CITY OF MUKILTEO
Planning and Community Development Department
Received 9/9/22

Estes Residence

Stormwater Site Plan

<u>PERMIT NUMBERS</u>
HUP-HE-2020-001
VAR-2021-001
ENG-2020-009
SFR-2020-005

April 12, 2021
Revised: June 10, 2022

Prepared for
Chris Estes
6116 Chennault Beach Dr
Mukilteo, WA 98275

BRANDON MICHAEL LOUCKS
STATE OF WASHINGTON
50085
REGISTERED
PROFESSIONAL ENGINEER
06/10/2022

Submitted by

ESM Consulting Engineers, LLC
33400 8th Avenue S, Suite 205
Federal Way, WA 98003

253.838.6113 tel
253.838.7104 fax

ESM

www.esmcivil.com

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SECTION 1 – PROJECT OVERVIEW

The proposed Estes Residence is located on approximately 0.31 acres within the City of Mukilteo on Webster Way. More particularly, the project is within the Northeast Quarter of Section 20, Township 28 Northwest, Range 4 East, W.M. and the parcel number is 00408600400300. The proposal of the project is to construct a new single-family residence associated utilities and landscaping. Refer to Figure 1.1 for a vicinity map.

The site is currently undeveloped with trees and an understory of vegetation. The site has about 38 feet of grade change and steep slopes exceeding 100 percent. The limit of disturbance is in the west portion of the site where there are moderately steep slopes that are less than 40 percent.

The proposed development area will disturb approximately 5,600 square feet and 3,984 square feet of the disturbed area is new impervious surfaces, which consists primarily of the rooftop and driveway area. The new residence will be served by utilities extended from Webster Way and access to the home will be provided from Webster Way.

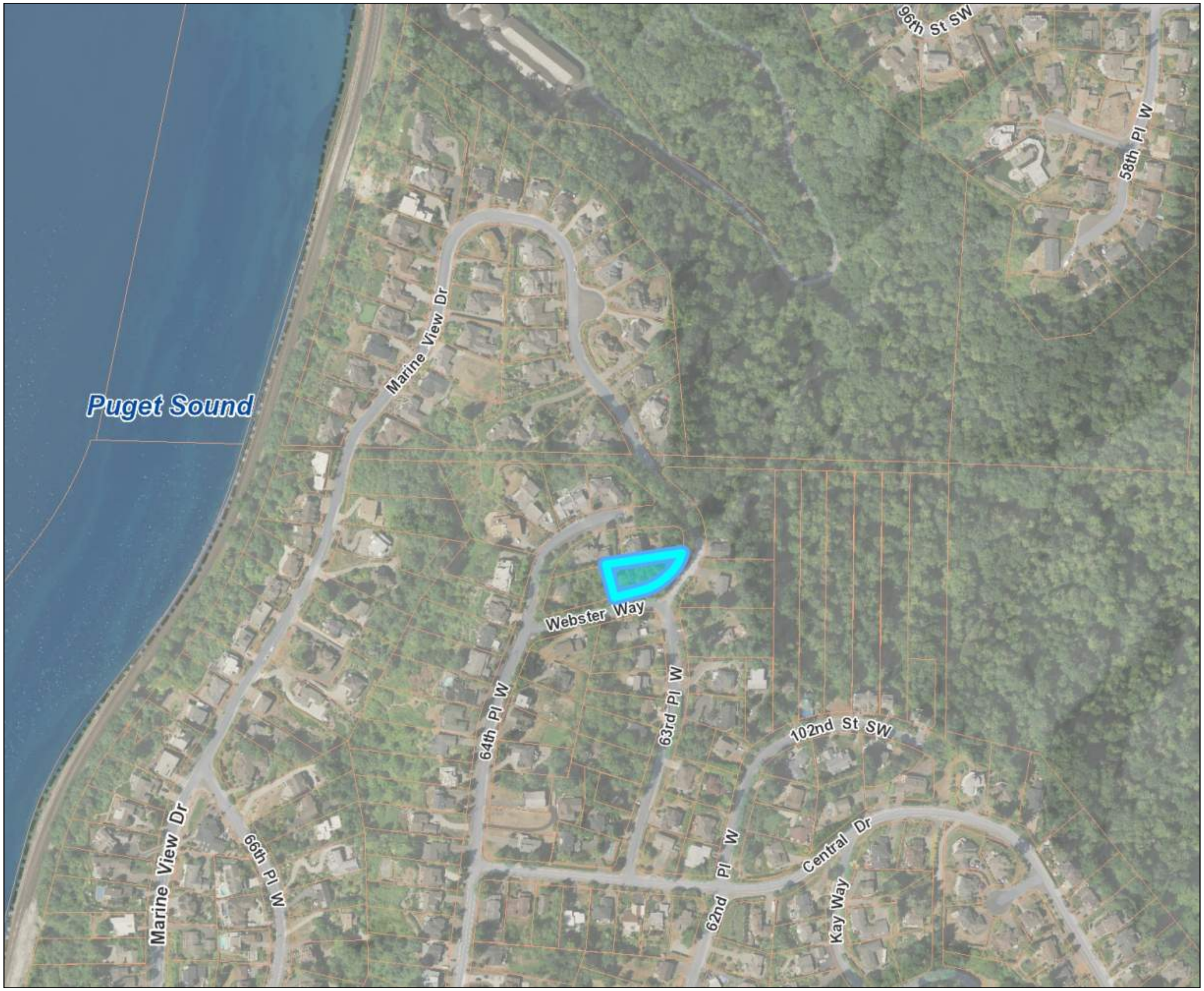
Runoff from the rooftop area will be conveyed in roof drains and will directly discharge to the existing roadside ditch. Runoff from the driveway area will sheet flow and disperse through the adjacent vegetation to the existing roadside ditch. The existing ditch along Webster Way will be preserved and remain after construction.

A geotechnical engineering study was completed by GeoSpectrum Consultants, Inc., dated November 27, 2017. Four test pits were logged and sampled where slopes were less than 40 percent on site. In summary, onsite soils are not sufficient to use for stormwater infiltration as the subsoils underlying the property are fine grained and silt advance outwash soil deposits. Refer to the geotechnical report for more information provided in Appendix A of this report.

The purpose of this report is to encapsulate the documents and analysis in accordance with the Stormwater Management Manual for Western Washington (Amended 2014)(SWMMWW) for the proposed single family residence.

Figure 1.1
Vicinity Map

Snohomish County Assessor
Washington



Legend

- Parcel
- Recent Sales 2021
- Recent Sales 2020
- Recent Sales 2019
- City Boundary
- County Park
- National Forest
- Water

Street Types

- Interstate
- State Route
- Local Road

0 450 900 Feet

4/12/2021

SECTION 2 – EXISTING CONDITIONS

The project site is a total of 0.31 acres and is zoned as Single-Family Residential, RD 12.5(S). The property is undeveloped with trees and vegetation. Parcels surrounding the project site have been developed with single-family residences. In the northwest corner of the property there is a shed and two keystone walls. In the same area, there is a landscape hedge and a small portion of a concrete driveway that encroaches into the property.

The project site contains significant grade change. There is approximately 38 feet of elevation drop northwest to southeast across the site. Runoff currently sheet flows from the site to the existing roadside ditch. The existing ditch flows west to east along Webster Way and follows the existing gravel driveway northeast.

SECTION 3 – OFFSITE ANALYSIS

This narrative is to provide an offsite analysis for the proposed residence. The analysis is to identify and evaluate any noted existing offsite flooding, erosion, and water quality problems or that that may potentially be created or aggravated by the proposed project. The primary component of this offsite analysis is the downstream corridor. The second component is to evaluate the upstream drainage system to verify that there is no offsite run-on that may impact the project.

Study Area Definition and Maps

The study area consists of 1/4-mile downstream field investigation of stormwater released from the existing site. See Figure 1.1 and Figure 3.1 for the Vicinity Map and Downstream Analysis Flow Path exhibit.

Offsite – Upstream runon:

Both parcels north of the project site (00408600400-100 & -400) are developed with existing residences located at a higher elevation than the site. Due to the topography and development, there is minimal upstream runon to the site. The insignificant amount of runoff will continue to sheet flow through the vegetation onsite to the existing roadside ditch.

Offsite – Downstream:

Stormwater runoff from the developed project site will continue to discharge to the existing roadside ditch. The existing ditch follows Webster Way and continues northeast along the gravel road. At the end of the gravel road, stormwater is dispersed in a vegetated corridor and flows approximately 1,700 feet through vegetation and ravines eventually discharging to the Puget Sound.

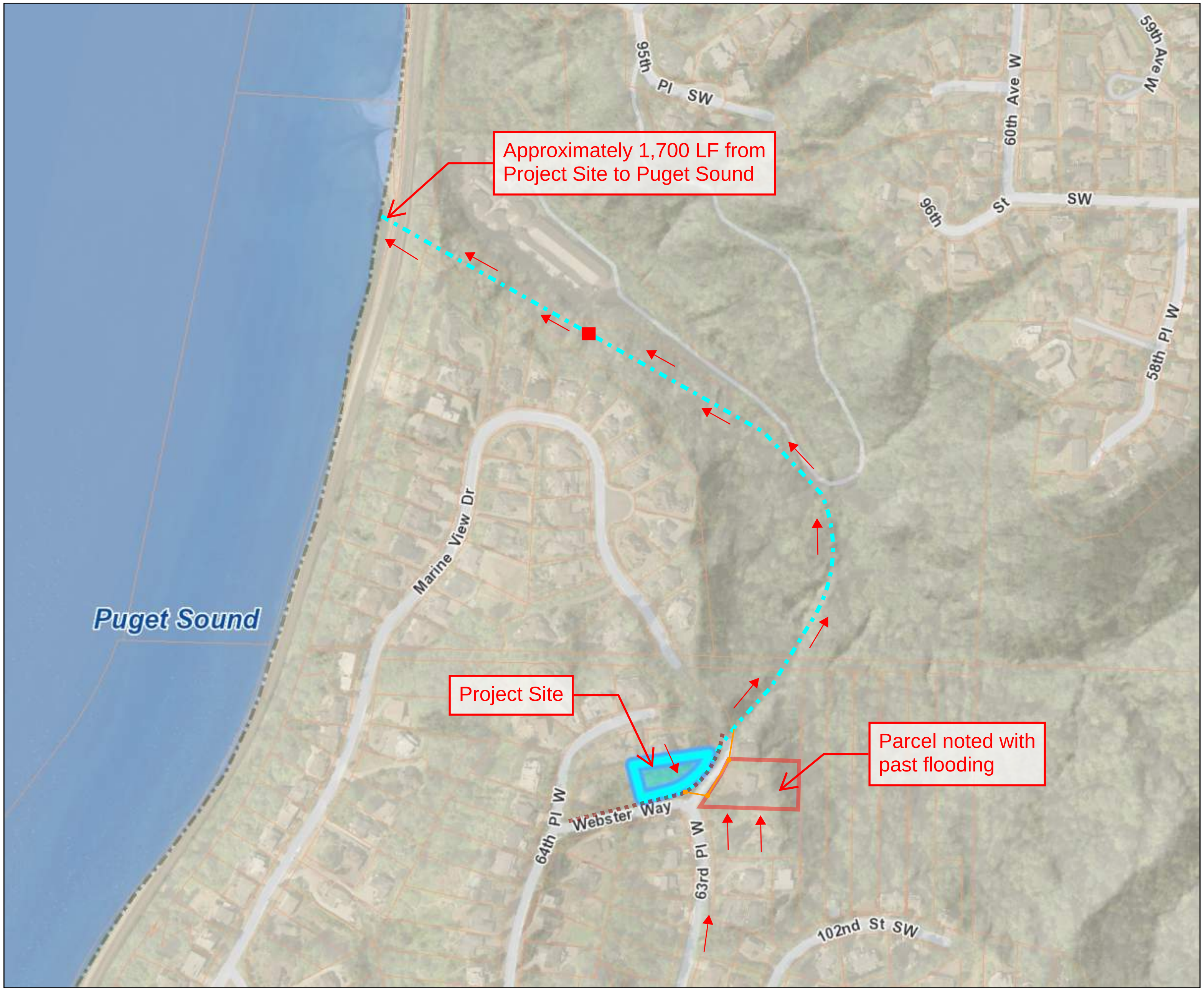
Drainage Description and Problem Descriptions

The downstream corridor is made up of an existing ditch and a vegetative corridor. There are potential problems that could arise by directing additional water through the roadside ditch, such as capacity concerns, increased maintenance, and acquiring necessarily easements.

Based on discussion with the City, the home at 10101 63rd Pl W has experienced flooding in the past. See Figure 3.1 for parcel location. From the records, it is unclear that it was caused by ROW drainage. After further investigation and discussions with the project owner, the residence of concern at 10101 63rd is the house is located in a partial closed depression on the south side of 63rd Pl W where runon to this property would come primarily from the houses to the south. The proposed project and downstream corridor is on the opposite/north side of the street and therefore is not tributary to 10101 63rd Pl W nor would it contribute to any potential flooding issues.

Figure 3.1
Downstream Analysis Flow Path Exhibit

Snohomish County Assessor Washington



Legend

- Parcel
- Recent Sales 2021
- Recent Sales 2020
- Recent Sales 2019
- City Boundary
- County Park
- National Forest
- Water

Street Types

- Interstate
- State Route
- Local Road

0 450 900 Feet

Slope arrow (where drainage is likely flowing) | Storm pipe | Type II Storm Structure
Flow path | Ditch | 1/4 mile downstream | Catch Basin

4/12/2021

SECTION 4 – PERMANENT STORMWATER CONTROL PLAN

Stormwater Control:
The project must comply with the City of Mukilteo surface water runoff standards, which the City has adopted the SWMMWW. Explanation and analysis of the proposed stormwater system is provided in the remainder of this report and has been summarized below.

Land use areas and basin limits
The buildable area for this project is limited to the west side where slopes are less than 40 percent. Approximately 0.13 acres of the site will be disturbed while the remaining area will be protected. After completion of this project, approximately 69 percent of the disturbed area will be impervious while the remaining 31 percent as landscaping.

Table 1 - Basin Areas (Ac)

Land Use Type	Predeveloped	Developed
Forest	0.13	0
Impervious	0	0.09
Pervious	0	0.04
Total	**0.13**	**0.13**

Runoff from roof areas will be conveyed through a roof line that is tightlined and connects to a conveyance pipe that routes the runoff to the roadside ditch. Runoff generation from the concrete driveway will disperse through a vegetated flow path to the existing ditch.

Onsite Stormwater Management BMPs
Figure 2.4.1 of the SWMMWW was followed to determine to what extent and what onsite BMPs are necessary. The project triggers Minimum Requirements 1-5 as more than 2,000 sf and less than 5,000 sf of new hard surface area are proposed. Project's triggering requirements #1-5 are subject to the On-site Stormwater Management BMPs from List #1 for all surfaces within each type of surface where feasible.

List #1 of the SWMMWW:

Lawns must implement Post-Construction Soil Quality and Depth (BMP T5.13)

Roofs must evaluate:

1. Full Dispersion or Full Infiltration
2. Bioretention facilities
3. Downspout Dispersion Systems (splash blocks or gravel trenches)
4. Perforated Stub-out Connections (BMP T5.10C)

Other Hard Surfaces must evaluate:

1. Full Dispersion
2. Permeable Pavement
3. Bioretention facilities
4. Sheet Flow Dispersion

This project elects to implement the feasible onsite BMPs summarized as follows:

- All lawns and pervious surfaces will have BMP T5.13 Soil Quality & Depth applied.
- All roofs (after evaluation) do not meet the criteria necessary to support any of the BMPs listed; therefore, none are proposed.

- All other hard surfaces (after evaluation) do not meet the criteria necessary to support any of the BMPs listed; therefore, none are proposed.

To support the BMP feasibility analysis, the Geotechnical Engineering Report prepared by GeoSpectrum Consultants, Inc., dated November 27, 2017, was utilized. GeoSpectrum's report describes the underlying soils on the property as fine grained and silt advance outwash soil deposits, so onsite infiltration of stormwater runoff is not feasible on the project site. Refer to the geotechnical report included in Appendix A of this Report.

Lawn and landscaped areas:

1. *Post-Construction Soil Quality and Depth in accordance with BMP T5.13: Post Construction Soil Quality and Depth.*

   The project proposes to amend all soils per BMP T5.13 where applicable.

Rooftops:

1. *Full dispersion in accordance with BMP T5.30 or Downspout Full Infiltration in accordance with BMP T5.10A.*

   Due to the topography and location of the proposed residence, there is insufficient vegetated area with slopes of 15 percent or flatter onsite to provide the minimum flow path required for Full Dispersion for both roofs and other hard surfaces.

   Based on the geotechnical report, infiltration is not feasible as test pit locations 1 through 3 were very fine silty sand/sandy silt that was underlain by silt and sandy silt; therefore, Full Infiltration is not feasible for the roofs.

2. *Rain Gardens in accordance with BMP T5.14A, or Bioretention in accordance with BMP T7.30 in accordance with BMP T5.30:*

   Bioretention is not feasible for this project site (roofs and other hard surfaces) due to existing silty soils onsite that are very stiff and dense.

3. *Downspout Dispersion system in accordance with BMP T5.10B*
4. *Perforated Stub-out Connection in accordance with BMP T5.10C*

   To provide proper flow control mitigation to the full roof area, the roof line is tightlined and must be piped into the proposed onsite conveyance pipe. Roof runoff is controlled and piped around the new residence in a non-erosive manner. The geotechnical report recommends that the roof drains should be tightlined into the storm drain (no splash blocks); therefore, Downspout Dispersion and Perforated Stub-out Connections are not feasible.

Other Hard Surfaces:

1. *Full dispersion in accordance with BMP T5.30 or Downspout Full Infiltration in accordance with BMP T5.10A.*

   Due to the topography and location of the proposed residence, there is insufficient vegetated area with slopes of 15 percent or flatter onsite to provide the minimum flow path required for Full Dispersion for both roofs and other hard surfaces.

2. *Permeable pavement in accordance with BMP T5.15, or Raingardens in accordance with BMP T5.14A.*

   Due to silty soils onsite, Permeable pavement is not feasible; therefore, permeable pavement is not feasible for this project site (other hard surfaces).

   Bioretention is not feasible for this project site (roofs and other hard surfaces) due to existing silty soils onsite that are very stiff and dense.

3. *Sheet Flow Dispersion in accordance with BMP T5.12 accordance with BMP T5.14A.*

   Other Hard Surfaces have insufficient flow path length available for Sheet Flow Dispersion with the exception of the turnaround area of the driveway on the south side of the house. It is proposed to allow runoff to sheet flow toward the roadside conveyance ditch in the Webster way ROW.

   Runoff from the main driveway will be directed toward the Webster Way roadside ditch.

SECTION 5 – DISCUSSION OF MINIMUM REQUIREMENTS

Minimum Requirements Summary:

The applicable Minimum Requirements for the proposed development (described in detail following this discussion of Minimum Requirements) are 1 through 5 per the flow chart shown on Figure I-2.4.1 of the 2014 Stormwater Management Manual of Western Washington (SWMMWW). The applicability and/or fulfillment for each minimum requirement is described herein.

Minimum Requirement #1 – Preparation of Stormwater Site Plans

This report fulfills the requirements of a Stormwater Site Plan.

Minimum Requirement #2 – Construction Stormwater Pollution Prevention

Temporary Erosion and Sedimentation Control will be provided with the construction plan submittal.

Minimum Requirement #3 – Source Control of Pollution

The applicable construction source control BMPs for this project include silt fence, stabilized construction access, and catch basin inserts to mitigate the effects of construction activities on downstream water quality.

Minimum Requirement #4 – Preservation of Natural Drainage Systems and Outfalls

Based on existing grades, stormwater runoff naturally flows southeast toward the existing ditch. With this proposal, stormwater runoff will continue to discharge to the roadside ditch.

Minimum Requirement #5 – On-site Stormwater Management

Figure 2.4.1 of the SWMMWW was followed to determine to what extent and what onsite BMPs are necessary. The project triggers Minimum Requirements 1-5 as more than 2,000 sf and less than 5,000 sf of new hard surface area are proposed. Project's triggering requirements #1-5 are subject to the On-site Stormwater Management BMPs from List #1 for all surfaces within each type of surface where feasible.

APPENDIX A - GEOTECHNICAL REPORT

December 18, 2020 G-5333

Mr. Chris Estes
6116 Chennault Beach Drive
Mukilteo, Washington 98275
Email: chrismestes@gmail.com

Subject: **ADDENDUM LETTER**
Proposed New Residence
6300 Webster Way
Mukilteo, Washington 98275

Ref: "City of Mukilteo Determination of Completeness, RUP-HE-2020-001 / SFR-2020-005 / ENG-2020-009, Linda Ritter Senior Planner, October 18, 2020."

"Geotechnical Reconnaissance, Residential Property Development, Snohomish County Parcel No. 00408600400, 100XX 63rd Place West, Mukilteo, Washington, Project No. 17-114-01, Geospectrum Consultants, Inc., November 27, 2017."

"Estes Residence, 6300 Webster Ave, Mukilteo, WA, Nash Associates Architects, September 30, 2020."

Dear Mr. Estes,

We understand that the City of Mukilteo has requested a geotechnical addendum letter regarding compliance of the proposed project plans with recommendations outlined in the above-referenced geotechnical report, and that the geotechnical engineering firm that wrote the report is no longer accepting new work. We have read the above-referenced geotechnical report and reviewed the project plans to ensure that they are in conformance with the conclusions and recommendations described in the geotechnical report.

BACKGROUND INFORMATION

Based on the information provided, we understand that you are proposing to develop the west section of the existing vacant lot with the construction of a new two-story single-family residence with a south-facing daylight basement and attached garage. The garage will be accessible by a driveway that begins east of the intersection of Webster Way and 63rd Place W and runs parallel to the south property line towards the residence along gradually sloped topography. The residence will have a footprint of approximately 1700 square feet, with a total interior living space of 4,000 square feet within the three floors. We understand that excavations into the existing slope at the west section of the property will be required for the construction of the new residence, but this section of the property is not mapped as a steep slope. As noted in the above-referenced geotechnical report, the site contains steep to very steep slopes within the central and northeast areas and therefore is considered a Geologic Sensitive Area. The existing steep slope and the footprint of the proposed residence are illustrated in in Plate 1 – Site Plan.

GEOLOGIC SENSITIVE AREA REVIEW

The above-referenced geotechnical report mentions that a slope stability analysis was conducted for the steep slope areas in the central and northeast sections of the property. The analysis found that the property has safety factors for deep-seated slope failures greater than 1.5 for the static condition and 1.2 for the seismic condition, indicating that the property is stable in its existing condition. The geotechnical engineer recommended that the proposed residence's setback from the steep slope could be reduced from 25 feet due to the nature of the boundary between the moderate and steep slope areas because the boundary is lateral and not above or below the steep slope.

The geotechnical report concluded that site disturbance for the new residence is acceptable up to the edge of the steep slope area, but that a setback distance of 10 feet from the slope should be implemented regardless. The project plans indicate that the proposed footprint of the residence will be located no less than 10 feet from the edge of the steep slope, and that the driveway area will be located more than 25 feet away from the bottom of the steep slope. It is our opinion that the reduced setback will not cause any adverse impacts to the steep slope area at the central and northeast sections of the property.

CONCLUSIONS AND RECOMMENDATIONS

On December 16, 2020, Bryce Frisher, staff geotechnical engineer from our office, visited the property to conduct a site reconnaissance and ensure that the existing site conditions correspond with the conditions mentioned in the above-referenced geotechnical report. We observed that the west section of the property is relatively flat compared to the central and northeast sections of the property, and that the site appeared stable. The conditions we observed were similar to those described in the above-referenced geotechnical report. Based on the very dense and hard, grayish brown silts and silty sands observed during the subsurface investigation, we agree with the safety factors calculated by the slope stability analysis. Based on our review of the project plans, it is our opinion that the recommendations outlined the above-referenced geotechnical report have been properly implemented into the design and, therefore, the project site will remain stable during and after construction of the new residence.

During construction, a representative from GEO Group Northwest, Inc. should be on site to monitor excavations to suitable bearing soils for the foundations. We should also be on site to inspect the progress of backfill and compaction, subsurface drainage installation, temporary and permanent erosion control, and to verify slope stability throughout the construction process, as noted in the geotechnical report.

PLAN REVIEW AND MINIMUM RISK STATEMENT

Based on the site conditions observed and our review of the project plans, it is our opinion that the recommendations outlined in the above-referenced geotechnical report have been properly implemented into the design of the proposed new single-family residence. The plans show that the residence will be located at the west section of the property where the topography does not contain any critical slopes with inclinations greater than 40%, and that site development will not occur within a setback distance of 10 feet from the western edge of the steep slope. In our opinion, these plans will not adversely impact the steep slope or the adjacent properties to the north and west.

Based on our final review of the project plans, it is our opinion that the property will not be adversely impacted by the new residence. The project will not increase the potential for soil movement, and the risk of damage to the new residence and to adjacent properties from soil instability will be minimal, provided that the recommendations outlined in the geotechnical report are satisfied during construction. Minimum risk does not mean no risk, but that necessary design measures have been taken to reduce the level of risk to a low or minimal quantity.

Sincerely,

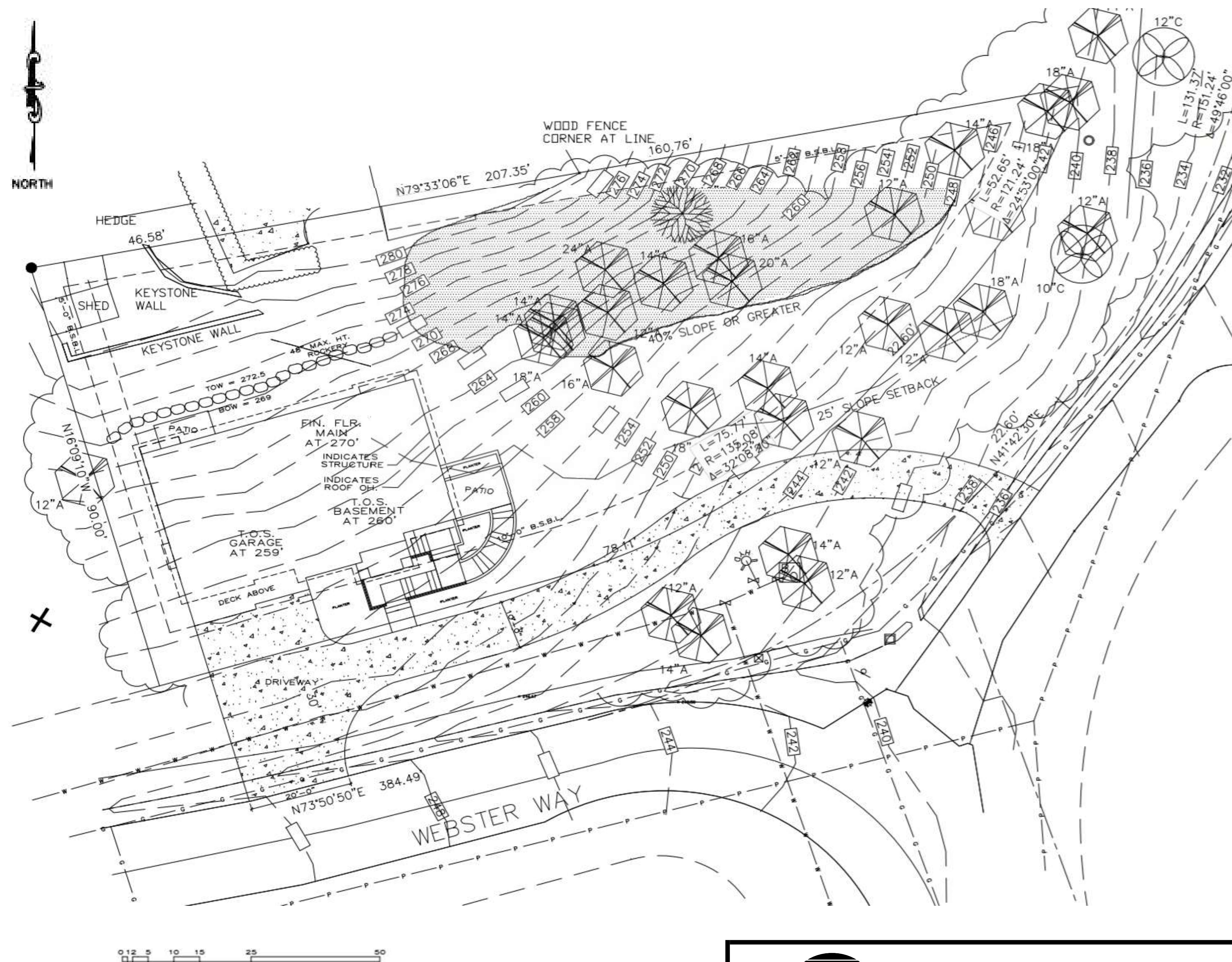
GEO GROUP NORTHWEST, INC.

Bryce Frisher, E.I.T.
Staff Geotechnical Engineer

William Chang, P.E.
Principal Engineer

Plates:

Plate 1 – Site Plan



Source: Estes Residence, Site Plan, Nash & Associates Architects, Dated 9/30/2020

GEO Group Northwest, Inc.

Geotechnical Engineers, Geologists, &
Environmental Scientists

SITE PLAN

PROPOSED NEW RESIDENCE
6300 WEBSTER WAY
MUKILTEO, WASHINGTON

SCALE	AS SHOWN	DRAWN BY	BF	CHECKED BY	WC	DATE	12/18/2020	PROJECT NO.	G-5333	PLATE	1

GEOSPECTRUM CONSULTANTS, INC.

Geotechnical Engineering and Earth Sciences

November 27, 2017

Victoria and Karl Bratvold
816 2nd Street
Kirkland, WA 98033

SUBJECT: GEOTECHNICAL RECONNAISSANCE
Residential Property Development
Snohomish County Parcel No. 00408600400
100XX 63rd Place West
Mukilteo, Washington
Project No. 17-114-01

Dear Vicki and Karl,

This report presents the results of our evaluation of your subject parcel for residential development . Our work was performed in accordance with the conditions of our proposal dated November 3, 2017. The purpose of our work was to evaluate the site stability and provide our recommendations for slope buffer and setbacks as well as recommendations for site grading and foundation design for development residential development.

At this time you have no specific plans for site development. We have assumed the residential structure would be wood frame construction with 1 or 2 stories above a daylight basement. Based on our experience structural wall loads are assumed to be in the range of about 1 to 3 kips per foot and isolated column loads are assumed to be 25 kips or less. If actual loads are different our office should be notified.

P.O. Box 276, Issaquah, WA 98027-0276 • Phone: (425) 391-4228 Fax: (425) 391-4228

SCOPE OF WORK

Our scope of work included site reconnaissance, subsurface explorations, laboratory testing, engineering evaluations and the preparation of this report. The scope of work included the following specific tasks:

- o Reviewed published geologic mapping and topographic mapping of the site vicinity.
- o Performed a site reconnaissance to observe the surface conditions at the site and note relevant features on the site.
- o Excavated four test pits to observe and sample the shallow subsurface conditions. Approximate locations of the test pits are shown on Figure 3 and logs of the test pits are included in Appendix A.
- o Performed laboratory testing including moisture content and classification.
- o Performed engineering evaluations and analyses based on the site conditions observed and encountered in our explorations and the results of our laboratory testing.
- o Prepared this geotechnical report summarizing our findings, evaluations and recommendations for development of the subject property.

OBSERVED SITE CONDITIONS

Surface Conditions

The subject lot is generally located within the coastal bluff area of Mukilteo on the south side of a system of incised drainages (see site vicinity map of Figure 1). The topography of Figure 1 shows the site to be located along the SE flank of a broad ridge north of Central Drive in Mulkilteo. Specifically the subject lot is at the NW corner of the intersection of Webster Way and 63rd Place West (see Figures 2 and 3).

Figure 3 shows that the subject lot includes a relatively flat lying area in the northwest corner above moderately sloped areas to the southwest and steep to very steep slopes in the northeast area. Based on the topography of Figure 3, the subject property has about 35+ feet of elevation difference across the lot from the NE corner to the NW corner. The topography shown on Figure 3 and our own supplemental measurements indicates gradients of the subject lot range from moderate slopes of about 25 to 35 percent along the west side increasing to steep to very steep slopes of 40 to 100+

percent in the central to northeastern areas as shown in Figure 3. Approximate deliniation of the slope gradient breaks based on our site observations and measurements are shown on Figure 3. The moderate slopes within the western area of the lot are only about 20 feet in height within the property but the steep slope areas range from about 25 to 30 feet in height.

The entire sloped area of the lot is wooded with alder trees that range from about 8" up to about 24" in diameter. Many of the trees, particularly within the steep and very steep slope areas (40%+ to 100% gradients) were bowed and/or leaning. Understory vegetation within the moderate and steep slope areas included alder saplings, blackberries and sword fern. Understory vegetation within the very steep slope area (70 to 100% gradients) generally consisted of a heavy growth of an ivy-like ground cover and scattered blackberries. Vegetation within the upper flat area of the site included grasses and landscaping plants such as rhododendron and arborvitae at the eastern end.

We also noted a plastic storage shed and a 2 ft high landscape block wall in the relatively flat lying area above the slope at the NW corner of the lot as shown in Figure 3 as well as a thick layer of yard waste debris near the top of slope in the NW corner also shown on Figure 3.

Numerous marmot burrows were also observed on the property, particularly in the western area of the lot.

Subsoils

Subsurface conditions were explored by four test pits excavated within the subject lot at the approximate locations shown on Figure 3. More detailed descriptions of the subsurface conditions encountered at each test pit as well as laboratory test results are presented in Appendix A.

Our observations of the subsoils exposed in the test pits indicated that the subsoils encountered are natural. The upper subsoils at the three western test pit locations (TP-1, TP-2 and TP-3) were very fine silty sand/sandy silt that was generally underlain by silt and sandy silt at depths of about 2 to 3 feet to the maximum depths of the test pits. However, at TP-4 in the southeastern area of the lot the deeper soils became increasingly coarse and gravelly with depth.

The surficial natural silt/sand soils were loose to medium dense. Surface probing across the lot indicated the loose surficial soils ranged from about 0.5 feet to 2.5 feet in thickness and were typically 1.5 to 2 feet thick. The natural deeper silt soils were typically very stiff to hard and cemented.

The surface soils were dark brown and the deeper natural soils were generally brown to light brown and gray-brown to the depths explored.

Surface and Subsurface Water

No active surface seepage or springs were observed on the site and no free ground water was observed in any of the test pits. The upper subsoils were generally classified as moist to very moist and the deeper subsoils generally became less moist with increasing depth. Measured moisture contents of the subsoils ranged from about 8 to 22 percent of dry weight.

Subsurface Variations

Based on our experience, it is our opinion that some variation in the continuity and depth of subsoil deposits and ground water levels should be anticipated due to natural deposition variations and previous onsite grading. Due to seasonal moisture changes, ground water conditions should be expected to change with time. Care should be exercised when interpolating or extrapolating subsurface soils and ground water conditions between or beyond our test pits.

SITE EVALUATIONS

General

The referenced geologic map of Figure 1 indicates the site to expose advance outwash (Qva) soils deposited during the advance of the Vashon glaciation, the last glacial advance into the Puget Sound area, approximately 13,000 to 16,000 years ago. The referenced map describes the Qva soils as mostly sand and gravel deposits but fine grained sand and silt deposits are common in lower part of the unit. Based on the soils observed on the site and the fact that the site lies within the lower part of the mapped limits of Qva deposits shown on the referenced map, it is our opinion that the natural subsoils underlying the subject property are fine grained sand and silt Qva deposits.

Our communications with the City of Mukilteo indicated that the subject lot was originally platted in 1943 and was annexed into Mukilteo in 1991. However, we understand that no grading plans or original topography data for the lot is on file with Mukilteo. Considering that all of the slope areas of the lot are wooded with Alder trees ranging up to about 24 inches in diameter indicates that the site was likely cleared and possibly graded at some time in the past.

Based on a Growth Factor of 2.0 to 4.0 years of growth per inch of tree diameter for Alder and Maple trees based on communications with Olaf K. Riberio, Director of Plant Pathology, Compliance Services International, we estimate the age of the largest Alder trees onsite (24 inches) to be in the range of about 48 to 96 years with a best estimate average of about 72 years. A 72 year tree age would indicate the trees began to grow in about 1945 which corresponds well to the 1943 plat date provided by Mukilteo.

Geologic Hazards Assessment

Slope Stability: The City of Mukilteo landslide hazard map provided to us indicates the site is mapped within a "Moderate" landslide hazard area. The geologic map of Figure 1 indicates no mapped landslides within the site vicinity but the topography of Figures 2 and 3 and our own observations indicate that the site does contains steep to very steep slopes within the central and northeastern area of the lot. Therefore the lot is considered to be a Geologic Sensitive Area and development must comply with the regulations presented in Chapter 17.52A of the Mukilteo Municipal Code (MMC).

Our site observations indicate the subject lot is currently stable and most of the smaller trees on the lot are growing relatively straight, but we observed several of the larger Alder trees particularly within the very steep slope area (70 - 100% area shown on Figure 3) that are severely bowed and/or leaning indicating possible past shallow soil movement. The bowed trees are bowed in the downslope direction near the base of the trees indicating that the trees were tilted downslope early in their life (1940's to 1950's).

As with all development on or near slopes, the owner, must be aware of and accept the risk that future slope failures may occur and may result in damage to his property and/or neighboring property. The risk of structure damage resulting from a slope failure varies with the distance from the slope, the slope height and its steepness as well as other factors. We evaluated the stability of the slopes by performing stability analyses based on the subsurface conditions observed in our explorations and considering both static conditions and the IBC seismic criteria discussed below under the seismic evaluations. Results of our analyses indicate that the slopes on the subject lot have a safety factor for deep seated slope failures greater than 1.5 under static conditions and greater than 1.2 under seismic loading conditions.

With regard to the potential for shallow surface failures, it is our opinion that the potential for shallow failures within the western moderate gradient area of the lot area is low but potential for shallow failures within the central steep slope area (gradients 40% to 50%) is moderate and the potential for shallow failures within the northeastern very steep slope area (gradients 70% to 100%) is high. The risk of shallow slope failures will generally be greatest during periods of heavy rainfall and/or seismic loading conditions. Reduction or elimination of the apparent Marmot population on the lot will reduce the disturbance of the shallow slope soils and reduce water infiltration into the shallow soils which will reduce the potential for shallow slope failures. Removal of the yard waste debris is also recommended to reduce the slope load.

<u>Slope Buffers and Structure Setbacks:</u> We do not recommend development or disturbance of the steep to very steep slope areas of the site. Development should be limited to the moderately sloped areas of the subject lot (slope gradients less than 40%) which are generally in the western 1/3 of the lot and along the southern boundary as shown in Figure 3. In general to minimize development risk, structures should be set back from the top and toe of steep to very steep slopes as far as possible within the constraints of the development plans.

Section 17.52A .50.A of the MMC states that the setback from a steep slope shall in no case be less than 25 feet unless allowed through the "Reasonable Use" provision (RUP) of the MMC and supported by a geotechnical report approved by the public works director. Figure 4 shows the approximate location of a 25 foot setback line from the steep slope boundary. Based on the approximate 25 foot setback line of Figure 4 and considering a 10 foot side yard setback, the width of the buildable area would range from only about 15 feet at the northwest corner up to a maximum of about 35 feet at the southwest corner. It appears that without relief through the "Reasonable Use" provision of the MMC the developable area of the lot would be significantly reduced.

However, in our opinion the structure setback from the steep slope area could be significantly reduced due to the nature of the boundary between the moderate and steep slope areas. The moderately sloped western area is not above or below the steep slope areas of the lot, but rather the boundary between the moderate and steep slope areas is a lateral boundary as approximately shown on Figure 3. Considering that the boundary with the steep slope area is lateral, in our opinion development related site disturbance of the moderately sloped area may extend to the edge of the steep slope area provided the disturbed areas are stabilized after construction, but we recommend that structure foundations be set back at least 10 feet from the edge of the steep slope area or greater as required for temporary construction excavations per our recommendations in this report. Therefore for "Reasonable Use" provision considerations for the subject lot our recommended minimum steep slope buffer would be zero and our recommended minimum buffer setback would be 10 feet.

In addition, square footings and continuous footings located in slope areas should be deepened as required to provide a horizontal setback of at least 5 feet or two footing widths (whichever is greater) from the sloping surface of the very stiff or dense natural bearing soils (typically expected to be about 2 to 2.5 feet below the existing surface). Footings should also be deepened as required to be below a 1:1 (h:v) projection up from adjacent lower footings. Where the natural bearing soils slope, the footing excavation should be stepped to maintain a horizontal bearing surface.

<u>Erosion:</u> We observed that the site is well vegetated and we observed no indication of any seepage or concentrated water flow or current or past erosion on your property but did note numerous Marmot burrows and waste mounds on the lot. Based on our site observations and explorations and assuming that the Marmot population is controlled or eliminated, it is our opinion that there is generally low erosion risk at the lot and any erosion potential resulting from development will be mitigated by our recommended grading procedures and drainage/erosion control measures and by final re-vegetation/landscaping recommended to be incorporated into the proposed development plans.

<u>Seismic:</u> The lot is mapped by Mukilteo as a "Moderate" seismic hazard. The Puget Sound region in general is a seismically active area. About 17+ moderate to large earthquakes (M5 to M7+) have occurred in the Puget Sound and northwestern Cascades region since 1872 (145 years) including the 2/28/01 M6.8 Nisqually earthquake and it is our opinion that the proposed structures will very likely experience significant ground shaking during their useful lives.

The nearest known fault to this site is the northwest-southeast trending South Whidbey-Lake Alice fault zone which has a postulated maximum credible earthquake magnitude of 7.0 to 7.5 and is mapped to pass through the immediate site area with surface traces mapped to both the north and south within about 1 to 2 miles of the site. Other regional faults include the Seattle fault zone about 25 miles south of the site which also has a postulated maximum credible magnitude of 7.0 to 7.5. A study of the Vashon-Tacoma area also provided evidence for the east-west trending Tacoma Fault which is indicated to pass through the south end of Vashon and the middle of Maury Island about 40 miles south of the site. The study suggests that the Tacoma Fault and the Seattle fault may be linked by a master thrust fault at depth.

The recurrence of a maximum credible event on the South Whidbey fault is not known but some experts have assigned a recurrence of about 3000 years, however smaller events will occur more frequently as evidenced by the 5.3 event on May 2, 1996 which was attributed to that fault. The Seattle fault has been documented to have moved at its west end (Bainbridge Island) about 1000 to 1100 years ago and evidence of movement at the east end has also recently been documented. Some experts feel that the recurrence interval between large events on the Seattle Fault may be on the order of several thousands of years but our calculations indicate it may be on the order of 1200 to 1400 years. The activity of the documented Tacoma fault is considered to be on the same order as the Seattle fault.

In addition to Puget Sound seismic sources, a great earthquake event (M8 to M9+) has been postulated for the Cascadia Subduction Zone (CSZ) along the northwest coasts of Oregon, Washington and Canada. The current risk of a future CSZ event is not precisely known, but a published report indicates that the recurrence intervals for CSZ events over the last 2300+ years have averaged about 234 years between events and have not exceeded 300 years during that 2300+ year period. However, the time of the last event has been well documented to have occurred 317+ years ago (January 1700) and therefore in our opinion a CSZ event should be expected in the near future.

Considering all of the above, it is our opinion that the proposed residential development will very likely experience significant ground shaking during its useful life. The 2015 International Building Code (IBC) which has been adopted by the City of Mukilteo requires that a Maximum Considered Earthquake Geometric Mean (MCE_G) ground motion peak horizontal ground acceleration (PGA) be used for site liquefaction evaluations but the 2015 IBC Design Earthquake which is defined as 2/3 of the MCE_G ground motions in ASCE 7-10 may be used for consideration in other geotechnical seismic site evaluations for new construction.

The MCE_G PGA for the 2015 IBC per ASCE 7 is based on consideration of both probabilistic ground motions with a 2475-year recurrence interval and deterministic ground motions based on a model of known fault locations and characteristics adjusted for site specific soil conditions. Per section 1803.5.12(2) of the 2015 IBC, we have estimated the MCE_G PGA for this site to be about 0.53g in accordance with Section 11.8.3 and Figure 22-7 of ASCE 7-10. We estimate the IBC Design Earthquake ground motion PGA for this site to be 0.35g per the definition in Chapter 11 of ASCE 7-10. Please note that the Design Earthquake ground motion PGA is not intended for structural analyses. Spectral accelerations presented in the 2015 IBC should be considered in structural design.

This site is considered to be a Site Class C per the 2015 IBC and the referenced definitions presented in Chapter 20 of ASCE 7-10.

Secondary seismic hazards due to earthquake ground shaking include induced surface rupture, slope failure, liquefaction, lateral spreading and ground settlement. Considering the close proximity to the South Whidbey-Lake Alice fault zone the potential for surface rupture is considered low to moderate. Considering the lack of shallow ground water at the site, it is our evaluation that the potential for damage to the development due to liquefaction and lateral spreading is very low. Provided the structures are founded on very stiff/hard or dense/very dense natural bearing soils as recommended herein, the potential for significant induced settlement is considered very low. The potential for seismically induced shallow failures is considered low in the non-steep slope areas recommended for development and moderate to high in the steep to very steep slope areas. Structures that are setback from the steep slopes per the steep slope setbacks of Figure 4 and supported on the recommended natural bearing soils should not be significantly affected by seismically induced shallow slope movements.

Structure Support Considerations

Our explorations indicate that the site is underlain by advance outwash soils that are typically very stiff/hard and dense below depths of about 2 to 2.5 feet, however based on the site topography and vegetation (alder trees) it is apparent that the site has been previously cleared and possibly graded and therefore it is possible that there may be fill deposits on the site. Structure support should be extended through any existing fill soils and loose natural soils to bear on undisturbed medium dense to dense natural soils.

Preparation of slab-on-grade subgrade areas should include excavation of all fill soils and loose or organic surficial soils in the subgrade area and replacement with structural fill. Existing sand soils could likely be re-used as structural fill with proper compaction but moisture content of onsite silt soils will likely be difficult to control for proper compaction. As a minimum we recommend that subgrade preparation for a slab-on-grade floor include excavation of all existing fill, organic and loose soils to expose dense/stiff natural soils and replacement with structural fill to final slab subgrade.

Recommendations for foundation design, retaining walls, subgrade preparation and structural fill placement and compaction are presented below in RECOMMENDATIONS.

RECOMMENDATIONS

Recommendations for foundation design, retaining wall design, site grading, drainage control, erosion control, plan review and recommended construction observations are presented below.

Spread Footing Foundations on Natural Soils

Conventional spread footings founded on undisturbed very stiff/hard silt and dense/very dense natural sand/gravel soils can be used for structure support. Any existing fill and loose surface soils should be excavated as required to expose undisturbed very stiff/hard silt and dense/very dense natural sand/gravel soils for foundation support. All footings should be founded at least 18 inches below the lowest adjacent final grade. Square footings should be at least 24 inches wide and continuous wall footings should be at least 18 inches wide. Footings may be designed based on a maximum allowable vertical bearing pressure of 2000 psf.

In addition, square footings and continuous footings located in slope areas should be deepened as required to provide a horizontal setback of at least 5 feet or two footing widths (whichever is greater) from the sloping surface of the very stiff or dense natural bearing soils (typically expected to be about 2 to 2.5 feet below the existing surface). Footings should also be deepened as required to be below a 1:1 (h:v) projection up from adjacent lower footings. Where the natural bearing soils slope, the footing excavation should be stepped to maintain a horizontal bearing surface.

As an alternative to deep spread footings to penetrate unsuitable soils and/or satisfy the footing setback requirements discussed above, foundation loads may be transferred from the recommended minimum foundation depths to the recommended bearing soils by a monolith of lean concrete having a minimum compressive strength of 1000 psi. The width of an un-reinforced lean concrete monolith should be at least as wide as the footing or at least one-third of the monolith height, whichever is greater. Reinforced monoliths should be designed by a structural engineer. A suitable width trench should be excavated with a smooth edged excavator bucket (no teeth) to expose the dense/very dense bearing soils under observation by our office and backfilled as soon as possible with the lean concrete to the footing elevation.

Settlement of spread footing foundations supported on a compacted subgrade with bearing pressure of 2000 psf or less are expected to be about 1/4 to 1/2 inch for loads up to 3 klf. Differential settlements between adjacent foundations is expected to be about ½ inch or less. Settlements are expected to occur primarily during construction.

For lateral design, resistance to lateral loads can be assumed to be provided by friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.35 may be assumed with the dead load forces in contact with onsite soils. An allowable static passive earth pressure of 150 psf per foot of depth may be used for the sides of footings poured against existing loose soils but may be increased to 250 psf per foot for footings bearing laterally against properly compacted structural fill.

The bearing values indicated above are for the total dead load plus frequently applied live loads. If normal code requirements are applied for design, the vertical bearing pressure and the allowable lateral passive pressures may be increased by 33% for wind and seismic forces.

Retaining Walls

Cantilevered retaining walls as referred to in this report are walls which yield or move outward during and after backfilling. Actual wall movements will depend on the wall design and method of backfilling and can range from 0.1% to 0.3% of the wall height. Design pressures for cantilevered walls given below assume that the top of the wall will deflect at least 0.15% of the wall height. Design of wall foundations should be in accordance with the recommendations presented in this report.

Static design of permanent cantilevered retaining walls which support a horizontal surface of properly compacted clean free-draining granular material may be based on an equivalent fluid density of 40 pcf. These pressures assume that there is no water pressure with the wall backfill. For support of sloped backfill up to a 3:1 (h:v) slope a lateral pressure equivalent fluid density of 50 pcf is recommended. An additional uniform lateral pressure due to backfill surcharge should be computed using a coefficient of 0.27 times the uniform vertical surcharge load.

Static design of walls supporting horizontal backfill and structurally braced against movement should be based on an equivalent fluid density of 60 pcf. This pressure assumes that the wall supports a horizontal backfill of properly compacted free-draining granular material and that there is no water pressure behind the wall. For braced support of sloped backfill up to a 3:1 (h:v) slope a lateral pressure equivalent fluid density of 80 pcf is recommended. Uniform lateral pressure due to a uniform vertical surcharge behind a braced wall should be computed using a coefficient of 0.43 times the uniform vertical surcharge load.

Seismic design of retaining walls should include a dynamic soil loading. Dynamic soil pressure should be assumed to have an inverted triangular distribution. Based on a 0.35g IBC design ground motion level the dynamic soil pressure at the top of the wall should be at least 25H (psf) where H is the height of the wall above the footing base. The dynamic soil pressure should diminish linearly to zero at the base of the wall. Combined static plus dynamic soil pressure should be used for seismic design of the walls.

Care should be exercised in compacting backfill against retaining walls. Heavy equipment should not approach retaining walls close enough to intrude within a 1:1 line drawn upward from the bottom of the wall. Backfill close to walls should be placed and compacted with hand-operated equipment. Recommendations for placement and compaction of structural fill are presented under "Site Grading".

Design wall pressures given above assume no water pressure behind the wall. We recommend that a drainage zone be provided behind all walls and a adequate drain system be provided at the base of the walls. Wall drains should consist of a four-inch diameter perforated PVC drain pipe placed in at least one cubic foot of drain gravel per lineal foot along the base of the wall. Drain gravel should be washed material with particle sizes in the range of 3/4 to 1-1/2 inches.

As a minimum, the drainage zone within the upper wall should consist of a Miradrain drainage mat or equivalent attached to the wall surface for the full height and embedded into the drain gravel at the base of the wall. As an alternative a clean sand drainage zone could be placed the full height of the wall with a horizontal width equal to at least 1 foot. Backfill within the drainage zone should be a clean sand/gravel mixture with less than 5 percent fines based on the sand fraction. A membrane of Mirafi 140 filter fabric or equivalent should be provided between the drainage zone material and onsite silty soil backfill. The drainage zone backfill should be capped with 12 inches of silty soils to reduce surface water infiltration.

Site Grading

Site grading is expected to consist of driveway construction and subgrade preparation for construction of foundations, slabs and pavements. Recommendations for site preparation, temporary excavations, structural fill and subgrade preparation are presented below.

Site Preparation: All existing fill soils, organic and loose soils should be stripped from planned structural fill areas. Debris and trash, plus rocks and rubble over 6 inches in size, should be removed from the subgrade. Subsoil conditions on the site may vary from those encountered in the test pits. Therefore, the soils engineer should observe the prepared areas prior to placement of any new fills.

Temporary Excavations: Sloped temporary construction excavations may be used where planned excavation limits will not interfere with other construction. Based on the conditions observed at the site it is our opinion that temporary excavations which will require workers to enter them can be made vertically to 3 feet but deeper excavations in un-saturated soils should be sloped no steeper than 1:1 (horizontal:vertical). Where there is not enough room for sloped excavations, shoring should be provided. It should be noted that the contractor is responsible for maintaining safe construction excavations.

Structural Fill: On site soils may be used for general structural fill (subject to final approval during construction) provided that the soil moisture content is suitable for compaction and they do not contain any organics. All imported fill should be clean, sand and gravel materials free of organic debris and other deleterious material. Structural fill should be placed in horizontal lifts not exceeding 8 inches in loose depth and compacted to the required density.

General structural fill should be compacted to at least 90 percent of the maximum dry density as determined by the ASTM D1557 test method unless otherwise specified. Structural fill within the optional structural fill zone for foundation support should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D1557 test method.

Pavement and Slab Subgrade Preparation: All topsoil, fill and organic soils in subgrade areas should be excavated to expose dense/stiff natural soils and replaced with compacted structural fill to final slab subgrade.

Concrete slabs-on-grade should be supported on a subgrade consisting of general structural fill over dense/stiff natural soils. As a minimum we recommend that subgrade preparation for a slab-on-grade floor include excavation of all existing fill, organic and loose soils to expose dense/stiff natural soils and replacement with structural fill to final slab subgrade.

Risk of slab cracking can be reduced by placing 2-way reinforcement steel, and greater excavation and replacement of the existing soils with new structural fill. If interior concrete slabs are constructed they should be underlain by a polyethylene vapor barrier of at least 6 mil thickness.

Asphalt pavement sections (AC and base course) should be supported on a subgrade consisting of at least 6 inches of crushed gravel over the general structural fill subgrade prepared as recommended above. In driveway areas a minimum 8-inch depth of crushed gravel should be provided above the general structural fill. The imported crushed gravel fill should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D1557 test method.

Drainage Control

Surface drainage from the adjoining upslope areas should be controlled and diverted around the subject lot in a non-erosive manner. Adequate positive drainage should be provided away from the structures and on the site in general to prevent water from ponding and to reduce percolation of water into subsoils. A desirable slope for surface drainage is 2% in landscaped areas and 1% in paved areas.

Roof drains should be tightlined into the storm drain system (no splash blocks). A footing drain independent of the roof drain system and placed adjacent to the base of the continuous exterior foundations. The footing drain should consist of a four-inch diameter perforated PVC drain pipe placed in at least one cubic foot of drain gravel per lineal foot along the base of the foundations. The drain gravel zone around the pipe should be encapsulated with a membrane of Mirafi 140 filter fabric or equivalent between the drainage zone material and onsite silty soil backfill.

Erosion Control

Onsite materials are expected to be moderatelyerodible when exposed to concentrated water flow in slope areas. No excavated material should be wasted on the slopes. Siltation fences or other suitable detention devices should be provided around soil stockpiles and around the lower sides of exposed soil areas during construction to control the transport of eroded material. The lower edge of the silt fence fabric should have "J" shaped embedment in a trench extending at least 12 inches below the ground surface. Surface drainage should be directed away from slopes and exposed soil areas should be planted immediately with grass and deep rooted plants to help reduce erosion potential.

No cutting and clearing should be performed in the steep slope areas and should be minimized in the non-steep slope areas. Pruning or cutting back of trees with a minimum of disturbance to the existing slope vegetation is recommended as opposed to felling. If felling is required, stumps should be left intact where possible to reduce disturbance to the shallow soils.

Observations and Testing During Construction

Recommendations presented in this report are based on the assumption that soil conditions exposed during construction will be observed by our office so that any necessary design changes or supplemental recommendations may be made. All footing excavations should be observed prior to placement of steel and concrete to see that they have penetrated into bearing soils and that excavations are free of loose and disturbed materials. Proper fill placement and compaction should be verified with field and laboratory density testing by a qualified testing laboratory. Installation and load testing of driven pipe piles should be observed by our office to confirm allowable capacities. Drainage control systems construction should be observed to verify proper construction.

Plan Review

This report has been prepared to aid in the evaluation of this site and to assist the owners and their consultants in the design and construction of the project. It is recommended that this office be provided the opportunity to review the final design drawings and specifications to determine if the recommendations of this report have been properly implemented and to make any supplemental design recommendations which may be required.

CLOSURE

This report was prepared for specific application to the subject site and for the exclusive use of Victoria and Karl Bratvold and their representatives. The findings and conclusions of this report were prepared with the skill and care ordinarily exercised by local members of the geotechnical profession practicing under similar conditions in the same locality. We make no other warranty, either express or implied.

Variations may exist in site conditions between those described in this report and actual conditions encountered during construction. Unanticipated subsurface conditions commonly occur and cannot be prevented by merely making explorations and performing reconnaissance. Such unexpected conditions frequently require additional expenditures to achieve a properly constructed project. If conditions encountered during construction appear to be different from those indicated in this report, our office should be notified.

Respectfully submitted,

GEOSPECTRUM CONSULTANTS, INC.

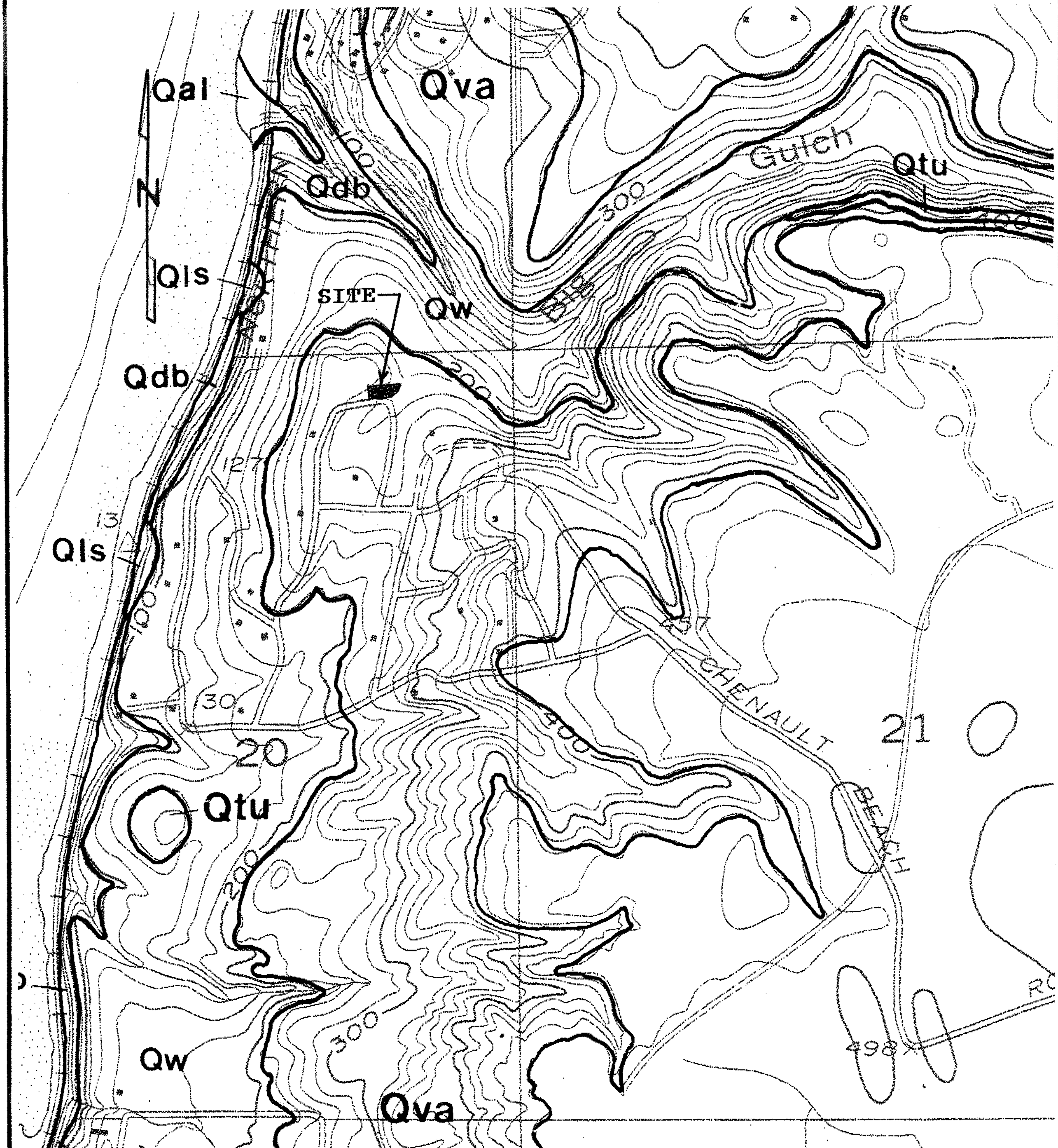
James A. Doolittle
Principal Engineer

JAMES A. DOOLITTLE
STATE OF WASHINGTON
23623
REGISTERED PROFESSIONAL ENGINEER
EXPIRES 3/19/2018

11/27/17

Encl: Figures 1 through 4
Appendix A

Dist: 1/Addressee via email



ref: Distribution and Description of Geologic Units,
in the Mukilteo Quadrangle, Washington,
USGS Map MF-1438, by James P. Minard, 1982,
Enlarged Scale: 1"= 1000'

SITE VICINITY GEOLOGIC MAP

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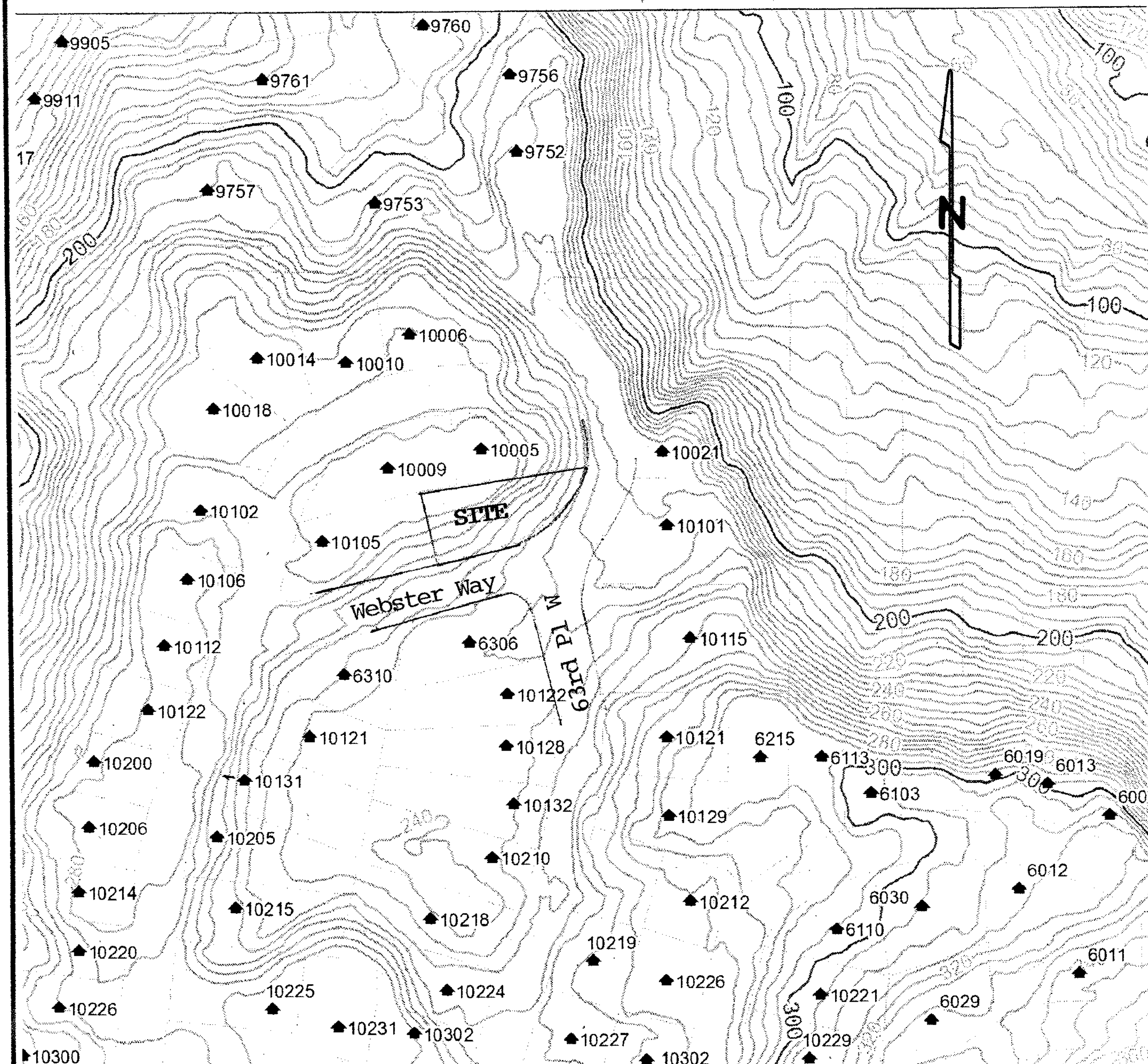
Geotechnical Engineering and Earth Sciences

Residential Property Development
SCPN 00408600400,100XX 63rd Pl West
Mukilteo, Washington

Proj. No.17-114	Date 11/17	Figure 1



ref: Snohomish County PDS Map, 2017
Scale: 1"= 200'±, 5' contours

SITE AREA TOPOGRAPHY

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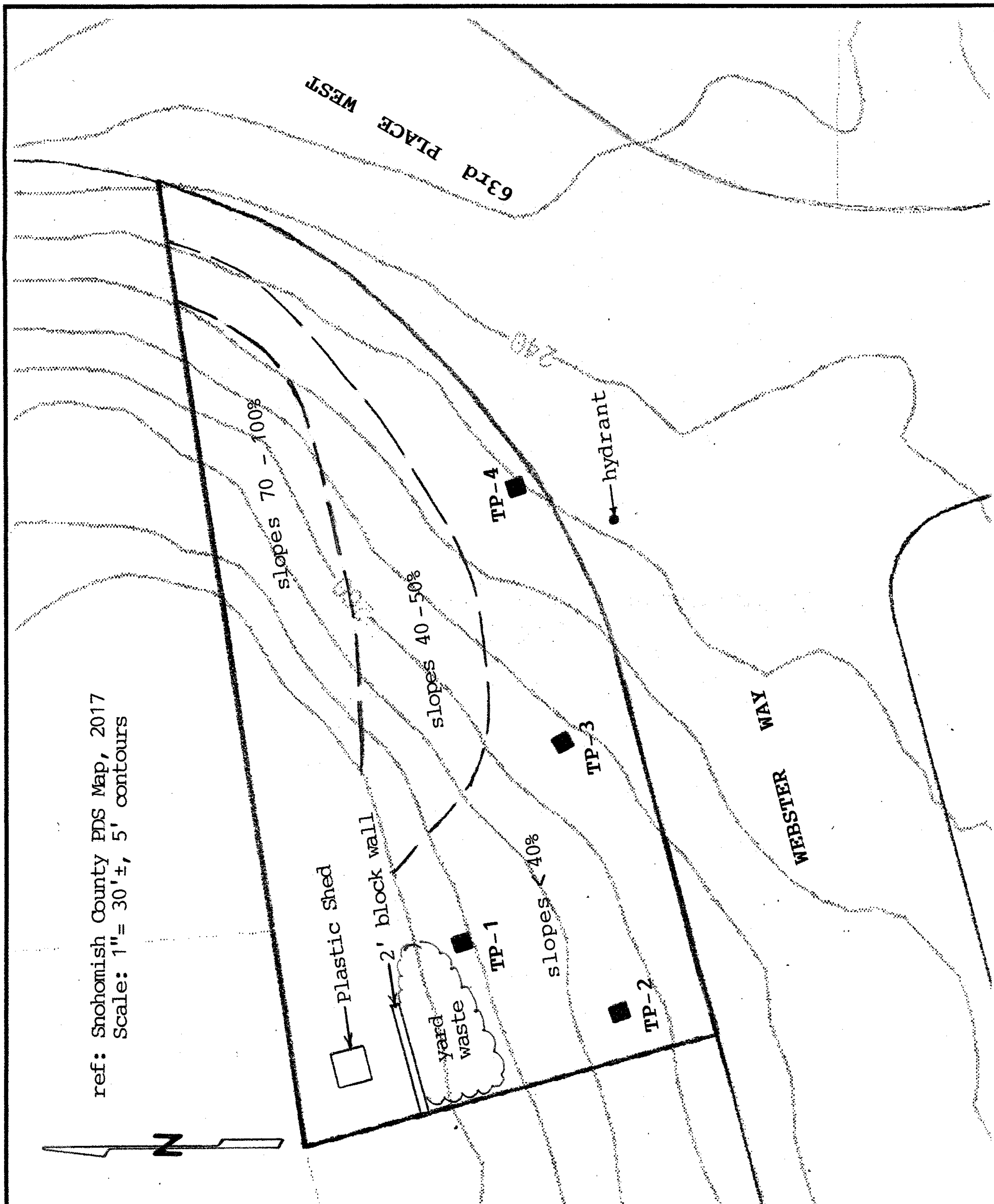
Proj. No. 17-114	Date 11/17	Figure 2



SITE CONDITIONS AND EXPLORATION PLAN

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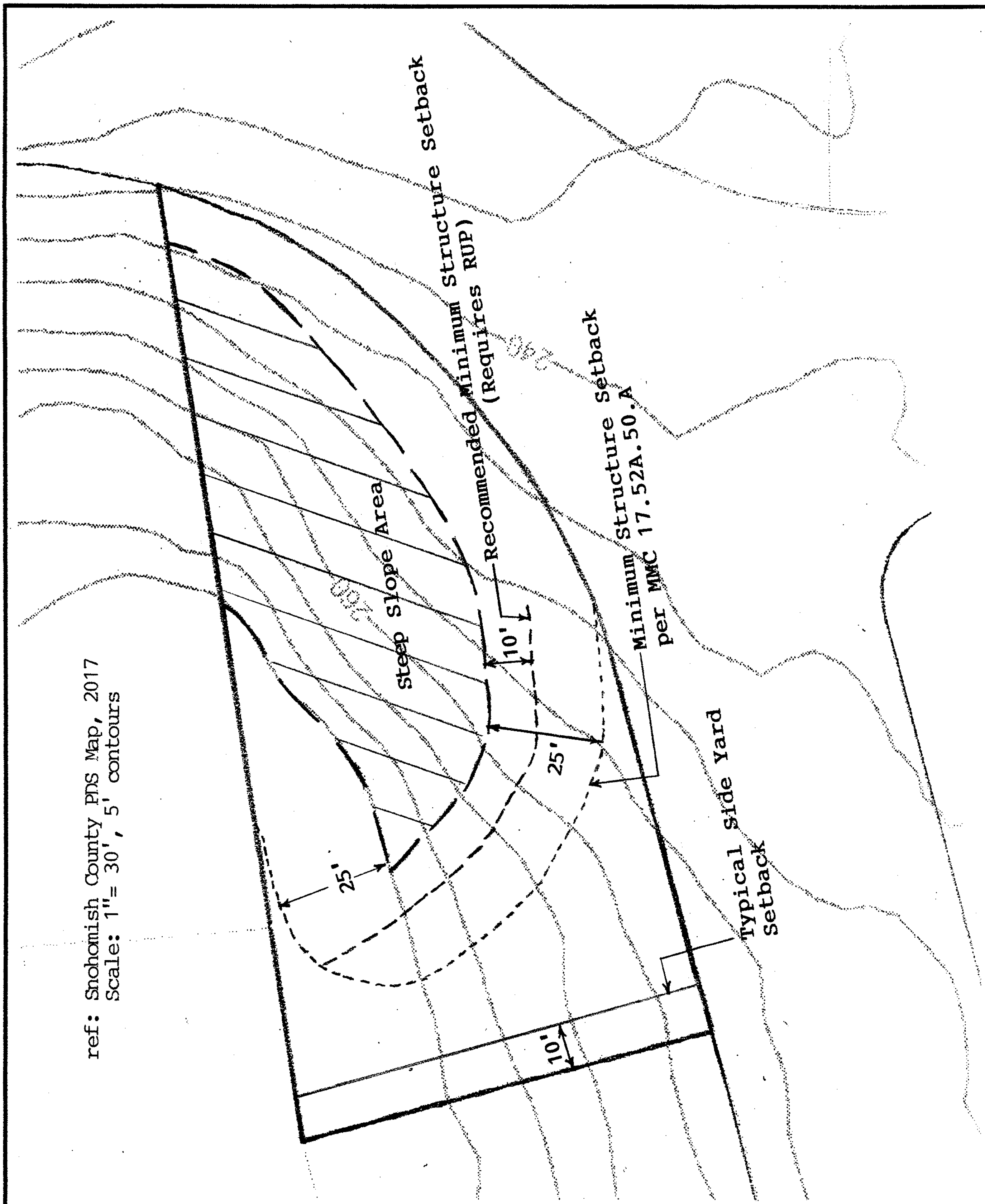
Proj. No.17-114	Date 11/17	Figure 3



MINIMUM STRUCTURE-SLOPE SETBACKS

GEOSPECTRUM CONSULTANTS, INC. Geotechnical Engineering and Earth Sciences	Residential Property Development SCPN 00408600400,100XX 63rd Pl West Mukilteo, Washington	
Proj. No. 17-114	Date 11/17	Figure 4

APPENDIX A
FIELD EXPLORATION

Our field exploration included a site reconnaissance and subsurface exploration program. During the site reconnaissance, the surface site conditions were noted, and the locations of the test pits were approximately determined (see Figure 3). Elevations were based on the topography of Figure 3 and our own measurements.

Test pits were excavated using a Cat 304 trackhoe. Soils were continuously logged and classified in the field by visual examination, in accordance with the ASTM Soil Classification system.

Logs of the test pits are presented on the test pit summary sheets A-1 and A-2. The test pit summaries include descriptions of the soils and pertinent field data. Soil consistency and moisture conditions indicated on the logs are interpretations based on the conditions observed in the field. Boundaries between soil strata indicated on the logs are approximate and actual transitions between strata may be gradual.

TEST PIT NO. 1

Logged by JAD

Date: 11/16/17

Elevation: 271'

Depth	Blows	Class.	Soil Description	Consistency	Moisture	Color	W(%)	Comments
0		OL	Duff/Topsoil	loose	moist	dk brn		
1		SM/ML	Silty very fine Sand/Sandy Silt with roots to 3"		to	brown		
					very moist			
2				m. dense		gray-brown	17.2	
				dense/ v. stiff				
3		ML	Silt cemented	very stiff/ hard	moist		12.3	
4								
5								
							16.5	
6			Maximum depth 6 feet. No ground water encountered.					
7								

TEST PIT NO. 2

Logged by JAD

Date: 11/16/17

Elevation: 262.5'

Depth	Blows	Class.	Soil Description	Consistency	Moisture	Color	W(%)	Comments
0		OL	Duff/Topsoil	loose	moist	dk brn		
1		SM/ML	Silty Sand, very fine/Sandy Silt		to	brown		
					very		14.6	
2				medium dense	moist	light brown to gray-brown		
3			with some cementation	dense/ very dense & hard	moist		11.2	
4			Maximum depth 4 ft. No ground water encountered.					
5								
6								
7								

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Proj. No. 17-114	Date 11/17	Figure A-1

TEST PIT NO. 3

Logged by JAD

Date: 11/16/17

Elevation: 258'

Depth	Blows	Class.	Soil Description	Consistency	Moisture	Color	W(%)	Comments
0		OL	Duff/Topsoil	loose	moist	dk brn		
1		ML / SM	Sandy Silt/ Silty very fine Sand with roots hair to 2"		to very moist	brown & light brown	21.6	
2				m. dense				
		ML	Silt cemented	v. stiff/ hard	moist	gray-brown		
3							11.5	
		ML	Sandy Silt, very fine cemented	hard			7.9	
4			Maximum depth 4 feet. No ground water encountered.					
5								
6								
7								

TEST PIT NO. 4

Logged by JAD

Date: 11/16/17

Elevation: 245'

Depth	Blows	Class.	Soil Description	Consistency	Moisture	Color	W(%)	Comments
0		OL	Duff/Topsoil	loose	moist	dk brn		
1		SM/ ML	Silty Sand, very fine/Sandy Silt		to very moist	brown	19.7	
2				m. dense				
3		SM	Silty Sand, fine with gravel to 6"	very dense	moist	moist gray-brown		
		SM/ GM	Gravelly Silty Sand				7.9	
4			Maximum depth 4 ft. No ground water encountered.					
5								
6								
7								

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	Proj. No. 17-114	Date 11/17	Figure A-2