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.

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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

<u>Surface Conditions</u>

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.

<u>Pavement and Slab Subgrade Preparation:</u> 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.

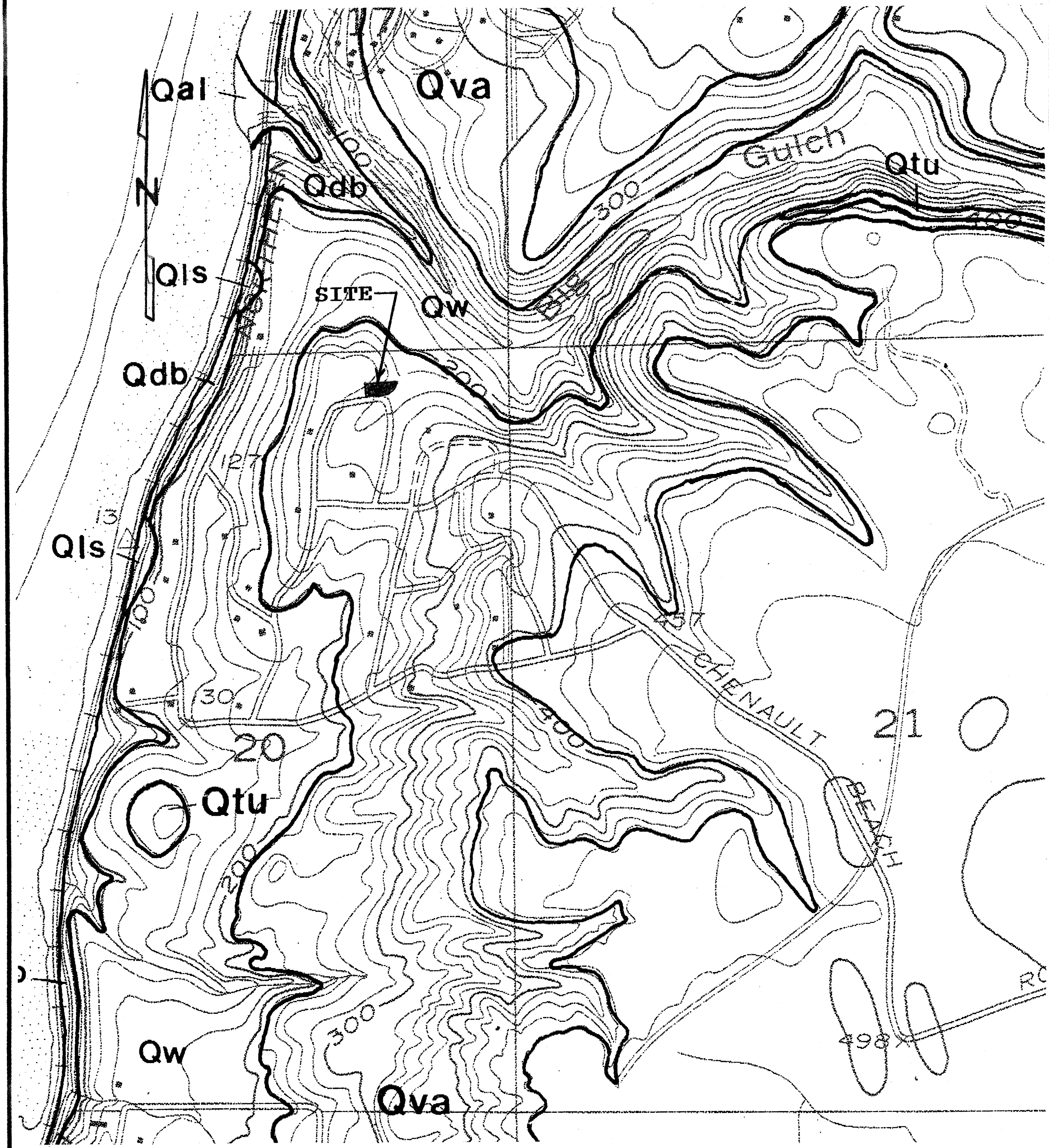
James A. Doolittle
Principal Engineer

JAMES A. DOOLITTLE
STATE OF WASHINGTON
23523
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 3/19/ 2018

11/27/17

Encl: Figures 1 through 4
Appendix A

Dist: 1/Addressee via email

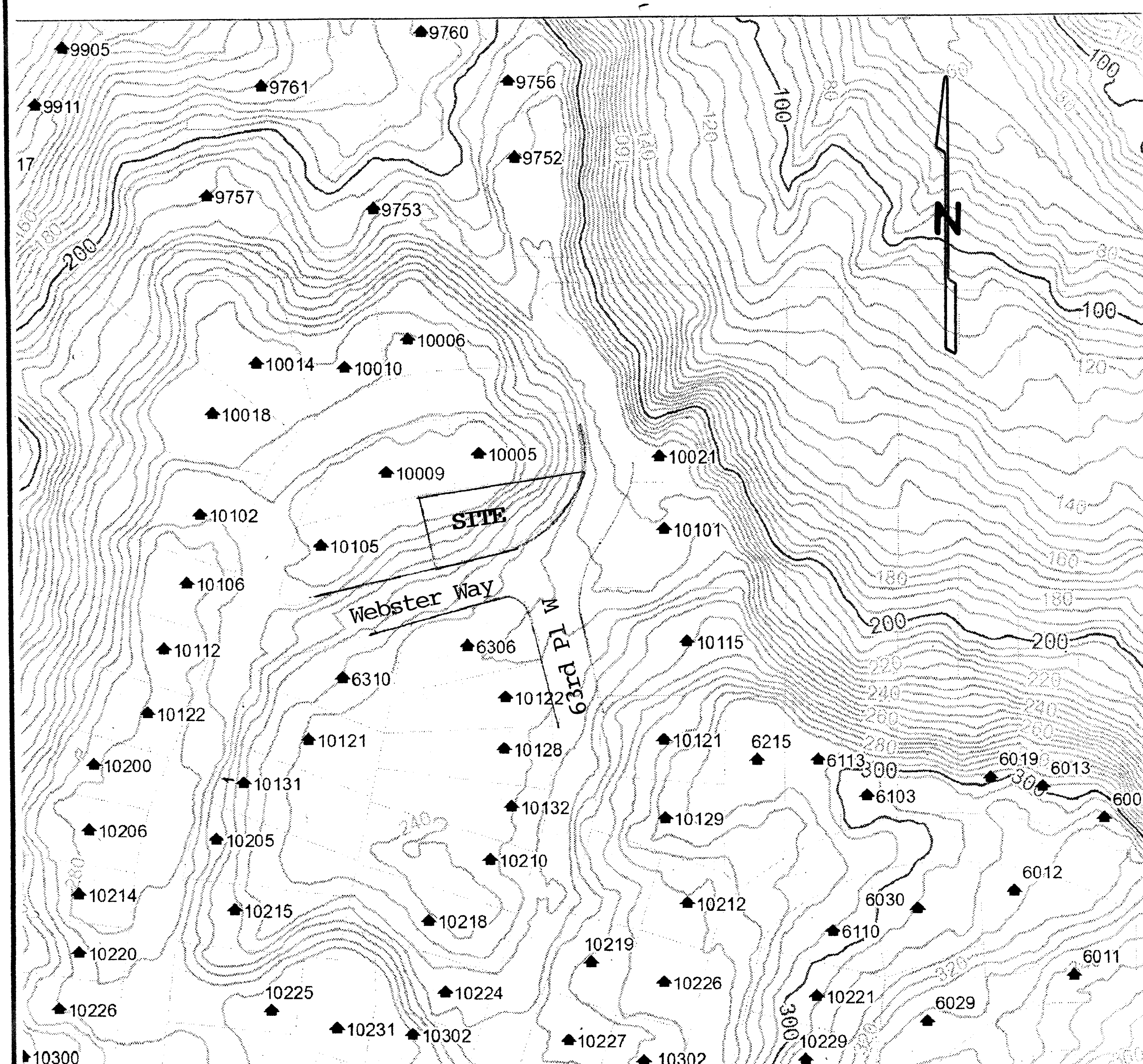


ref: Distribution and Description of Geologic Units, in the Mukilteo Quadrangle, Washington, USGS Map MF-1438, by James P. Minard, 1982, Enlarged Scale: 1"= 1000'

SITE VICINITY GEOLOGIC MAP

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 1



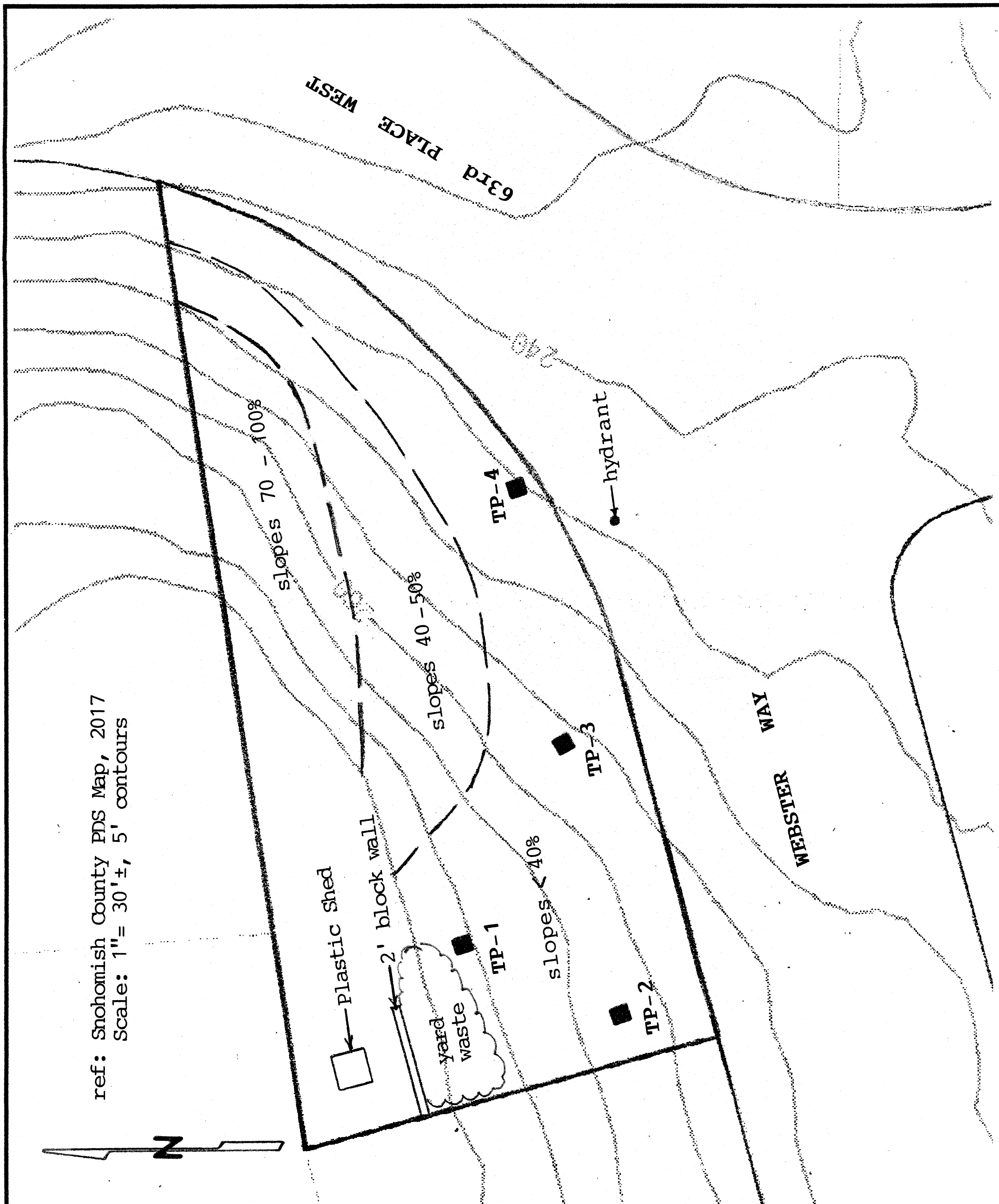
ref: Snohomish County PDS Map, 2017
Scale: 1"= 200'±, 5' contours

SITE AREA TOPOGRAPHY

GEOSPECTRUM CONSULTANTS, INC.
Geotechnical Engineering and Earth Sciences

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

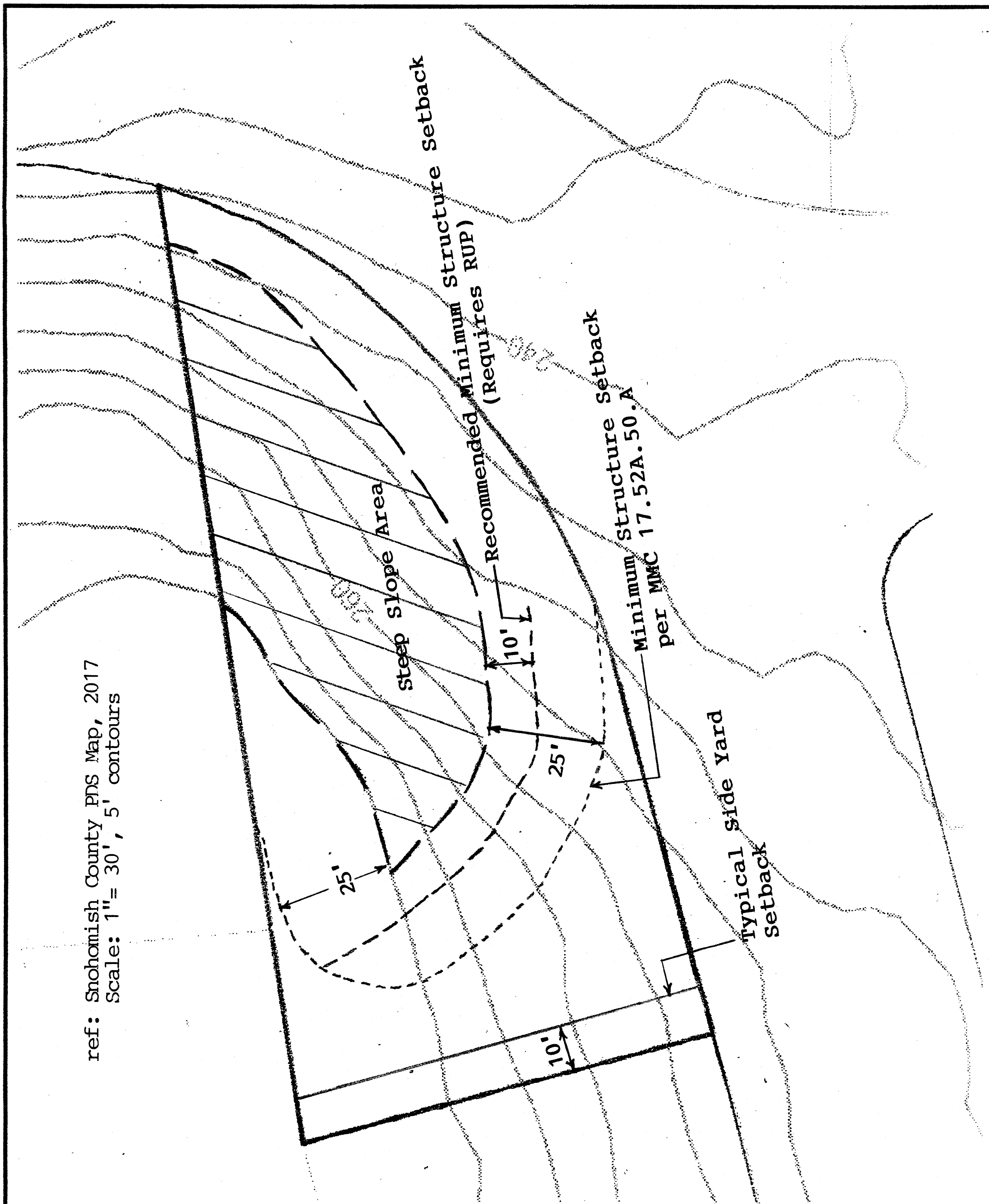
Proj. No. 17-114	Date 11/17	Figure 2



SITE CONDITIONS AND EXPLORATION PLAN

GEOSPECTRUM CONSULTANTS, INC. *Geotechnical Engineering and Earth Sciences*	Residential Property Development SCPN 00408600400,100XX 63rd Pl West Mukilteo, Washington

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



MINIMUM STRUCTURE-SLOPE SETBACKS

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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 very moist	brown		
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 very moist	brown	14.6	
2				medium dense		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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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								
5			Maximum depth 4 feet. No ground water encountered.					
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	7.9	
		SM/ GM	Gravelly Silty Sand					
4								
5			Maximum depth 4 ft. No ground water encountered.					
6								
7								

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