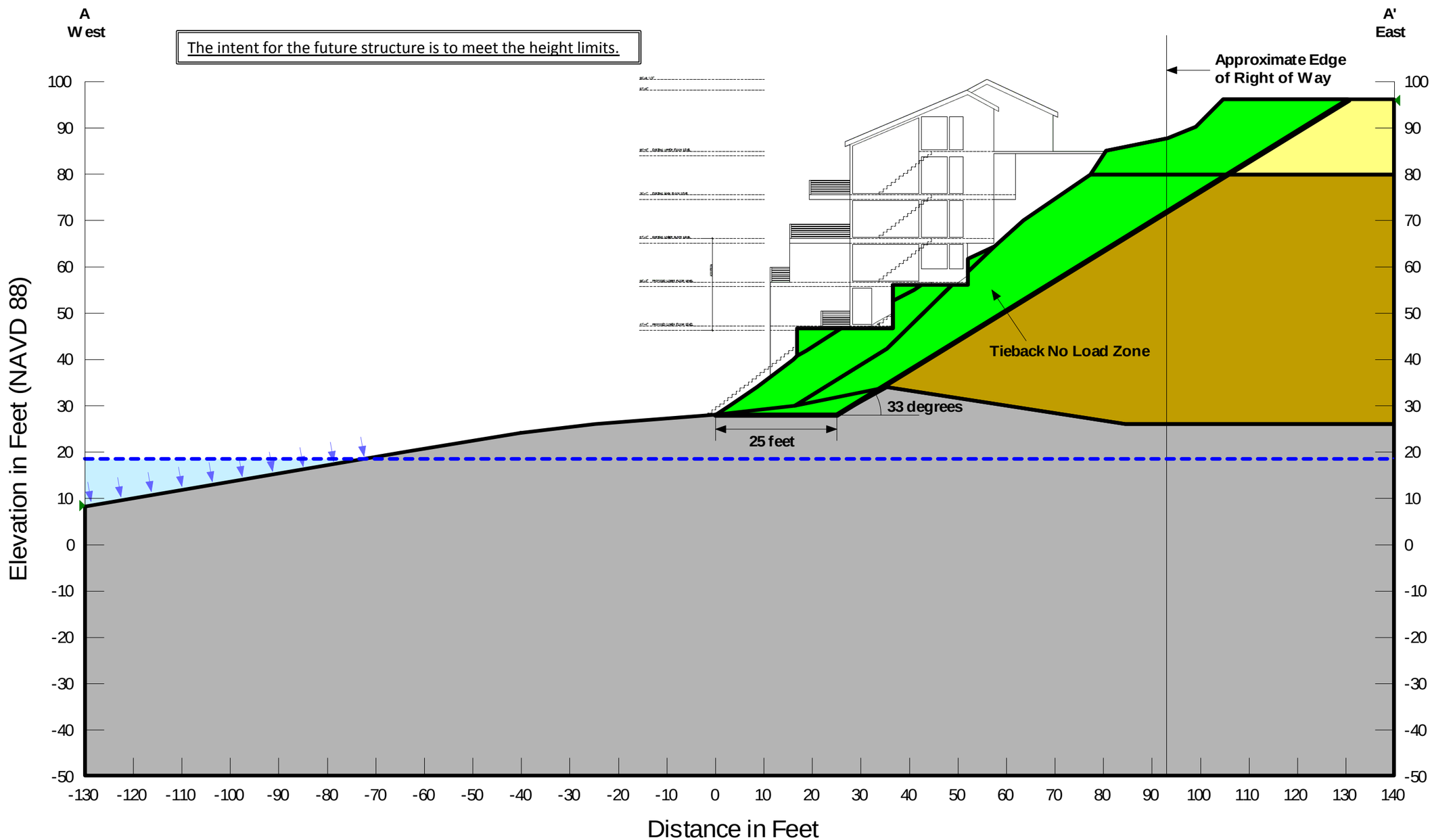
Figure 9



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Tieback No Load Zone - West-Facing Basement Walls

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Figure **10**

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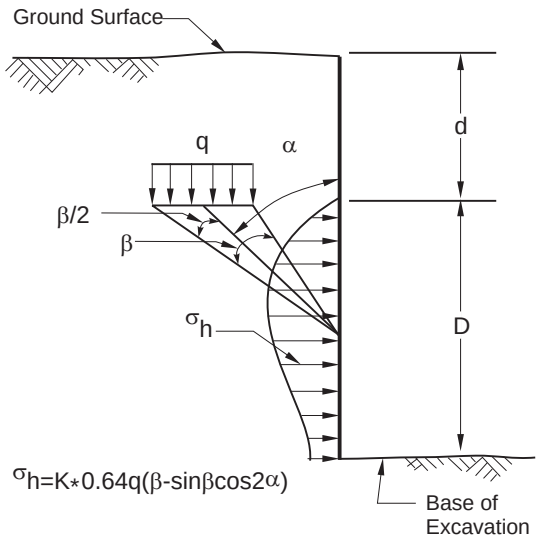
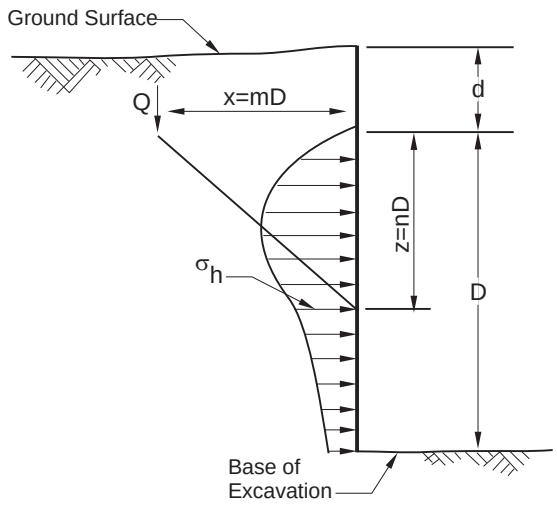
A. Strip Footing Cross Section View

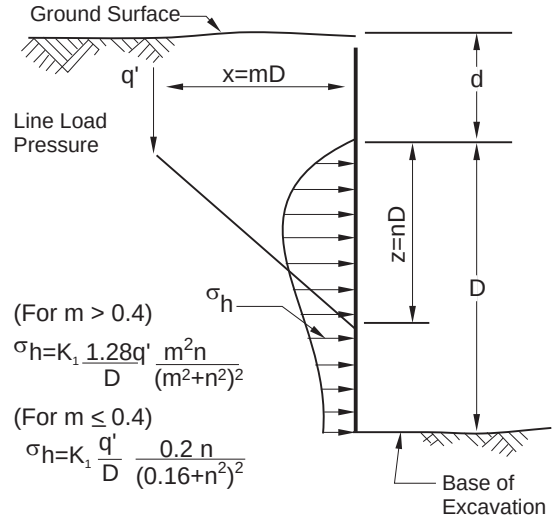


$\sigma_h = K * 0.64q(\beta - \sin\beta\cos 2\alpha)$

B_1. Small Isolated Footing Cross Section View



C. Continuous Wall Footing Parallel to Excavation Cross Section View



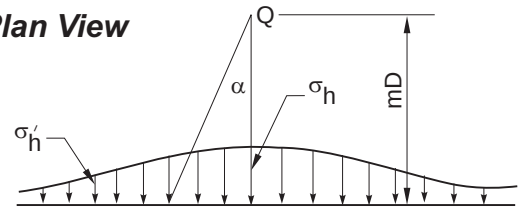
(For m > 0.4)

$$\sigma_h = K_1 \frac{1.28q'}{D} \frac{m^2 n}{(m^2+n^2)^2}$$

(For m ≤ 0.4)

$$\sigma_h = K_1 \frac{q'}{D} \frac{0.2\,n}{(0.16+n^2)^2}$$

B_2. Plan View



$\sigma'_h = \sigma_h \cos^2(1.1\alpha)$

(For m> 0.4)

$$\sigma_h = K_1 \frac{1.77Q}{D^2} \frac{m^2 n^2}{(m^2+n^2)^3}$$

(For m≤ 0.4)

$$\sigma_h = K_1 \frac{0.28Q}{D^2} \frac{n^2}{(0.16+n^2)^3}$$

Definition and Units

Q	Footing Load in Pounds
D	Excavation Depth below Footing in Feet
d	Depth to Base of Footing in Feet
σ_h	Lateral Soil Pressure in PSF
q	Unit Loading Pressure in PSF
q'	Footing Load in Pounds per Foot
α, β	Radians

K_1	Conditions
0.35	Active earth pressures on a flexible wall (e.g., shoring)
0.5	At rest conditions, where surcharge loads exist prior to excavation
1.0	At rest conditions, where surcharge loads are applied after construction of permanent wall

Notes:
1. Lateral pressures from adjacent structures should be added to lateral pressures on Figures 9 and 10.
2. Wall footings acting other than parallel to the excavation can be treated as series of discrete point loads, using Approach B.
3. Contact Hart Crowser for surcharge recommendations, if necessary.

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Surcharge Pressures Determination of Lateral Pressure Acting on Adjacent Shoring

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Figure 11

ATTACHMENT 1
Tieback Testing Program

ATTACHMENT 1
TIEBACK TESTING PROGRAM

Performance Test. A minimum of two performance tests per soil type should be completed prior to installation of production anchors. Each performance test should be conducted according to the following procedure:

1. The geotechnical engineer will select the testing locations with input from the shoring subcontractor.
2. The maximum stress in the prestressing steel should not exceed 80 percent of the ultimate tensile strength during performance testing (based on the Post Tensioning Institute manual). The soldier pile and tieback may require extra reinforcement to permit stressing to 200 percent of design load (DL) as required for the performance test.
3. The performance test will measure anchor stress and displacement incrementally to values of unit skin friction equal to 200 percent of the DL. For temporary shoring, load the anchor in increments as presented in Table 1-1.

Table 1-1 – Performance Test for Temporary Shoring

Load Level	Hold Time	Load Level	Hold Time
AL	Until Stable	1.75DL	Until Stable
0.25DL	10 min	1.50DL	Until Stable
0.50DL	10 min	1.25DL	Until Stable
0.75DL	10 min	1.00DL	Until Stable
1.00DL	10 min	0.75DL	Until Stable
1.25DL	10 min	0.50DL	Until Stable
1.50DL	60 min (Creep)	0.25DL	Until Stable
1.75DL	10 min	AL	Until Stable
2.00DL	10 min		

4. For 10-minute hold times, obtain and record deflection measurements during loading at intervals of 30 seconds, 1 minute, 2 minutes, 3 minutes, 5 minutes, 6 minutes, and 10 minutes. Measure to an accuracy of 0.01 inch.
5. Perform a creep test at the 150 percent of design stress reading by holding the load constant to within 50 psi and recording readings at 30 seconds, 1 minute, 2 minutes, 3 minutes, 5 minutes, 6 minutes, 10 minutes, 20 minutes, 30 minutes, 50 minutes, and 60 minutes. Continue the creep test if the creep criterion between 6 and 60 minutes is not met. This creep test may be extended to as long as 5 hours, taking a reading every 15 minutes after the initial 60 minutes if the creep criteria (see 6 below) is not met before the 300-minute reading.
6. A successful test is one that does not experience pullout failure, holds the maximum test unit stress without considerable creep, and satisfies the apparent free-length criterion.
   - Pullout failure occurs when test measurements no longer exhibit a linear or near-linear relationship between unit stress and movement over the entire 200 percent stress range.

- Noticeable creep is defined as a rate of movement of not more than 0.04 inch between the 1- and 10-minute readings, or not more than 0.08 inch between the 6- and 60-minute readings. If the reading does not stabilize to 0.08 inch or less per log cycle, the test shall be considered to fail the creep criterion.
- Minimum apparent free length, based on the measured elastic and residual movement, should be greater than 80 percent of the designed free length plus the jack length.

7. Perform tests without backfill ahead of the anchor, if the hole will remain open, to avoid any contributory resistance by the backfill. If the hole will not remain open during testing, provide a bond breaker on the tie rods and backfill the no-load zone with a non-cohesive, non-structural mixture.

Proof Test. For each production tieback anchor, follow the proof testing procedures outlined below:

1. Load each anchor to 133 percent of the DL in increments of approximately 25 percent of the design load (i.e., 0.25 DL, 0.50 DL, 0.75 DL, 1.00 DL, 1.25 DL, and 1.33 DL). The maximum stress in the prestressing steel should not exceed 80 percent of the ultimate tensile strength during proof testing.
2. Hold each incremental load for a period long enough to obtain a stable deflection measurement while recording deflections at each load increment. Hold the 133 percent load for a minimum of 10 minutes, recording the movement at intervals of 30 seconds, 1 minute, 2 minutes, 5 minutes, 6 minutes, and 10 minutes.
3. A successful test is one that meets the same acceptance criteria as performance anchors, except that the creep portion of the test need not exceed 10 minutes if the 10-minute creep criterion is met.
4. Following proof loading, lock off each tieback anchor to 80 to 100 percent of the DL, except as specified.

ATTACHMENT 2
Shoring Monitoring Program

ATTACHMENT 2
SHORING MONITORING PROGRAM

The intent of the shoring monitoring program is to provide early warning of the potential need for remedial measures if the shoring does not perform as anticipated. We recommend that adjacent building surveys and optical surveying be included in the shoring monitoring program during construction.

All monitoring data should be submitted to Hart Crowser for review on a weekly basis. The data will be included in our field transmittals to the project team during construction. Details of our expectations for shoring monitoring are included in the following paragraphs.

Adjacent Building Surveys. We recommend that the adjacent buildings be surveyed before, during, and after construction. The pre-construction survey will document the baseline of existing conditions (e.g., identifying the size and locations of any cracks). The surveys should consist of a videotape and/or photographs of the interior and exterior of adjacent buildings and detailed mapping of all cracks. Any existing cracks could be monitored with a crack gauge placed across the crack.

Optical Surveying. We recommend optical surveying of horizontal and vertical movement of the following items: (1) the surface of the adjacent streets, (2) structures on and adjacent to the site, and (3) the shoring system itself. The contractor, in coordination with the geotechnical engineer, should establish reference lines adjacent to the excavation. The points on the structures can be set either at the base or on the roof of the buildings. Monitoring of the existing structure should be coordinated with the structural engineer.

Monitoring of the shoring system should include measurements of vertical and horizontal movements at approximately a 15-foot spacing or every other soldier pile.

The measuring system for the shoring monitoring should have an accuracy of at least 0.005 foot. All reference points on the existing ground surface should be installed and read prior to commencing the excavation. The frequency of readings will depend on the results of previous readings and the rate of construction. Readings on the external points should be taken at least once a week through construction until below-grade structural elements (floors, decks, columns, etc.) are completed, or as specified by the structural and geotechnical engineers. Readings on the top of the shoring wall and the face of existing buildings on or adjacent to the property should be taken at least twice a week during this time. We recommend that an independent surveyor hired by the owner to record the data at least once per week with the other reading taken by the surveyor or contractor.

APPENDIX A
Field Exploration Methods and Analysis

APPENDIX A
FIELD EXPLORATION METHODS AND ANALYSIS

This appendix documents the processes Hart Crowser used in determining the nature (and quality) of the soil and groundwater underlying the project site addressed by this report. The discussion includes information on the following subjects:

- Explorations and Their Location;
- Hollow-Stem Auger Borings; and
- Standard Penetration Test (SPT) Procedures.

Explorations and Their Location

Subsurface explorations for this project include hollow-stem auger borings. The exploration logs within this appendix show our interpretation of the drilling (probing/excavation), sampling, and testing data. The logs indicate the depth where the soils change. Note that the change may be gradual. In the field, we classified the samples taken from the explorations according to the methods presented on Figure A-1 - Key to Exploration Logs. This figure also provides a legend explaining the symbols and abbreviations used in the logs.

Location of Explorations. Figure 2 shows the location of explorations, located by hand measuring or pacing from existing physical features. The ground surface elevations at these locations were interpreted from elevations shown on the site survey by GeoDimensions dated March 1, 2013. The method used determines the accuracy of the location and elevation of the explorations.

Hollow-Stem Auger Borings

With depths ranging from 21.5 to 61.5 feet below the ground surface, three hollow-stem auger borings, designated HC-1 through HC-3, were drilled from April 16 to 17, 2014. The borings used a 4-inch inside diameter hollow-stem auger and were advanced with a track-mounted drill rig or an Acker portable drill rig subcontracted by Hart Crowser. The drilling was continuously observed by an engineering geologist from Hart Crowser. Detailed field logs were prepared of each boring. Using the Standard Penetration Test (SPT) and thin-walled Shelby tubes, we obtained samples at 2-1/2- to 5-foot-depth intervals.

The borings logs are presented on Figures A-2 through A-4 at the end of this appendix.

Standard Penetration Test (SPT) Procedures

This test is an approximate measure of soil density and consistency. To be useful, the results must be used with engineering judgment in conjunction with other tests. The SPT (as described in ASTM D 1586) was used to obtain disturbed samples. This test employs a standard 2-inch outside diameter split-spoon sampler. Using a 140-pound autohammer, free-falling 30 inches, the sampler is driven into the soil for 18 inches. The number of blows required to drive the sampler <u>the last 12 inches only</u> is the Standard Penetration Resistance. This resistance, or blow count, measures the relative density of

granular soils and the consistency of cohesive soils. The blow counts are plotted on the boring logs at their respective sample depths.

Soil samples are recovered from the split-barrel sampler, field classified, and placed into water-tight jars. They are then taken to Hart Crowser's laboratory for further testing.

In the Event of Hard Driving

Occasionally very dense materials preclude driving the total 18-inch sample. When this happens, the penetration resistance is entered on logs as follows:

Penetration less than 6 inches. The log indicates the total number of blows over the number of inches of penetration.

Penetration greater than 6 inches. The blow count noted on the log is the sum of the total number of blows completed after the first 6 inches of penetration. This sum is expressed over the number of inches driven that exceed the first 6 inches. The number of blows needed to drive the first 6 inches are not reported. For example, a blow count series of 12 blows for 6 inches, 30 blows for 6 inches, and 50 (the maximum number of blows counted within a 6-inch increment for SPT) for 3 inches would be recorded as 80/9.

Key to Exploration Logs

Sample Description

Classification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide.

Soil descriptions consist of the following:
Density/consistency, moisture, color, minor constituents, MAJOR CONSTITUENT, additional remarks.

Density/Consistency

Soil density/consistency in borings is related primarily to the Standard Penetration Resistance. Soil density/consistency in test pits and probes is estimated based on visual observation and is presented parenthetically on the logs.

SAND or GRAVEL Density	Standard Penetration Resistance (N) in Blows/Foot	SILT or CLAY Consistency	Standard Penetration Resistance (N) in Blows/Foot	Approximate Shear Strength in TSF
Very loose	0 to 4	Very soft	0 to 2	<0.125
Loose	4 to 10	Soft	2 to 4	0.125 to 0.25
Medium dense	10 to 30	Medium stiff	4 to 8	0.25 to 0.5
Dense	30 to 50	Stiff	8 to 15	0.5 to 1.0
Very dense	>50	Very stiff	15 to 30	1.0 to 2.0
		Hard	>30	>2.0

Sampling Test Symbols

- 1.5" I.D. Split Spoon
- Grab (Jar)
- 3.0" I.D. Split Spoon
- Shelby Tube (Pushed)
- Bag
- Cuttings
- Core Run

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

Moisture

Dry	Little perceptible moisture
Damp	Some perceptible moisture, likely below optimum
Moist	Likely near optimum moisture content
Wet	Much perceptible moisture, likely above optimum

Minor Constituents	Estimated Percentage
Trace	<5
Slightly (clayey, silty, etc.)	5 - 12
Clayey, silty, sandy, gravelly	12 - 30
Very (clayey, silty, etc.)	30 - 50

Laboratory Test Symbols

GS	Grain Size Classification
CN	Consolidation
UU	Unconsolidated Undrained Triaxial
CU	Consolidated Undrained Triaxial
CD	Consolidated Drained Triaxial
QU	Unconfined Compression
DS	Direct Shear
K	Permeability
PP	Pocket Penetrometer Approximate Compressive Strength in TSF
TV	Torvane Approximate Shear Strength in TSF
CBR	California Bearing Ratio
MD	Moisture Density Relationship
AL	Atterberg Limits

Water Content in Percent
Liquid Limit
Natural
Plastic Limit

PID	Photoionization Detector Reading
CA	Chemical Analysis
DT	In Situ Density in PCF
OT	Tests by Others

Groundwater Indicators

- Groundwater Level on Date or (ATD) At Time of Drilling
- Groundwater Seepage (Test Pits)

Sample Key

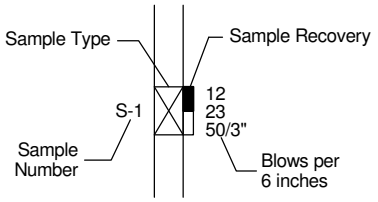

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Figure A-1

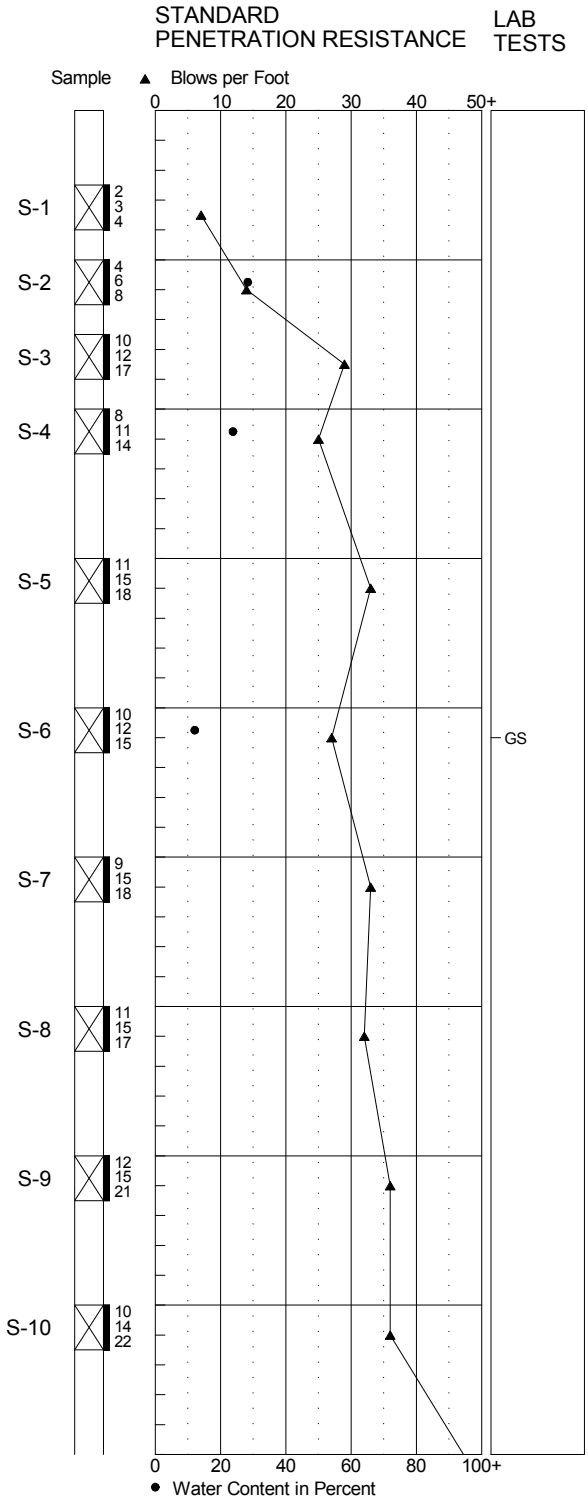
KEY SHEET 1901600-BL.GPJ HC_CORP.GDT 5/30/14

Boring Log HC-1

Location: See Figure 2.
Approximate Ground Surface Elevation: 87 Feet
Horizontal Datum:
Vertical Datum: NAVD 88

Drill Equipment: Bobcat Track
Hammer Type: SPT
Hole Diameter: 4 inches
Logged By: J. Overton Reviewed By: B. Cook

USCS Class	Soil Descriptions	Depth in Feet
	Concrete.	0
ML	Medium stiff to stiff, moist, mottled light brown, fine sandy SILT; occasional fine sand lenses. (Possible FILL above 7 feet.)	
	Grades to very stiff.	5, 10
SM	Dense, damp to moist, light brown, very silty, fine SAND.	15
ML	Hard, damp to moist, light brown, very fine sandy SILT.	20, 25
	Becomes wet.	30, 35
ML	Hard, wet, gray, slightly fine sandy SILT.	40
ML	Hard, moist to wet, mottled gray brown, slightly fine sandy SILT.	45



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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Figure A-2 1/2

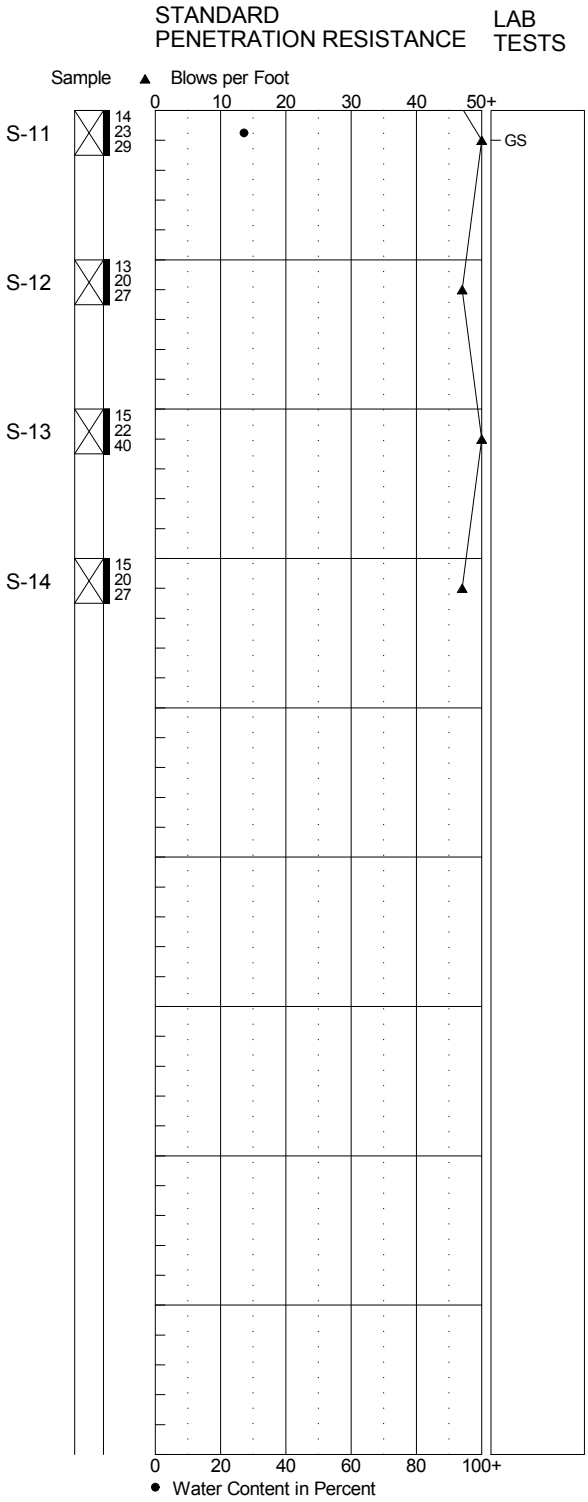
NEW BORING LOG 1901600-BL.GPJ HC_CORP.GDT 6/13/14

Boring Log HC-1

Location: See Figure 2.
Approximate Ground Surface Elevation: 87 Feet
Horizontal Datum:
Vertical Datum: NAVD 88

Drill Equipment: Bobcat Track
Hammer Type: SPT
Hole Diameter: 4 inches
Logged By: J. Overton Reviewed By: B. Cook

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet
ML		Hard, moist to wet, mottled gray brown, slightly fine sandy SILT. (cont'd)	45
			50
			55
			60
CL/CH		Hard, moist, mottled blue-gray CLAY; occasional fine sand lenses.	
		Bottom of Boring at 61.5 Feet.	
		Started 04/17/14.	
		Completed 04/17/14.	65
			70
			75
			80
			85
			90



NEW BORING LOG 1901600-BL.GPJ HC_CORP.GDT 6/13/14

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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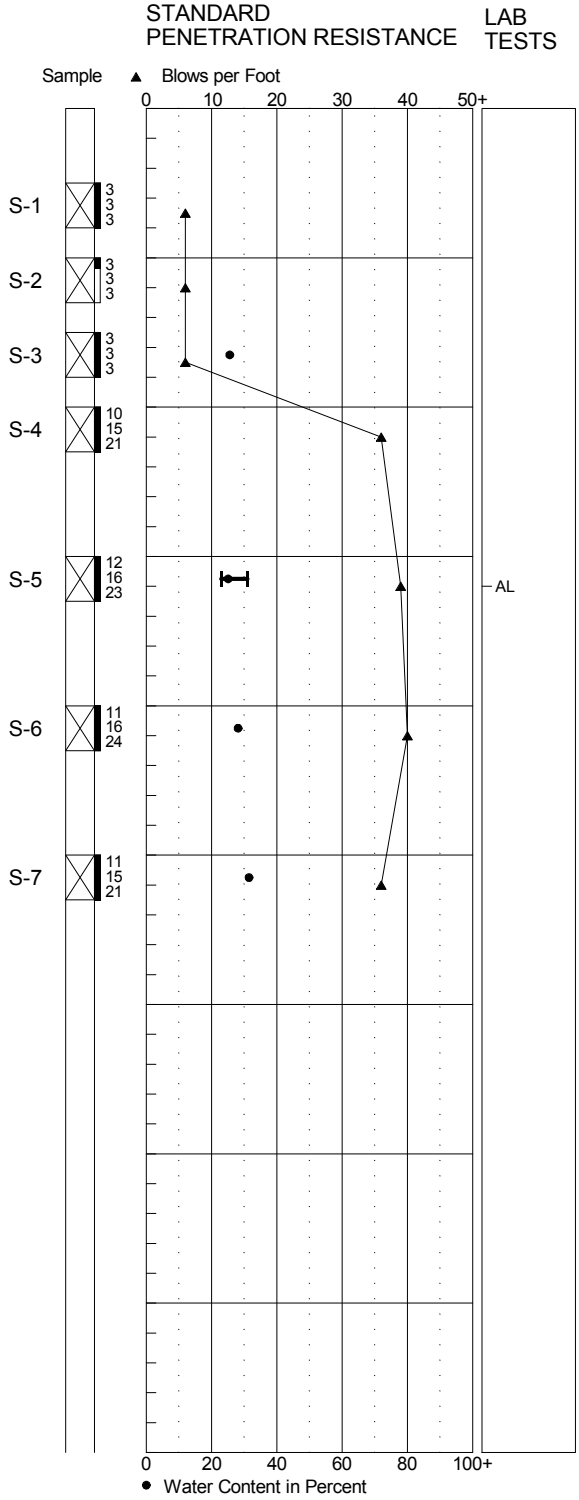
Boring Log HC-2

Location: See Figure 2.
Approximate Ground Surface Elevation: 52 Feet
Horizontal Datum:
Vertical Datum: NAVD 88

Drill Equipment: Acker
Hammer Type: SPT
Hole Diameter: 4 inches
Logged By: J. Overton Reviewed By: B. Cook

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet
ML		Medium stiff, damp, light brown, very sandy SILT. (Possible FILL.)	0
SM		Loose, damp, light brown, very silty, fine SAND; scattered organic debris.	5
ML		Medium stiff, moist, light brown, very fine sandy SILT; trace organic material.	
ML		Hard, moist, blue-gray, slightly fine sandy SILT.	10
			15
CL/CH		Hard, moist, blue-gray CLAY; sheared texture with scattered slickensided surfaces (oriented approximately 64° from vertical).	20
			25
		Bottom of Boring at 26.5 Feet. Started 04/16/14. Completed 04/16/14.	30
			35
			40
			45



NEW BORING LOG 1901600-BL.GPJ HC_CORP.GDT 6/13/14

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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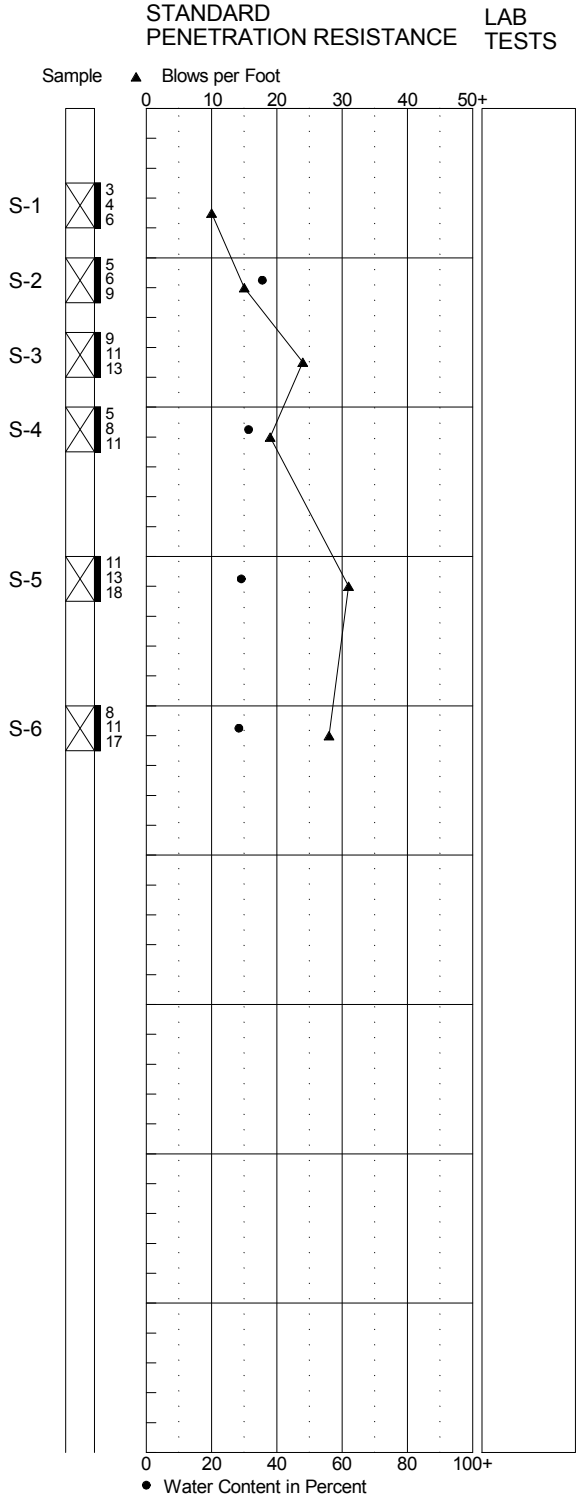
Figure A-3

Boring Log HC-3

Location: See Figure 2.
Approximate Ground Surface Elevation: 28 Feet
Horizontal Datum:
Vertical Datum: NAVD 88

Drill Equipment: Acker
Hammer Type: SPT
Hole Diameter: 4 inches
Logged By: J. Overton Reviewed By: B. Cook

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet
ML		Stiff, moist, light brown SILT.	0
CL		Stiff to hard, moist, blue-gray CLAY; drill action indicated possible gravel interbeds.	5
		Sheared texture below about 11 feet with scattered slickensided surfaces (oriented approximately 40° from vertical).	10, 15, 20
		Bottom of Boring at 21.5 Feet. Started 04/16/14. Completed 04/16/14.	25, 30, 35, 40, 45



NEW BORING LOG 1901600-BL.GPJ HC_CORP.GDT 6/13/14

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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Figure A-4

APPENDIX B
Laboratory Testing Program

APPENDIX B
LABORATORY TESTING PROGRAM

A laboratory testing program was performed for this study to evaluate the basic index and geotechnical engineering properties of the site soils. Both disturbed and relatively undisturbed samples were tested. The tests performed and the procedures followed are outlined below.

Soil Classification

Field Observation and Laboratory Analysis. Soil samples from the explorations were visually classified in the field and then taken to our laboratory where the classifications were verified in a relatively controlled laboratory environment. Field and laboratory observations include density/consistency, moisture condition, and grain size and plasticity estimates.

The classifications of selected samples were checked by laboratory tests such as Atterberg limits determinations and grain size analyses. Classifications were made in general accordance with the Unified Soil Classification (USC) System, ASTM D 2487, as presented on Figure B-1.

Water Content Determinations

Water content was determined for most samples recovered in the explorations in general accordance with ASTM D 2216, as soon as possible following their arrival in our laboratory. Water content was not determined for very small samples or samples where large gravel contents would result in values considered unrepresentative. The results of these tests are plotted at the respective sample depth on the exploration logs. In addition, water contents are routinely determined for samples subjected to other testing. These are also presented on the exploration logs.

Grain Size Analysis

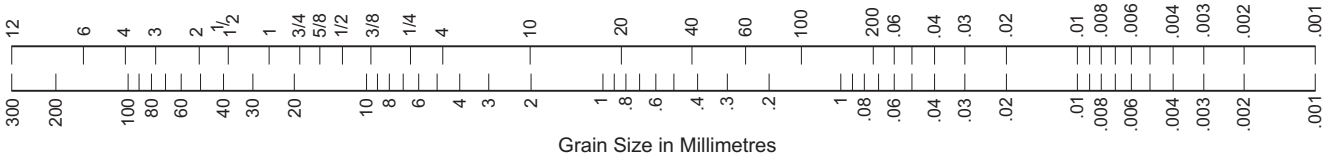
Grain size distribution was analyzed on representative samples in general accordance with ASTM D 422. Wet sieve analysis was used to determine the size distribution greater than the U.S. No. 200 mesh sieve. The size distribution for particles smaller than the No. 200 mesh sieve was determined by the hydrometer method for a selected number of samples. The results of the tests are presented as curves on Figures B-2 plotting percent finer by weight versus grain size.

Atterberg Limits

We determined Atterberg limits for selected fine-grained soil samples. The liquid limit and plastic limit were determined in general accordance with ASTM D 4318-84. The results of the Atterberg limits analyses and the plasticity characteristics are summarized in the Liquid and Plastic Limits Test Report, Figures B-3. This relates the plasticity index (liquid limit minus the plastic limit) to the liquid limit. The results of the Atterberg limits tests are shown graphically on the boring logs as well as where applicable on figures presenting various other test results.

Unified Soil Classification (USC) System
Soil Grain Size

Size of Opening In Inches	Number of Mesh per Inch (US Standard)	Grain Size in Millimetres



COBBLES	GRAVEL	SAND	SILT and CLAY
Coarse-Grained Soils			Fine-Grained Soils

Coarse-Grained Soils

G W	G P	G M	G C	S W	S P	S M	S C
Clean GRAVEL <5% fines	*	GRAVEL with >12% fines		Clean SAND <5% fines	*	SAND with >12% fines	
GRAVEL >50% coarse fraction larger than No. 4				SAND >50% coarse fraction smaller than No. 4			
Coarse-Grained Soils >50% larger than No. 200 sieve							

G W and S W $\left(\frac{D_{60}}{D_{10}}\right)$ >4 for G W, >6 for S W & $1 \leq \left(\frac{(D_{30})^2}{D_{10} \times D_{60}}\right) \leq 3$

G P and S P Clean GRAVEL or SAND not meeting requirements for G W and S W

G M and S M Atterberg limits below A line with PI <4

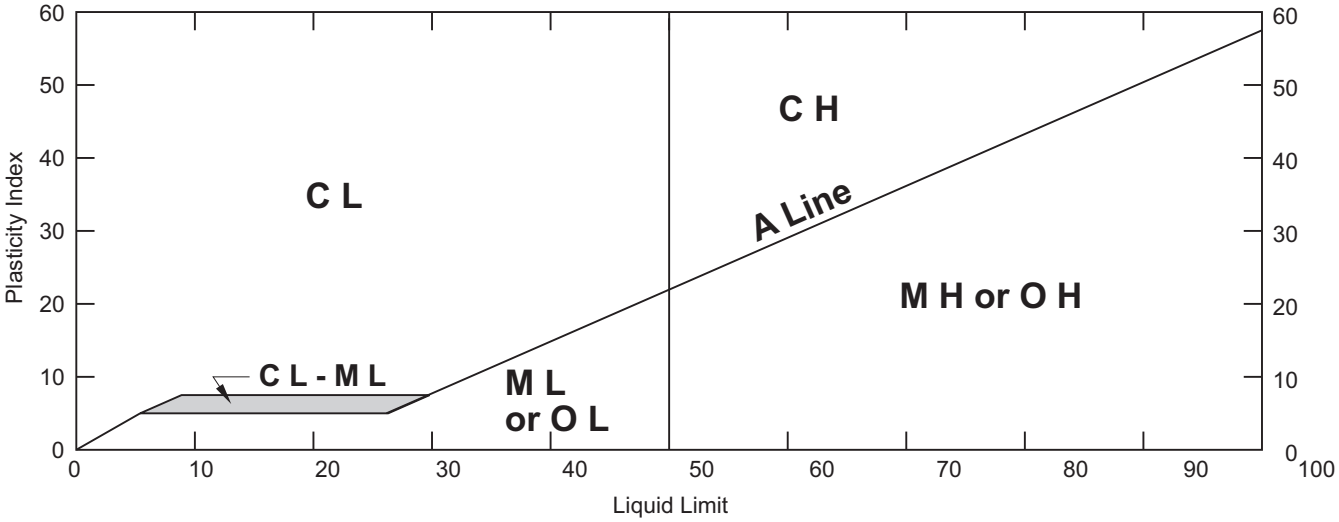
G C and S C Atterberg limits above A Line with PI >7

* Coarse-grained soils with percentage of fines between 5 and 12 are considered borderline cases requiring use of dual symbols.

D_{10}, D_{30}, and D_{60} are the particles diameter of which 10, 30, and 60 percent, respectively, of the soil weight are finer.

Fine-Grained Soils

M L	C L	O L	M H	C H	O H	Pt
SILT	CLAY	Organic	SILT	CLAY	Organic	Highly Organic Soils
Soils with Liquid Limit <50%			Soils with Liquid Limit >50%			
Fine-Grained Soils >50% smaller than No. 200 sieve						



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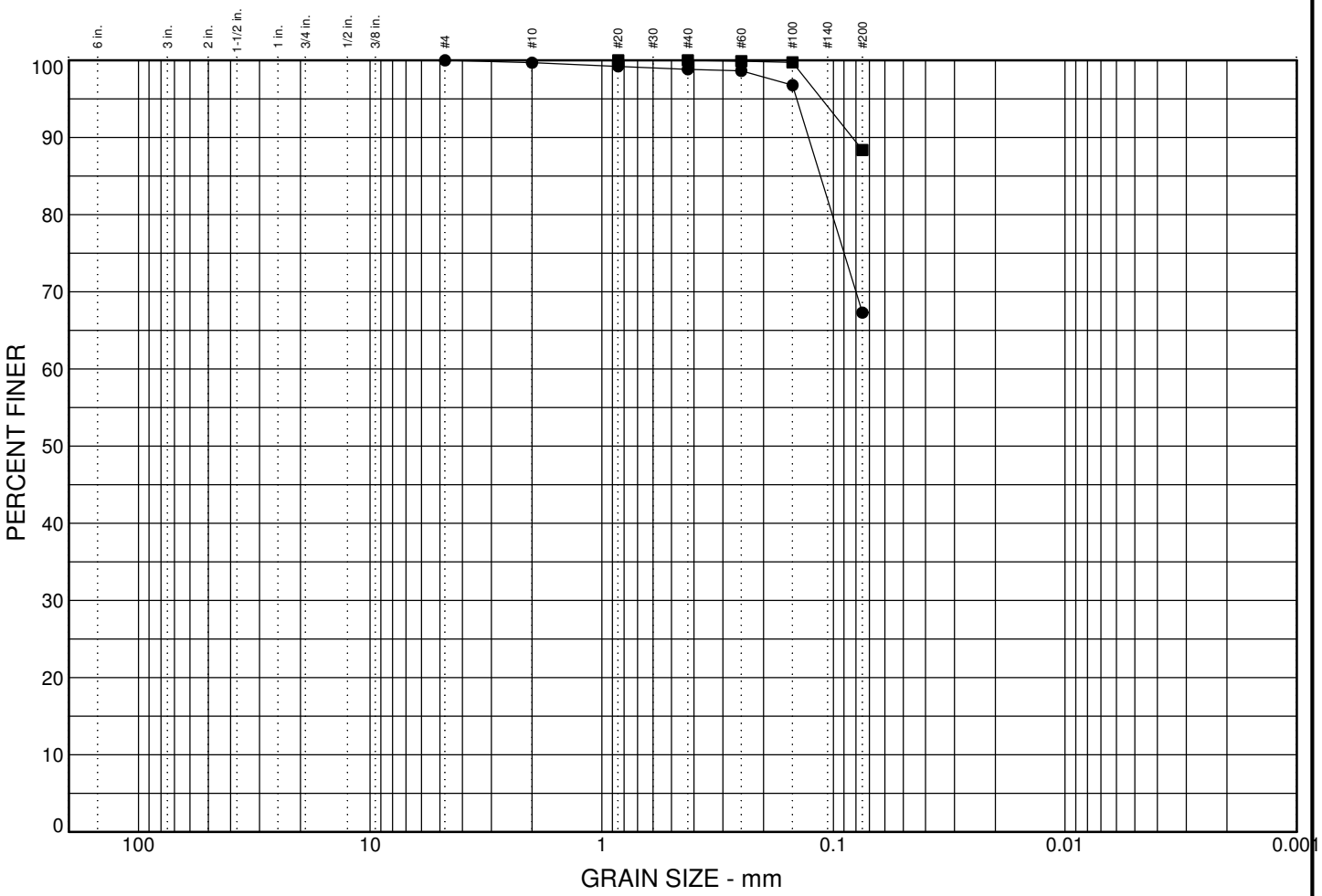
SRF Grain Size (B-1).cdr 3/06

19016-00 4/14

Figure B-1

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	32.7	67.3	
■	0.0	0.0	11.6	88.4	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.114							
■										

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● very sandy SILT	ML	12.1%
■ slightly sandy SILT	ML	27.2%

Remarks:

●

■

Project: 9201 SE Shoreland Place

Client:

● Source: HC-1 Sample No.: S-6 Depth: 20.0 to 21.5

■ Source: HC-1 Sample No.: S-11 Depth: 45.0 to 46.5

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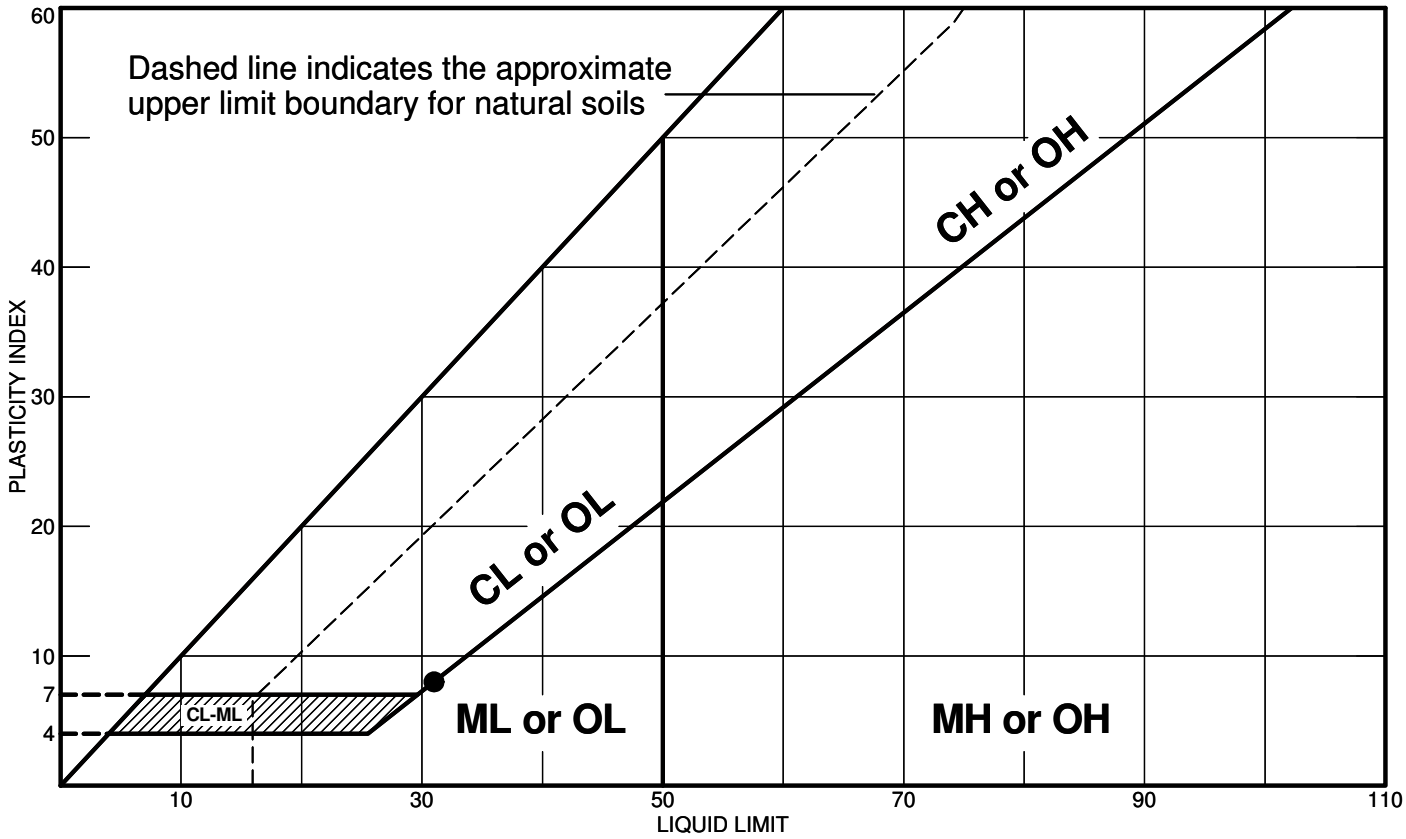
Figure B-2

GRAIN SIZE 1901600-BL.GPJ HC_CORP.GDT 5/30/14

Liquid and Plastic Limits Test Report



Location + Description	LL	PL	PI	-200	USCS
● Source: HC-2 Sample No.: S-5 Depth: 15 SILT	31	23	8		ML

Remarks:

●

Project: 9201 SE Shoreland Place

Client:

Location:

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Figure B- 3

ATTERBERG LIMITS 1901600-BL.GPJ HC_CORP.GDT 5/30/14

APPENDIX C
Past Explorations by Others

APPENDIX C
PAST EXPLORATIONS BY OTHERS

In addition to the explorations and laboratory test results presented in Appendices A and B, respectively, previous soil explorations by others were reviewed to gain an understanding of the subsurface conditions near the project site. Boring logs for these previous explorations are included within this appendix. Logs produced by others are presented for reference only and Hart Crowser is not responsible for the accuracy or completeness of the information presented in the logs. Approximate locations of these borings are as shown in the site plan figure included in this appendix, actual locations may differ from those shown.

19239

GEOTECHNICAL ENGINEERING STUDY
PROPOSED RESIDENCE
9205 S.E. SHORELAND DRIVE
LOT 10, MEYDENBAUER POINT
BELLEVUE, WASHINGTON
G-0978-R

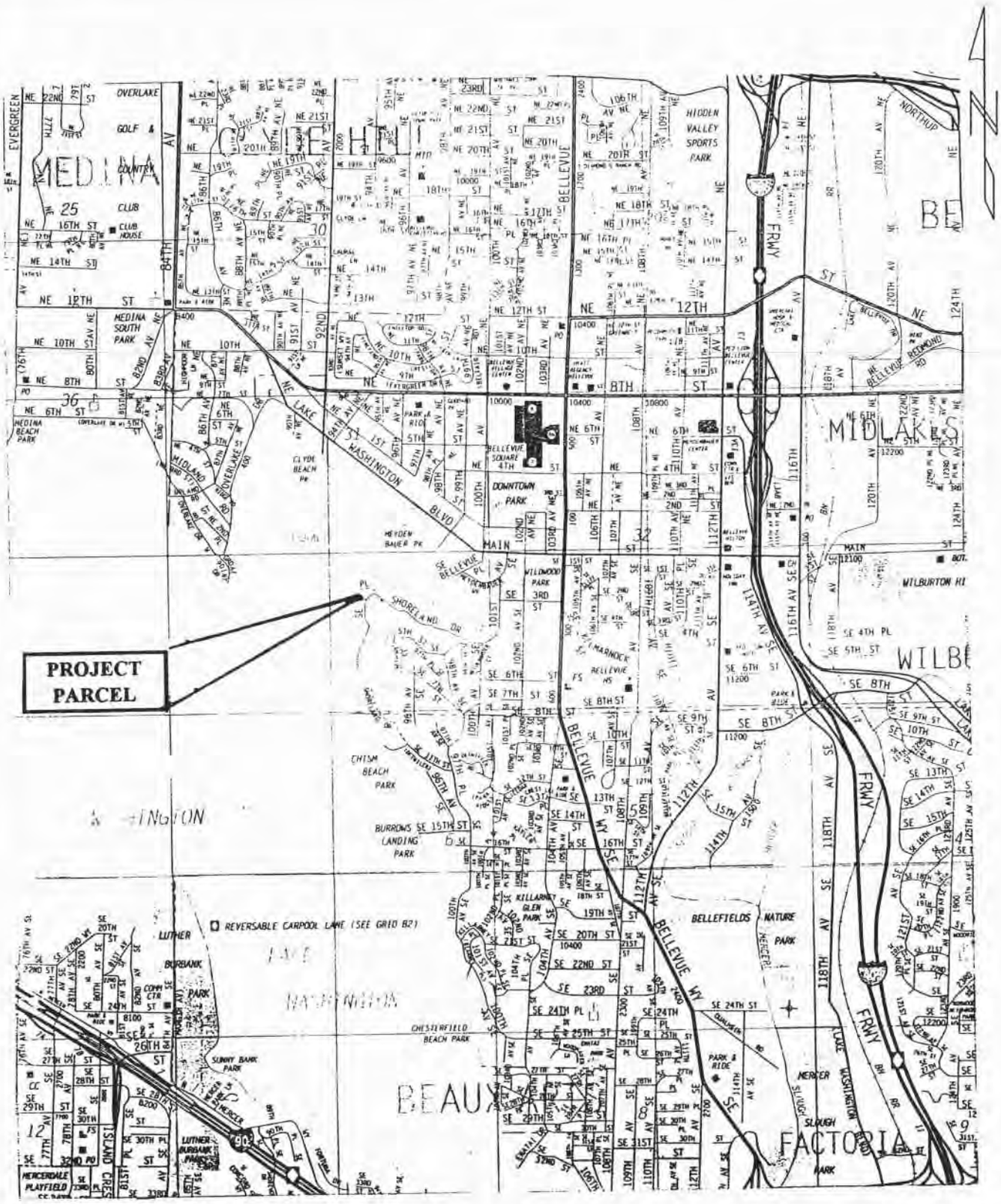
WB 99 00067
21923

Prepared for

Mr. Gary T. Williams
Capital Properties
7683 S.E. 27th, Suite 153
Mercer Island, WA 98040

January 21, 1999

Geo Group Northwest, Inc.
13240 NE 20th Street, Suite 12
Bellevue, WA 98005
Phone: (425) 649-8757



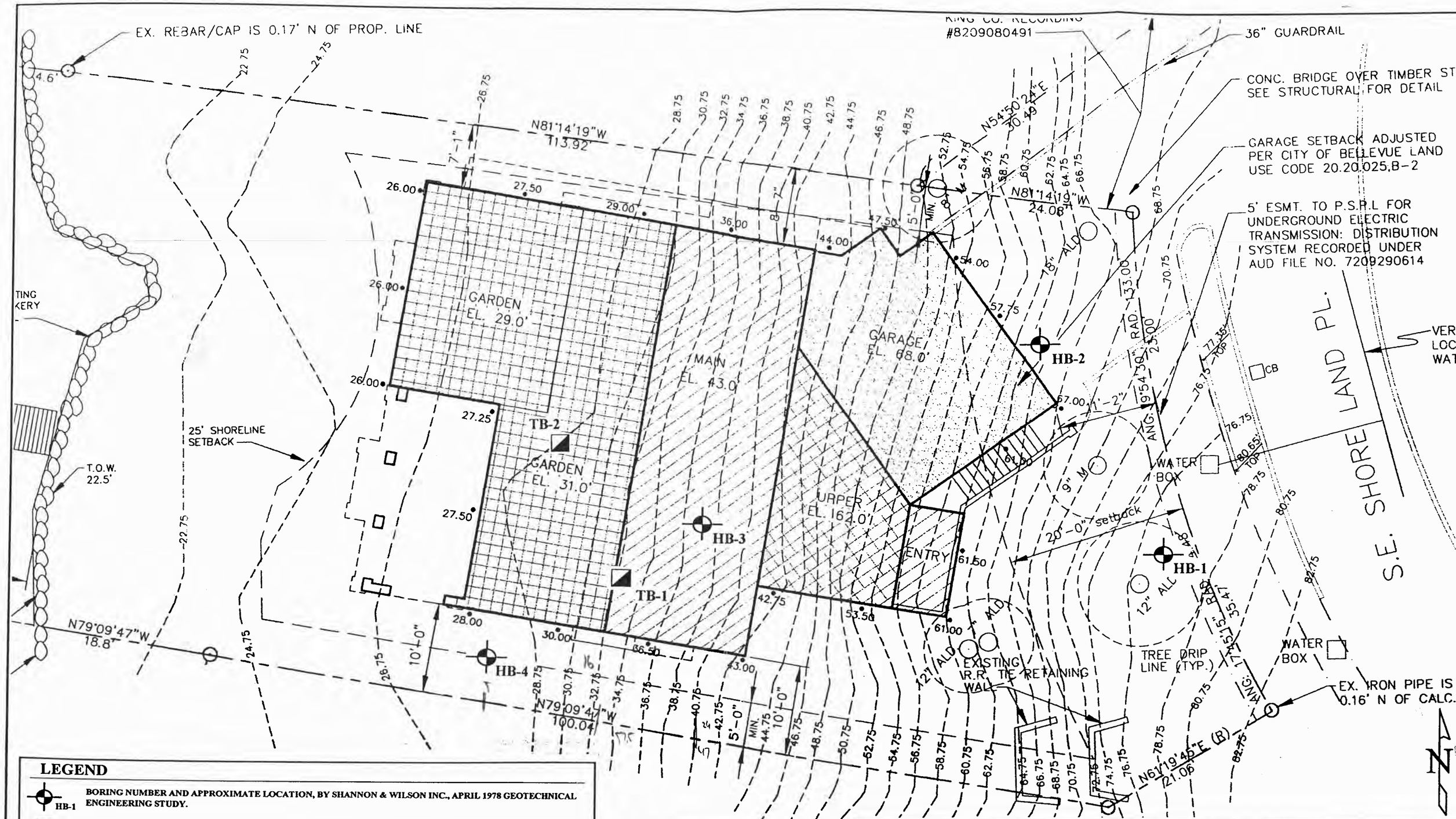
GEO Group Northwest, Inc.

Geotechnical Engineers, Geologists, & Environmental Scientists

VICINITY MAP

PROPOSED RESIDENCE
LOT 10, MEYDENBAUER POINT
BELLEVUE, WASHINGTON

SCALE:	DATE:	MADE:	CHKD:	JOB NO:	PLATE
1" = 2400'	9/1/98	AG	WC	G-0978	1



LEGEND

HB-1 — BORING NUMBER AND APPROXIMATE LOCATION, BY SHANNON & WILSON INC., APRIL 1978 GEOTECHNICAL ENGINEERING STUDY.

TB-1 — TEST BORING NUMBER AND APPROXIMATE LOCATION, BY GEO GROUP NORTHWEST, INC.

Site plan adapted from Site Plan prepared by Andrew Valente Design, dated 1/15/99. Hand boring locations adapted from Shannon & Wilson report, dated April 1978.

0 10 20

REDUCED SCALE: 1 inch = 10 feet

Group Northwest, Inc.
Geotechnical Engineers, Geologists, & Environmental Scientists

SITE PLAN
PROPOSED RESIDENCE
LOT 10, MEYDENBAUER POINT
BELLEVUE, WASHINGTON

SCALE	DATE	MADE	CHKD	JOB NO.	PLATE
1" = 10'	1/20/99	WL	WC	G-0978R	2

BORING NO. HB-1

Logged By: Shannon & Wilson, Inc. **Date Drilled:** 4/17/78 **Surface Elev:** 80 feet +/-

Depth ft.	USCS	Soil Description	SAMPLE Type	SAMPLE No.	Calc. SPT N Blows per 1-ft	Porter Penetration resistance blows per 1 foot
		Loose, brown, silty, gravelly, fine to medium SAND with scattered organics (FILL?)			1	4
2.5		Loose, light brown, silty, fine to medium SAND with scattered coarse sand.			2	12
					3	20
5.0		Medium dense, gray-brown, interlayered clayey, silty, fine to medium SAND; sandy, silty CLAY; and CLAY			7	38
					9	50
7.5		Medium dense, brown, clayey, silty, fine to medium SAND			11	63
		Bottom of Boring 8.5 ft. Completed 4/17/78				
10						
15						
20						

LEGEND:
- 2" O.D. Split-Spoon Sampler
- 3" O.D. Shelby-Tube Sampler
- 3" O.D. California Sampler

Boring Log adapted from Shannon & Wilson, Inc., report titled: "Proposed Residence Meydenbauer Point, Bellevue" April 21, 1978

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

BORING LOG

PROPOSED RESIDENCE
LOT 10 MEYDENBAUER POINT
BELLEVUE, WASHINGTON

DATE: 9/22/98	JOB NO: G-0978	PLATE: 5

BORING NO. HB-2

Logged By: Shannon & Wilson, Inc. **Date Drilled:** 4/17/78 **Surface Elev:** 70 feet +/-

Depth ft.	USCS	Soil Description	SAMPLE Type	SAMPLE No.	Calc. SPT N Blows per 1-ft	Porter Penetration resistance blows per 1 foot
		Loose, brown, organics			2	11
		Loose to medium dense, lt. Brown to gray brown, silty, fine SAND, scattered gravel and organics near surface. Occasional zones of light brown to gray brown silt.			4	23
					8	46
5.0					18	104
10		Medium dense, gray-brown, SILT with occassional zones of silty, fine sand, wet at 15.5 ft.			19	109
					18	103
15					19	109
					23	133
20					19	110
					26	150
25		Bottom of Boring Completed 4/17/78 Groundwater measured at 15 ft.				
30						
35						
40						

LEGEND:
- 2" O.D. Split-Spoon Sampler
- 3" O.D. Shelby-Tube Sampler
- 3" O.D. California Sampler

Boring Log adapted from Shannon & Wilson, Inc., report titled: "Proposed Residence Meydenbauer Point, Bellevue" April 21, 1978

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

BORING LOG

PROPOSED RESIDENCE
LOT 10 MEYDENBAUER POINT
BELLEVUE, WASHINGTON

DATE: 9/22/98	JOB NO: G-0978	PLATE: 6

BORING NO. HB-3

Logged By: Shannon & Wilson, Inc. **Date Drilled:** 4/18/78 **Surface Elev:** 41 feet +/-

Depth ft.	USCS	Soil Description	SAMPLE Type	SAMPLE No.	Calc. SPT N Blows per 1-ft	Porter Penetration resistance blows per 1 foot
		Loose, brown, clayey, silty, SAND with numerous organics			1	8
		Loose, brown, silty fine to medium SAND with scattered coarse sand.				
2.5		Loose, brown to gray-brown, SILT			2	10
					2	9
5.0						
		Loose, mottled gray brown and gray SILT with scattered organics			3	20
7.5						
		Medium dense, gray, clayey SILT, wet				
		Medium dense, gray, weith extensive iron staining, silty fine SAND, wet			10	58
10		Medium dense, gray SILT, wet				
					15	87
					21	120
		Bottom of Boring 13.2 ft. Completed 4/18/78				
15		Groundwater measured at 7.5 ft.				
20						

LEGEND:
- 2" O.D. Split-Spoon Sampler
- 3" O.D. Shelby-Tube Sampler
- 3" O.D. California Sampler

Boring Log adapted from Shannon & Wilson, Inc., report titled: "Proposed Residence Meydenbauer Point, Bellevue" April 21, 1978

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

BORING LOG

PROPOSED RESIDENCE
LOT 10 MEYDENBAUER POINT
BELLEVUE, WASHINGTON

DATE: 9/22/98	JOB NO: G-0978	PLATE: 7

BORING NO. HB-4

Logged By: Shannon & Wilson, Inc. **Date Drilled:** 4/18/78 **Surface Elev:** 31 feet +/-

Depth ft.		USCS	Soil Description	SAMPLE Type	SAMPLE No.	Calc. SPT N Blows per 1-ft	Porter Penetration resistance blows per 1 foot
			Soft, dark brown, clayey SILT (wet at 8 in.)			3	17
2.5			Loose to medium dense, gray SILT, wet			8	49
5.0						15	88
7.5						21	120
			Bottom of Boring 8.5 ft. Completed 4/18/78 Groundwater measured at 8 in.				
10							
15							
20							

LEGEND:
- 2" O.D. Split-Spoon Sampler
- 3" O.D. Shelby-Tube Sampler
- 3" O.D. California Sampler

Boring Log adapted from Shannon & Wilson, Inc., report titled: "Proposed Residence Meydenbauer Point, Bellevue" April 21, 1978

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

BORING LOG

PROPOSED RESIDENCE
LOT 10 MEYDENBAUER POINT
BELLEVUE, WASHINGTON

DATE: 9/22/98	JOB NO: G-0978	PLATE: 8

Critical Areas Report

9201 SE Shoreland Place

Bellevue, Washington

Prepared for

Everyoung Services, LLC

May 19, 2015

19016-01

HARTCROWSER

Critical Areas Report

9201 SE Shoreland Place

Bellevue, Washington

Prepared for

Everyoung Services, LLC

May 19, 2015
19016-01

Prepared by

Hart Crowser, Inc.

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APPENDIX A

Topograhic and Boundary Survey

APPENDIX B

Photographs

Critical Areas Report

9201 SE Shoreland Place

Bellevue, Washington

1.0 INTRODUCTION

This critical areas report (CAR) was prepared to support the City of Bellevue review of the proposed modifications to 9201 SE Shoreland Place Single Family Residence (SFR) in Bellevue, Washington (Project). A Vicinity Map showing the site location is presented on Figure 1.

The City of Bellevue Planning & Community Development requested a CAR based on discussions with Michael Payne on October 17, 2013, and a visit to the Permit Counter on May 20, 2014. For the purposes of this CAR, the study area consists of Parcel #5494400110 and adjacent parcels (Figure 2). Hart Crowser biologists performed a site visit on April 22, 2014 and a literature review to determine the presence of critical areas on and within 200 feet of the study area.

This CAR was prepared in accordance with City of Bellevue criteria, as defined in the Bellevue City Code (BCC) Critical Areas Chapter 20H.250 (City of Bellevue 2014). The following sections of this CAR describe the methods used in the field investigation and our findings:

- Section 2 provides a description of the study area.
- Section 3 provides a description of the proposed Project.
- Section 4 identifies the critical areas on-site and standards proposed for modification.
- Section 5 includes a habitat and cumulative impacts assessment.
- Section 6 includes a functional lift analysis.
- Section 7 includes mitigation and restoration concepts.
- Section 8 includes critical areas report decision criteria.
- Appendix A provides topographic and boundary survey.
- Appendix B provides site photographs.

Figure 1 – Vicinity Map

1.1 Review of Existing Information

As part of the analysis to identify and assess critical areas in the study area, we reviewed the following sources of information to support field observations:

- Aerial photographs.
- BCC (Bellevue 2014).

- Hart Crowser 9201 SE Shoreland Place Geotechnical Engineering Design Study (Hart Crowser 2014). Prepared for Everyoung Services, LLC. July 1, 2014.
- Nwmaps.net (ecitygov.net 2014).
- US Fish and Wildlife Service (USFWS) Wetlands Mapper for National Wetlands Inventory (NWI) map information (USFWS 2014).
- Washington Department of Fish and Wildlife (WDFW) Priority Habitat and Species (PHS) on the web (2014).
- Washington State Department of Natural Resources (WDNR) Natural Heritage Program database (WDNR 2014).

2.0 SITE DESCRIPTION

The site is located on a steep slope on the eastern shore of Lake Washington in the Meydenbauer neighborhood of Bellevue, Washington. The site is currently occupied by a two- to three-story, wood-frame, single-family house (zoned R-4) supported by three rows of wooden post foundations on steeply sloping ground (Appendix B, Photograph 1). There is also a bathhouse with a deck and a dock with a covered area (Appendix B, Photographs 2 and 3). Figure 2 shows the footprint of the existing structures as well as the ground surface topographic contours. A topographic and boundary survey is included in Appendix A. Note that elevations in this CAR are referenced to the North American Vertical Datum of 1988 (NAVD88) and were estimated from the site survey by GeoDimensions dated March 1, 2013.

2.1 Topography

Site grades in the Project area slope steeply to the west toward Lake Washington. Site elevations vary from about elevation 88 feet at the intersection of the driveway and SE Shoreland Place to about 20 feet at the west edge of the site along the shoreline. The steep slope portion of the site is about 50 feet tall with an average slope of about 70 percent (or 35 degrees) and covers an area of about 4,300 square feet (sf).

2.2 Soils

In general, subsurface conditions consist of a limited amount of possible fill and medium stiff to stiff sandy silt over very stiff to hard sandy silt and dense silty sand over very stiff to hard clay (Hart Crowser 2014). The native silty sand and sandy silt are interpreted to be advance outwash, based on available geologic maps, but could also include transitional glaciolacustrine deposits; the underlying clay is interpreted to be glaciolacustrine. The native soils are glacially overridden. For more information on soils please reference Hart Crowser's 9201 SE Shoreland Place Geotechnical Engineering Design Study (2014).

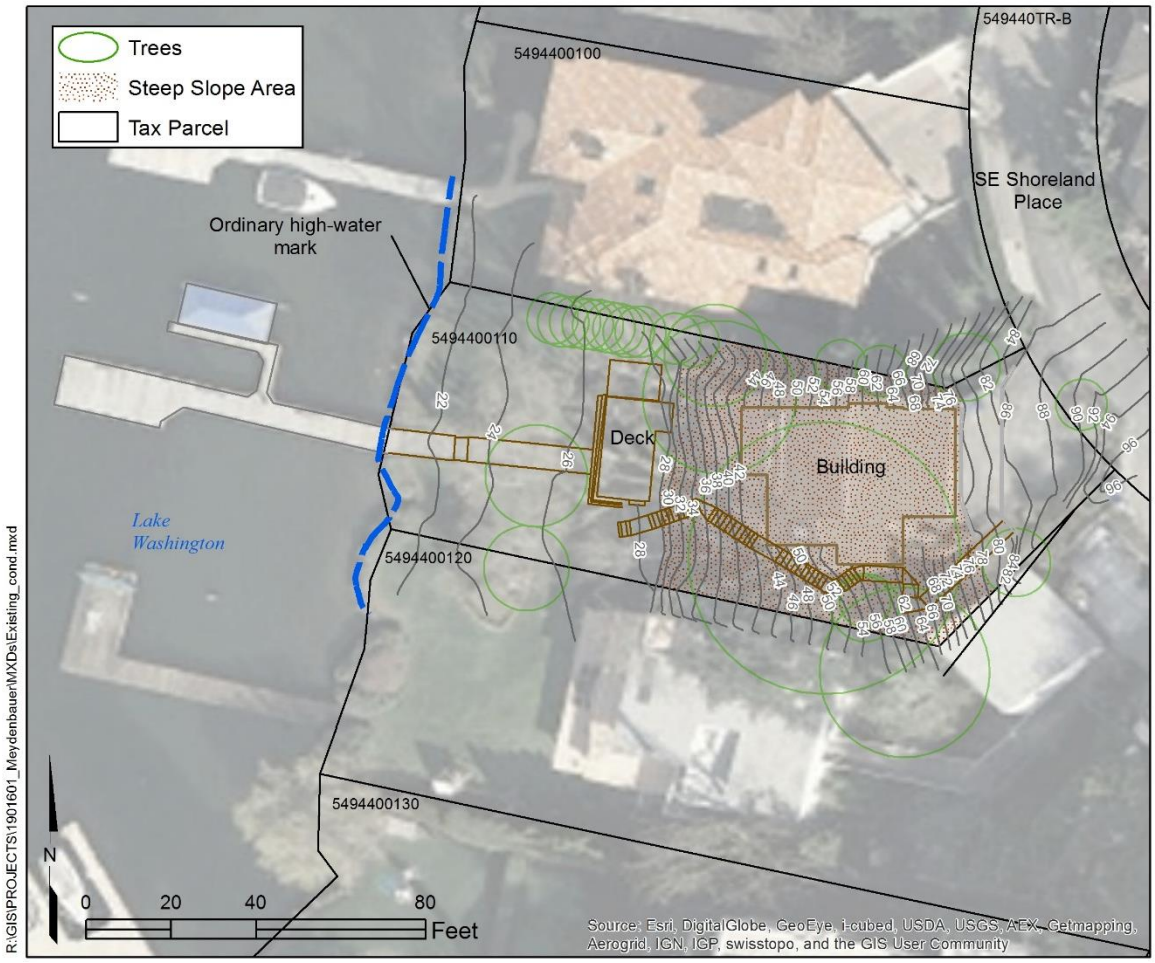

Figure 2 – Existing Site Conditions

2.3 Hydrology

The study area is located in the Cedar-Lake Washington watershed, Water Resource Inventory Area (WRIA) 8. Hydrologic characteristics in the study area are influenced by regional groundwater, direct precipitation, surface water runoff, and Lake Washington, which borders the western edge of the study area. No evidence of surface water or groundwater seepage was apparent during our site visit on April 22, 2014.

The ordinary high water mark (OHWM) of Lake Washington was not delineated due to the lack of vegetation (i.e., rip rap is present) along the shoreline (Appendix B, Photograph D). Therefore, for the purpose of this CAR, the OHWM was set at an elevation of 18.6 feet as shown in Appendix A.

3.0 PROJECT DESCRIPTION

Current development plans call for renovating the existing SFR with an addition that includes two levels below part of the existing structure and extending over the slope on the downhill side (west side) of the house. The footprints of both the existing structure (1,516 sf) and proposed addition

(426 sf) are shown on Figure 3. The renovation will include grading and installing new retaining walls and foundations to allow the house to be built on the terraced ground (Hart Crowser 2014).

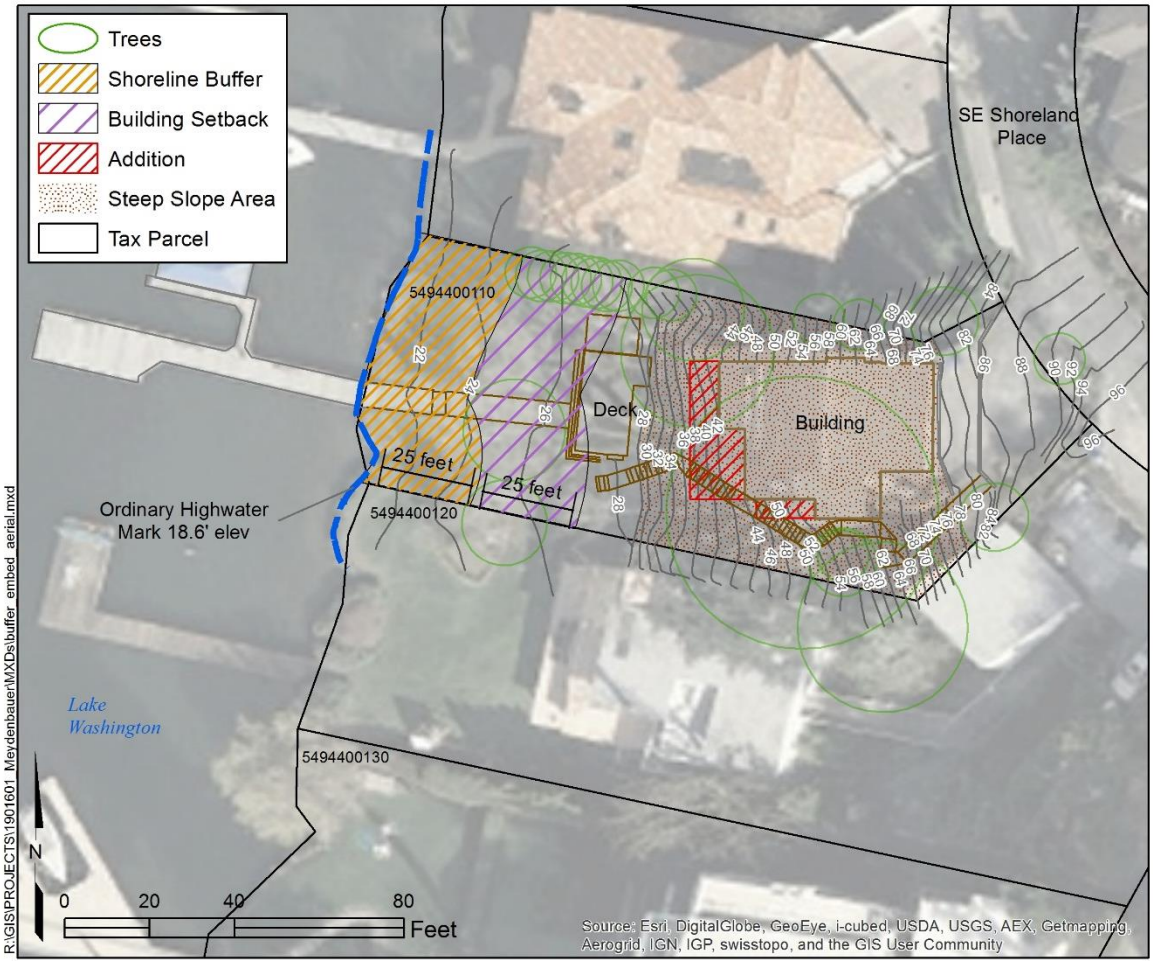

Figure 3 – Proposed Addition and Shoreline Buffer/Setback

4.0 CRITICAL AREAS ON-SITE AND STANDARDS PROPOSED FOR MODIFICATION

The following critical areas as defined in the City Land Use Code (LUC) 20.25H.025 are present on the site and on the properties adjacent to the site:

- Shorelines; and
- Geologic hazard areas.

4.1 Shorelines

Lake Washington borders the western edge of the site and adjacent properties. Lake Washington is designated as a shoreline critical area (LUC 20.25E.017.D.1). The shoreline critical area buffer for a

developed site is 25 feet with a 25-foot building setback (LUC 20.25H.035.A) (Figure 3). The shoreline critical area buffer will be enhanced as mitigation for the impact to the geologic hazard area.

4.2 Geologic Hazard Areas

There is an area of 4,300 sf on the site that is classified as a geologic hazard area (LUC 20.25H.120) (Hart Crowser 2014). The geologic hazard area is a steep slope critical area because it is taller than 10 feet, steeper than 40 percent, and covers an area greater than 1,000 sf. This steep slope is also identified in NWMaps.net (ecitygov.net 2014). The SFR modifications will occur within this steep slope critical area. Approximately 1,300 sf of existing bare ground on the steep slope will be excavated or filled by the proposed SFR modifications.

We propose to modify the steep slope through this CAR (LUC 20.25H.230) and the geotechnical report (Hart Crowser 2014) under LUC 20.25H.140, 20.25H.145, and the comprehensive plan policy EN-54. The geotechnical report (Hart Crowser 2014) includes provisions for landslide hazards and steep slopes described in LUC 20.25H.140. These provisions include:

- Site and construction plans (Hart Crowser 2014, Figure 2);
- Assessment of geological characteristics (Hart Crowser 2014, pages 3–8);
- Analysis of the proposal (Hart Crowser 2014, pages 8–28); and
- Minimum critical area setback (page 6 of this report).

See the Geotechnical report (Hart Crowser 2014) for more details on these provisions.

The geotechnical report and this CAR also demonstrate, as required by LUC 20.25H.145, that expansion of the existing single family primary structure:

- Will not increase the threat of the geological hazard to adjacent properties over conditions that would exist if the provisions of this part were not modified (Hart Crowser 2014, page 30);
- Will not adversely impact other critical areas (page 6 of this report);
- Is designed so that the hazard to the Project is eliminated or mitigated to a level equal to or less than would exist if the provisions of this part were not modified (Hart Crowser 2014, page 30);
- Is certified as safe as designed and under anticipated conditions by a qualified engineer or geologist, licensed in the state of Washington (Hart Crowser 2014, page 30);
- Will demonstrate that modification of the critical area or critical area buffer will have no adverse impacts on stability of any adjacent slopes, and will not impact stability of any existing structures (Hart Crowser 2014, page 30);
- Will comply with recommendations of the geotechnical report with respect to best management practices (BMP), construction techniques or other recommendations (page 8 of this report); and

- Will have no significant impact on habitat associated with species of local importance or such habitat that could reasonably be expected to exist during the anticipated life of the development proposal if the area were regulated under LUC 20.25H (pages 7 and 8 of this report).

Comprehensive plan policy EN-54 states the Director may utilize specific criteria in decisions to exempt specific small, isolated, or artificially created steep slopes from critical areas designation.

EN-54 supports the modification of the steep slope on-site as it is small and isolated from other steep slopes with significant habitat value.

5.0 HABITAT AND CUMULATIVE IMPACTS ASSESSMENT

Upland habitat on and adjacent to the Project is developed single family residential lots with a mix of native and non-native plant species. Vegetation on site includes tree, shrub, grass, and herbaceous species commonly associated with residential lots (Appendix B, Photograph 5). Vegetation is a mix of native and non-native species. Native plant species include big leaf maple (*Acer macrophylum*), western white pine (*Pinus monticola*), black cottonwood (*Populus balsamifera*), western redcedar (*Thuja plicata*), horsetail (*Equisetum spp.*) and sword fern (*Polystichum minutum*). Non-native species include a variety of ornamental shrubs, English ivy (*Hedera helix*), Himalayan blackberry (*Rubus armeniacus*), laurel (*Laurus* species), and arborvitae (*Thuja* species).

The shoreline critical area buffer is mostly non-native mowed grass (Appendix B, Photograph 1) with a few shrubs. The shoreline is composed of riprap and lacks overhanging vegetation. The substrate waterward of the rip rap is sandy with some cobbles and likely contains macroalgae in the photic zone during the summer which is common in Lake Washington.

The site does not meet the needs of any terrestrial species identified in LUC 20.25H.150. The WDFW PHS web mapper does not show any priority habitats or species on site with the exception of Lake Washington. Lake Washington does meet the needs of two fish species identified in LUC 20.25H.150 because juveniles of these fish species are known to migrate and forage in the nearshore of Lake Washington (WDFW 2014a). These fish species include Chinook salmon (*Oncorhynchus tshawytscha*) and coho salmon (*Oncorhynchus kisutch*). Management recommendations for these fish species include maintaining riparian buffers with native overhanging vegetation for shading and water quality protection and mitigating for shading impacts of docks through installation of light penetrating surfaces (LPS) (Knight 2009). No direct impacts within the shoreline critical area will result from the proposed Project.

Direct impacts from the proposed Project include permanent impacts to steep slopes (SFR additions). Existing steep slope habitat that will be impacted is bare ground under the existing house (Appendix B, Photograph 6) which is of very limited habitat value with the exception of a 60 inch diameter big leaf maple. The impacts to the big leaf maple are unavoidable.

Table 1 – Impact Summary

Critical Area	Total sf on-site	Impact (sf)	Vegetation Enhancement (sf)
Steep Slope	4,000	1,300 permanent (including one 60 inch diameter big leaf maple) ~~555~~ 846 temporary	0 846
Shoreline Buffer	1,500	0	1.300

Indirect impacts include potential water quality impacts from project construction (i.e., stormwater runoff). However, potential construction impacts would be temporary and minimized by construction BMPs.

Measures to avoid, minimize, and mitigate for Project impacts include:

- Care will be taken in all work such that no construction-related activity shall contribute to the degradation of the environment or allow foreign material to enter Lake Washington.
- Best Management Practices will be employed to reduce the potential for construction-related impacts on species and their habitats. This includes a temporary erosion and sediment control plan with BMP details.
- Oil spill response equipment will be on site and a spill response plan will be in hand and deployed as necessary by the contractor.

Existing degraded shoreline buffer will be enhanced to mitigate for impacts to the steep slope critical areas (see Mitigation and Restoration Concepts Section). After construction, a maintenance and monitoring program will be implemented for the shoreline critical area mitigation. This program will entail maintaining native vegetation and determining if mitigation performance standards are being met.

6.0 FUNCTIONAL LIFT ANALYSIS

We propose to mitigate for project impacts and provide a functional lift to existing habitat by enhancing 1,300 sf of the shoreline critical area buffer with native trees, shrubs, and groundcover. In addition we will revegetate ~~555~~ 846 sf of temporarily disturbed steep slope areas that are currently bare ground, non-native plant species, or disturbed by removal of the existing stairs (i.e. 291 sf). Enhancing the 1,300 sf of shoreline buffer will mitigate for the 1,300 sf of steep slope critical area impacts. Three additional trees will be added to the shoreline to mitigate for the impacts to the 60 inch diameter big leaf maple. The shoreline is often the most important sensitive feature and will provide the greatest lift in ecological function and value.

The mitigation measures outlined in Table 2 will result in a functional lift of the habitat on the property.

Table 2 – Functional Lift Analysis for Steep Slope Critical Area

Buffer Functions	Existing conditions	Standard Code Application	Proposed Modification	Functional Improvement with Proposed Modification
Stabilization of slope	Site currently stabilized by retaining walls and piles. Habitat degraded.	No change to steep slope or buffer.	Improve stability of slope and SFR but permanently impact 1,300 sf of steep slope. Revegetate ~~555~~ 846 sf of temporarily disturbed steep slope areas with native vegetation. 1,300 sf of shoreline critical area buffer enhanced with native tree and shrub species. Additional three trees included in shoreline buffer mitigation to offset impacts to 60 inch diameter big leaf maple.	Yes. Water quality improved by shoreline shading and pollutant/ stormwater filtration in the shoreline critical area buffer.
Creation of wildlife habitat on steep slope and in shoreline buffer	Degraded habitat on steep slope under house. Degraded habitat in shoreline buffer.	No change to existing habitat.	Enhance steep slope and shoreline buffer with native tree and shrub species to create wildlife and shoreline habitat in 7 to 10 years.	Diverse habitat for wildlife. Increase fish food from terrestrial invertebrate inputs through shoreline buffer enhancement.

7.0 CONCEPTUAL MITIGATION AND RESTORATION PLAN

The mitigation plan for the project will be submitted in phases per BCC 20.25H.220 with the conceptual plan presented here and the detailed plan to follow.

7.1 Goals

- Restore 1,300 sf of degraded shoreline critical area buffer.
  - Create a diverse, native plant community including trees, shrubs, and groundcovers.
  - Remove and prevent the re-establishment of invasive species in the planting areas.
- Restore ~~555~~ 846 sf of temporarily disturbed steep slope critical area.

- Create a diverse, native plant community including trees, shrubs, and groundcovers.
- Remove and prevent the re-establishment of invasive species in the planting areas.

7.2 Performance Standards

The following performance standards will be used to gauge the success of the project over time. If all performance standards have been satisfied by the end of Year 5, the project shall be considered complete. The following performance standards apply to the enhancement area and areas of temporary disturbance.

- Survival.
  - Achieve 100 percent survival of all installed plants by the end of Year 1.
  - Survival standards may be achieved through establishment of planted material, recruitment of native volunteers, or replacement plants as necessary.
- Diversity.
  - Establish at least four native tree species and seven native shrub species by the end of Year 5.
- Cover.
  - Achieve 60 percent cover of native trees, shrubs, and groundcovers (not including grasses) by the end of Year 3.
  - Achieve 80 percent cover of native trees, shrubs, and groundcovers (not including grasses) by the end of Year 5.
  - No more than 10 percent cover by invasive species listed by the King County Noxious Weed Control Board as Class A, B, C, or Non-regulated Noxious Weeds in any monitoring year (excluding reed canarygrass).

7.3 Description of Proposed Mitigation

The Project was designed to prevent an increase in the threat from the on-site geologic hazard by improving the stability of the SFR and steep slope. The Project was also designed to not extend into the shoreline critical area buffer and setback area. In addition, 1,300 sf of the shoreline critical area buffer will be enhanced with native vegetation (Figure 4 and Table 3). Any temporary areas (approximately ~~555~~ 846 sf) of disturbance on the steep slope will also be revegetated with native vegetation (Figure 4 and Table 3). Vegetation species were selected from the Bellevue Critical Areas Handbook and included the shoreline view sensitive planting template and steep slope planting template for sunny sites. Emergent plant species are not included because of the high bank associated with the property.

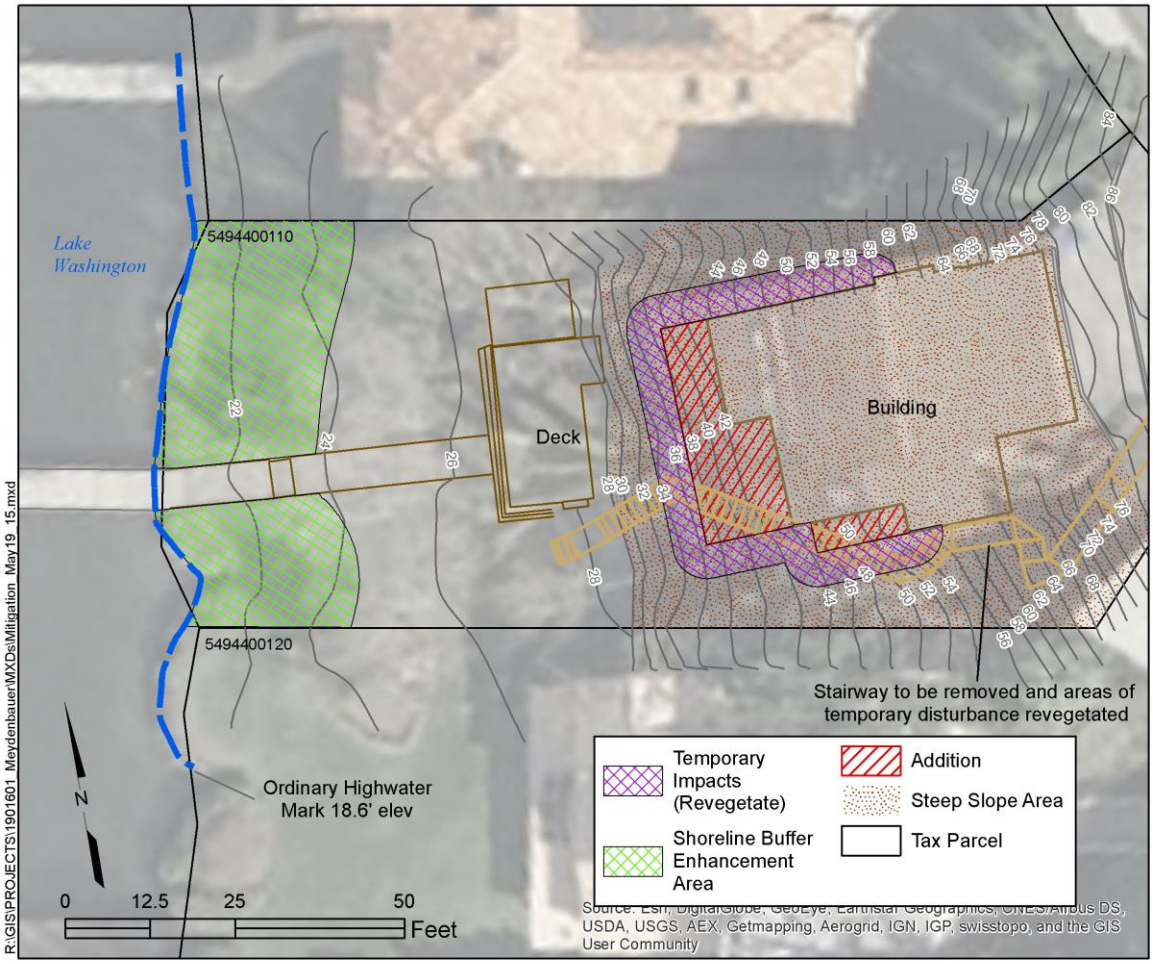

Figure 4 – Shoreline and Steep Slope Critical Area Buffer Enhancement Plan (1,300 sf shoreline enhancement and ~~555~~ 846 sf revegetation of temporarily disturbed areas)

Table 3 – Plant List and Quantities for Shoreline Buffer Enhancement Plantings

Scientific Name	Common Name	Spacing	Quantity (Shoreline buffer enhancement area)	Quantity (steep slope enhancement area)	Size
Fraxinus latifolia/	Oregon Ash	9´ O.C.	~~6~~ 7	0	2 gallon
Pinus contorta	Shore Pine	9´ O.C.	~~6~~ 7	0	2 gallon
Salix scouleriana	Scouler's Willow	9´ O.C.	~~6~~ 7	0	2 gallon
Acer macrophyllum	Big-leaf maple	9´ O.C.	0	~~3~~ 4	2 gallon
Alnus rubra	Red alder	9´ O.C.	0	~~3~~ 4	2 gallon
Pseudotsuga menziesii	Douglas fir	9´ O.C.	0	~~3~~ 4	2 gallon

Scientific Name	Common Name	Spacing	Quantity (Shoreline buffer enhancement area)	Quantity (steep slope enhancement area)	Size
Cornus sericea	Red-osier dogwood	4´ O.C.	16	0	1 gallon
Holodiscus discolor	Oceanspray	4.5´ O.C.	10	~~4~~ 6	1 gallon
Philadelphus lewisii/	Mock orange	4.5´ O.C.	10	~~4~~ 6	1 gallon
Rubus parviflorus	Thimbleberry	3.5´ O.C.	16	~~8~~ 12	1 gallon
Corylus cornuta	Beaked Hazelnut	6' O.C.	0	~~4~~ 6	1 gallon
Symphoricarpos albus	Snowberry	4.5´ O.C.	10	~~6~~ 9	1 gallon
Festuca idahoensis	Idaho fescue	2´ O.C.	16	~~10~~ 15	1 gallon
Arctostaphyllos uva-ursi	Kinnikinnick	2´ O.C.	32	~~14~~ 21	1 gallon

Note:
O.C. – on center

7.4 Timing of Work

Enhancement of the shoreline buffer and revegatation of temporarily disturbed areas will be completed prior to final inspection or issuance of a temporary certificate of occupancy or certificate of occupancy, as applicable, for its development.

7.5 Monitoring Program

The enhanced shoreline buffer and revegetated steep slope areas will be monitored for a period of five years. The monitoring inspection will take place once annually during late summer or early fall. During each late-season monitoring inspection, the following data will be collected:

- Percent survival of all installed plantings (Year 1 only);
- Native and invasive woody cover as determined using the line-intercept method along established transects;
- The species composition, noting whether a species is native or exotic and whether plants were installed or are volunteers;
- The general health and vigor of the installed vegetation;
- Photographs from fixed photopoints established during the as-built inspection; and
- Recommendations for maintenance or repair in the restoration area.

Annual monitoring reports will be submitted to the City of Bellevue by December of each of the five years. Photos from select photo points will be included in the monitoring reports to document the shoreline buffer and steep slope vegetation enhancements.

7.6 Contingency Plan

Should any monitoring report reveal that the mitigation plan has failed in whole or in part, and should that failure be beyond the scope of routine maintenance, the applicant will submit a Contingency Plan to the City of Bellevue for approval. This plan may include replanting, soil amendments or topdressing, substitutions for species selected in the original plan, and adaptive weed control methods.

7.7 Assurance Devices

Assurance devices in compliance with LUC 20.40.490 will be provided to ensure that the approved mitigation, monitoring program, contingency plan, and any conditions of approval are fully implemented.

8.0 CRITICAL AREAS REPORT DECISION CRITERIA

Responses to the critical areas report decision criteria as defined in the LUC 20.25H.255.B are provided in this section.

Decision Criteria – Proposals to Reduce Regulated Critical Area Buffer. The Director may approve, or approve with modifications, a proposal to reduce the regulated critical area buffer on a site where the applicant demonstrates:

- *The proposal includes plans for restoration of degraded critical area and critical area buffer functions which demonstrate a net gain in overall critical area and critical area buffer functions.*

  In Section 5.0 we describe the degraded nature of the existing shoreline buffer. Section 6.0 demonstrate a net gain in overall shoreline buffer functions through the enhancement of 1,300 sf of shoreline buffer with native riparian vegetation.

- *The proposal includes plans for restoration of degraded critical area or critical area buffer functions which demonstrate a net gain in the most important critical area or critical area buffer functions to the ecosystem in which they exist.*

  The shoreline critical area buffer is the most important critical area buffer in the vicinity of the Project. A conceptual restoration plan is presented in Section 7.

- *The proposal includes a net gain in stormwater quality function by the critical area buffer or by elements of the development proposal outside of the reduced regulated critical area buffer.*

  Restoration of 1,300 sf of shoreline critical area buffer will provide a net gain in stormwater water quality function through vegetation enhancements.

- *Adequate resources to ensure completion of any required restoration, mitigation and monitoring efforts.*

  The owner of the property has adequate resources to install and monitor shoreline buffer restoration.

- *The modifications and performance standards included in the proposal are not detrimental to the functions and values of critical area and critical area buffers off-site.*

  The Project will stabilize steep slopes on-site and restore shoreline critical area buffer. Critical areas off-site will not be impacted by the Project. An additional three trees will be planted in the shoreline buffer to offset the impacted 60 inch diameter big leaf maple.

- *The resulting development is compatible with other uses and development in the same land use district. (Ord. 5680, 6-26-06, § 3)*

  The Project is compatible with other uses and development in the same residential land use district which is single-family, residential.

9.0 REFERENCES

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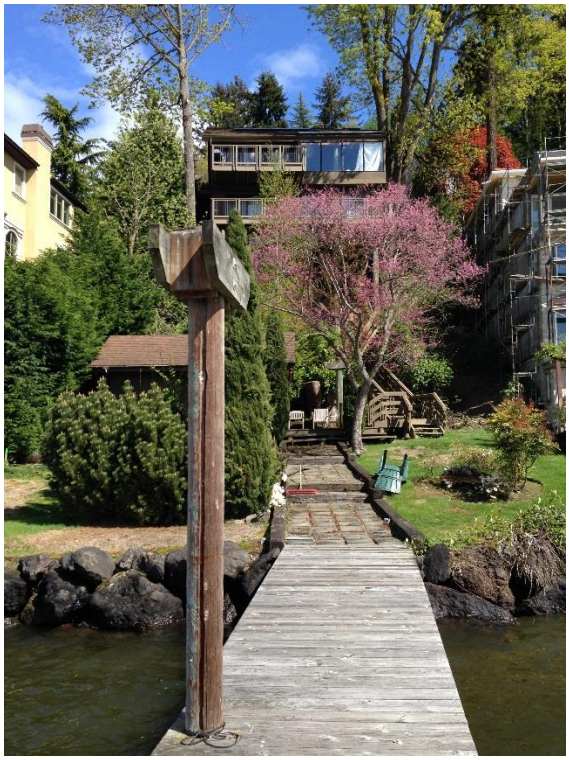
APPENDIX A
Topographic and Boundary Survey

APPENDIX B
Photographs

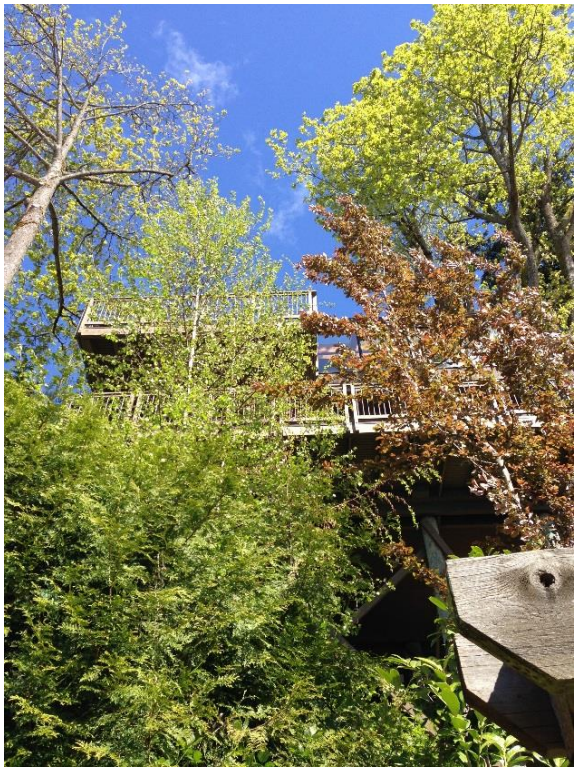


Photograph 1 – View of property facing east

Photograph 2 – Bath house

Photograph 3 – Dock with covered boat moorage

Photograph 4 – Rip rap shoreline



Photograph 5 – Representative vegetation on the parcel

Photograph 6 – Bare ground on steep slope under existing house