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Geotechnical Evaluation Report

Parcel#28040300203200, Mukilteo, Snohomish County, Washington

Date: April 18, 2018
Project No: 0178-WA17

April 18, 2018

PROJECT NO: 0178-WA18

CLIENT: Mr. Kevin Richardson
Mukilteo Ridge Home Owner's Association

Reference: Landslide Evaluation Report, Parcel # 28040300203200, Mukilteo, Snohomish County, WA

Dear Mr. Richardson,

At your request, we have completed the above referenced services for the referenced project in accordance with the American GeoServices, LLC (AGS) proposal and your authorization-to-proceed. Results of our evaluation and design recommendations are described below.

PROJECT INFORMATION

The site is located as shown in Figure 1 and Figure 2. The subject site is an area of interest located within the portion of Snohomish County Tax Parcel # 28040300203200 located in Mukilteo, Washington. The Tax Parcel is an irregular shaped, 6.32 acre lot primarily consisting of designated wetlands that abuts an existing residential development to the west. In general, the surrounding terrain slopes down from the southwest to the adjacent unnamed creek. The area of interest is the creek bluff located approximately 150 feet south of the east end of Deboralon Ln. and roughly 25 feet east of the existing unnamed paved road. A pump house presently occupies the top of bluff and the surrounding ground surface is covered with grass to the slope break. The slope itself is covered with fast growing vegetation and vines such as blackberry bushes.

We were retained by the Mukilteo Ridge Homeowners Association to assess the bluff just east of the pump house because the adjacent slope had failed. At the time of our site visit, the failed slope was covered with visqueen plastic to protect the exposed slope soils from water softening due to precipitation and storm water runoff. The failure surface was close to vertical for 14 feet in height and then the debris sloped moderately to the creek for another 10 horizontal feet. The slide was approximately thirty feet wide and "U" shaped from aerial view.

An approximately 12-inch diameter pipe extended from the failure surface for a distance of about 10 feet. The pipe appeared to possibly drain a detention basin located to the east of the unnamed paved road.

See Figures 2A and 2B for surface conditions and a generalized cross-section of the site. See attached photographs included in an appendix for further details on the surficial site conditions.

SCOPE OF WORK

Our scope of work included following specific items:

- Detailed site reconnaissance to evaluate surface conditions, and slope / landslide characteristics.
- Review of available reports and literature on soils, geology, natural hazards, and USGS maps along with local GIS mapping to evaluate geologic hazards and earthquake/seismic hazards. These include slope instability, ancient and recent landslides, active or inactive landslides, erosion, slope stability related issues, liquefaction, seismically induced slope instability, and lateral foundation stability for seismic conditions.
- Surface and subsurface soils/bedrock and groundwater/drainage conditions using soil auguring and Williamson drive probes. We performed two soil borings/ explorations/drive probes (B1 and B2) to evaluate subsurface soil types, consistencies and relative densities and to recommend most suitable area for proposed construction. We noted groundwater levels during exploration and at the completion of exploration. We will also review available literature to evaluate seasonal groundwater conditions in the site vicinity area. Prior to the beginning of exploration, we reviewed any information on existing on-site utilities provided by you. At the completion of exploration, boring locations were backfilled with soil cuttings and sealed at the top. All soil samples were identified in the field and were placed in sealed containers and transported to the laboratory for further testing and classification.
- Data obtained from site observations, limited subsurface exploration, laboratory evaluation, and previous experience in the area was used to perform engineering analyses. Results of engineering analyses, including slope stability analyses and retaining wall designs were then used to reach conclusions and recommendations presented in this report.
- Using the collected soil samples, we performed laboratory soil evaluation which included soil classification.
- We prepared this report providing conclusions and recommendations on site stability conditions.

SUBSURFACE CONDITIONS

Soil classification and identification is based on commonly accepted methods employed in the practice of geotechnical engineering. In some cases, the stratigraphic boundaries shown on Boring Logs represents transitions between soil types rather than distinct lithological boundaries. It should be recognized that subsurface conditions often vary both with depth and laterally between individual boring locations. The following is a summary of the subsurface conditions encountered at the site:

One borehole (B1) was drilled at the top of the slope extending to a depth of 25 feet. We also performed a hand augured borehole and a Williamson Drive Probe (WDP) penetration testing at the bottom of the slope upstream of the slide area. This apparatus was chosen due to accessibility issues. The WDP literature is included in an appendix.

Native Sand-Silt Mixtures: The site is primarily underlain by about 11 feet of medium dense sand-silt mixtures (SP, SM,SP/SM) followed by generally dense mixtures of sands, silts, and gravel extending to the maximum explored depth of 25 feet. These soils most likely represent locally known advance outwash deposit, as noted in Figure 3. It appears Vashon till is not present at the site.

Groundwater: Groundwater seepage was encountered during exploration at a depth of 18 feet at borehole location B1. In our opinion, perched groundwater conditions or wet soil conditions may exist at higher depths during heavy rains, throughout the site. This observation may not be indicative of other times or at locations other than the site. Some variations in the groundwater level may be experienced in the future. The magnitude of the variation will largely depend upon the duration and intensity of precipitation, temperature and the surface and subsurface drainage characteristics of the surrounding area.

GEOLOGIC HAZARDS EVALUATION

Based upon the results of our site exploration, engineering analysis, and literature review of following documents, we evaluated geologic hazards at the site.

- Washington Division of Geology and Earth Resources, Open File Reports
- U.S. Geological Survey Geologic Maps
- Snohomish County GIS
- City of Mukilteo GIS
- Soil Survey Maps, USCS
- Washington Geologic Information Portal, USGS

Landslides: Our review of available geologic maps and landslide hazard maps did not indicate that prior landslides had occurred at the site, and the site was not located within the existing known slide area. However, as shown in the attached landslide hazard map (Figure 5), the site is located in the area close to the area mapped as having moderate to high susceptibility for shallow as well as deep-seated landslides. During our site reconnaissance, shallow landslide features were mapped within the site boundary area as discussed earlier and as shown in attached photographs. Our site reconnaissance also revealed significant potential for slope failures, shallow slumps, or severe erosion at the site.

Sandy soils present in the site vicinity area are highly susceptible to erosion. Many of the nearby slopes appear to have been eroded over time. The failed slope may have received higher amounts of runoff due to its location (it might be in a natural drainage depression) and it may be collecting additional runoff from the pump house and the roadway (the road appears to be closest to the slope at this point and may be directing runoff in that direction). It may also be possible that the pipe exiting the failure surface could have leakage.

In our opinion, the site is located in landslide susceptible area and the site is underlain by soils and geologic conditions that are susceptible landslides and severe erosion. Historically, with construction and man-made drainage features in such areas, there is an inherent risk associated with ground movement. Although there was no global or local landsliding observed affecting the existing pump house building envelope at this time, the owner is completely responsible for all risks associated with any future potential for instability of any structures at the site, especially related to the existing pump house building or pipe structures. The potential for future global slope failure associated with movement of the global/ancient landslide and related local slope movement is high and the owner is responsible for any risks associated with any future potential for instability at the site or in the site vicinity. It should be noted that the detailed evaluation of the impact of any ancient or global landslides at the site or in the site vicinity area was beyond our scope of services. We highly recommend performing a detailed landslide hazards evaluation (related to global landslides) of the existing pump house on the site, especially the existing setbacks from steep slopes.

Slope Stability Evaluation: Using the results of subsurface exploration, and available survey data, we analyzed on-site slopes by performing preliminary slope stability analyses. We modeled existing slopes for global and local stability using SLOPE/W computer software, and performed analyses using several analytical methods (Bishop, Janbu, Spencer, etc.) to obtain the lowest factor of safety against slope failures. The SLOPE/W computer software calculates the most likely failure plane based on topography, subsurface conditions (including soil parameters), and groundwater conditions. The stability of the most likely failure plane is calculated as the factor of safety (FOS), which is a ratio of the resisting forces or shear strength to the driving forces or shear stress required for equilibrium of the slope. A FOS of 1.0 indicates the resistive forces and driving forces are equal. A FOS below 1.0 indicates the driving forces are greater and the landslide is active. A FOS above 1.0 indicates the resisting forces are greater and the slope is stable. Based on the engineering community and our experience, a factor of safety above 1.5 is generally acceptable to assure slope stability in commercial/residential applications.

We analyzed typical cross-sections to determine the impact of proposed retaining wall for landslide stabilization on existing slopes at the site and in the immediate vicinity of the site boundaries and concluded that the use of a gravity retaining wall would be required to minimize the potential for slope instability. Considering the site accessibility, size of the project, and wall height, we recommend the gabion retaining wall for landslide and/or slope stabilization. We analyzed typical cross-sections to determine the impact of proposed gravity retaining wall on existing overall site stability. Based on the results of our initial slope stability evaluation, we concluded that, under normal conditions, the site would remain stable with a FOS value of at least 1.5 under static conditions and FOS values in the range of 1.1-1.2 under seismic conditions.

Results of our slope stability analyses revealed that a minimum slope setback of 15 feet is required for the pump house foundations to avoid future instability. At present, there is a minimum setback of only 4 feet. Therefore, the existing foundations should be either underpinned adequately, or, a retaining wall should be built at the base of the slope to create an adequate setback for the pump house.

We recommend that our services be retained to design the gabion retaining wall, to design foundation underpinning (if required), to perform continuous monitoring of the wall installation and to approve the final installed wall so that the implementations of our recommendations can be confirmed or revised depending upon the conditions encountered during wall installation.

Earthquake Related Hazards: The following paragraphs describe potential earthquake related hazards that are known to exist within most of the northwestern United States.

Earthquakes in the Pacific Northwest occur due to tectonic activity associated with the subduction of the Juan de Fuca Oceanic plate beneath the North American Continental plate. The Juan de Fuca plate is converging on and thrusting beneath the North American Continental plate along the Cascadia Subduction Zone (CSZ), which is situated offshore along Washington. This convergence along the CSZ is the source of three types of earthquakes in western Washington. These are (1) deep intraplate earthquakes originating in the Juan de Fuca plate, (2) large subduction zone-interplate earthquakes that may occur along the interface between the Juan de Fuca and the North American Plates, and (3) shallow crustal earthquakes generated along faults.

Most of the intraplate earthquakes have occurred within the Puget Sound region. The estimated maximum magnitudes of CSZ intraplate earthquakes are in the range of M7.0 to M7.5.

Available research indicates that there is a potential for a large subduction zone earthquake near the Washington coast. To interpret earthquake potential of the CSZ plate interface, geologic lines of evidence such as coastal subsidence, stratigraphic evidence for flooding associated with earthquakes and turbidity in the ocean have been used. Based on the available geologic evidence, there is a sufficient scientific consensus to consider the CSZ plate interface as a potential earthquake source. The estimated maximum magnitudes of CSZ interplate earthquakes are in the range of M8.0 to M9.0+. The estimated recurrence interval is 350 to 500 years.

Crustal earthquakes are generally concentrated above a depth of approximately 10 to 20 km. Based on our literature review, the estimated maximum magnitudes of these crustal earthquakes are in the range of M6.0 to M6.5.

Based on site geology, topography, and our preliminary evaluation, in our opinion, the site may be susceptible to severe ground shaking and significant landsliding during a major earthquake. Ground acceleration more than 0.35g may occur at the site. As mentioned above, it should be noted that most of the northwestern United States is susceptible to similar earthquake-related hazards. A detailed site-specific seismic evaluation of any kind was beyond the scope of this report.

Based on the results of our subsurface explorations and review of available literature (2009 International Building Code), in our opinion, a site classification “C” and a design PGA of 0.35 may be used for this project. However, this site classification may be revised by performing a site-specific shear wave velocity study. The 1 Hz spectral acceleration with 2% probability of exceedance in 50 years is 120-140% g.

Subsurface soil conditions at the site are not susceptible to liquefaction. Seismically induced slope instability most likely will occur on a localized; however, such an evaluation was beyond our scope of services. A detailed seismic hazards evaluation of the site was beyond our scope of services. We recommend that a detailed seismic hazards evaluation should be performed for this site and the site should be stabilized for all potential hazards related to seismic conditions, especially the slope instability under seismic conditions.

CONCLUSIONS

Based on the results of our geotechnical evaluation, the shallow landslide occurred at the site due to one or a combination of the following factors:

- Inadequate drainage conditions and heavy rainfall causing significant surface and near-surface water run-off leading to soil saturation
- Presence of sand-silt loam and local geology and topography that is susceptible to landsliding and slope instability
- Site location and topography marked as having moderate to high slope stability hazards
- Slope steepness without adequate lateral support
- Sandy soils present in the site vicinity area highly susceptible to erosion. Many of the nearby slopes appear to have been eroded over time. The failed slope may have received higher amounts of runoff due to its location (it might be in a natural drainage depression) and it may be collecting additional runoff from the pump house and the roadway (the road appears to be closest to the slope at this point and may be directing runoff in that direction). It may also be possible that the pipe exiting the failure surface could have leakage.

RECOMMENDATIONS

We make following recommendations for stabilizing the landslide area and for making site improvements.

- Stabilize the landslide area using a gravity gabion retaining wall. The gabion retaining wall should be designed using the geotechnical design and construction recommendations given in the following paragraphs.
- Develop a drainage plan to collect all the surface water run-off and near-surface run-off towards the site and to discharge into suitable receptacle. Install adequate drainage piping and/or repair to assure that no storm water discharge occurs towards the proposed retaining wall and the steep slopes present at the site near the pump house. As a minimum, install an interceptor drain along the uphill property line. The interceptor drain should be 24 inches deep and 16 inches wide with drain rock in a filter fabric envelope with a 4-inch diameter drain pipe discharged into suitable receptacles so that onsite as well as offsite slope stability is not adversely impacted.
- Provide adequate erosion protection on the exposed slope surfaces to avoid further erosion. Erosion protection measures should be integrated into the recommended gabion wall design. If riprap or crushed rock fill is used as backfill behind the retaining wall, the erosion protection blanket may not be required.
- Perform leakage investigation of the existing piping that runs through the slope and the slide area, and fix all the leaks to restore and maintain integrity of all the piping.

- Results of our slope stability analyses revealed that a minimum slope setback of 15 feet is required for the pump house foundations to avoid future instability. At present, there is a minimum setback of only 4 feet. Therefore, the existing foundations should be either underpinned adequately, or, a retaining wall should be built at the base of the slope to create an adequate setback for the pump house. We recommend performing foundation evaluation of the existing pump house and provide necessary underpinning and/or stabilization measures to assure long-term stability of the structure. Use following soil design parameters for foundation stability analyses and mitigation design.

GEOTECHNICAL DESIGN and CONSTRUCTION RECOMMENDATIONS

Based on our evaluation, we recommend following design parameters for wall design and foundation stabilization analyses and design.

- Granular backfill friction angle = 40 degrees.
- Granular backfill cohesion = 0 psf.
- Granular backfill unit weight = 125 pcf.
- Retained soil friction angle = 28-30 degrees.
- Retained soil cohesion = 100 -125 psf.
- Retained soil unit weight = 115 pcf.
- Foundation soil friction angle = 34 degrees.
- Foundation soil cohesion = 100 psf.
- Foundation soil unit weight = 115 pcf.
- Minimum depth of embedment for gabion wall = 36 inches.
- Minimum depth of embedment for underpinning elements for the pump house = 20 feet
- No hydrostatic pressure on the wall. To achieve this condition, proper surface and subsurface drainage should be provided at and around all wall locations. Additional drainage pipes should be installed as shown on the attached wall construction plans.

We provide the following general recommendations for the wall construction.

- Construction should be preferably performed during summer or dry season of the year. If not, then specific fill materials per geotechnical engineer's recommendations may be used. We recommend continuous and periodic inspections by American GeoServices, LLC representative during predrilling, pile installation and fill placement operations to assure proper wall construction.

- All wall foundation subgrades and any exposed cuts at wall locations must be evaluated and approved by a registered geotechnical engineer from our office for estimated soil design parameters.

- Normal vehicular surcharge load during and after construction must not exceed 250 psf.

- Proper installation of drainage pipe behind the wall and drainage control after the completion of wall construction is important. Under no conditions should this wall drainpipe be connected to house drainage system. As a minimum, all new building downspouts should be drained away from wall areas or discharged into suitable receptacle to avoid ponding near the wall base. All pavement or toe areas should be drained away so that drainage towards wall areas is minimized.

GENERAL DRAINAGE RECOMMENDATIONS

All drainage systems must be maintained leak-free. Proper surface and subsurface drainage is critical for long-term performance of the retaining structures. In general, proper surface drainage should be maintained at this site. Irrigation should be minimal and limited to maintain plants. Roof downspouts should discharge on splash-blocks or other impervious surfaces and directed away from the proposed retaining wall and steep slopes. Ponding of water should not be allowed immediately adjacent to the proposed retaining wall area.

It is important to follow these recommendations to minimize settling or movement of the retaining wall elements throughout the life of the facility. Construction means and methods should also be utilized which minimizes saturation of soils during construction. Again, positive drainage away from the new structures is essential to the successful performance of retaining walls, and should be provided during the life of the structure. Downspouts from all roof drains, if any, should cross all backfilled areas such that they discharge all water away from the backfill zones and structures.

Drainage pipes installed behind the proposed retaining wall, should be discharged into suitable receptacles without adversely impacting the on-site and off-site stability.

GENERAL CONSTRUCTION RECOMMENDATIONS

Subgrade Preparation: In general, we recommend that any surface water within construction areas be drained away by cutting drainage ditches or by pumping from a sump hole, if necessary. Surface vegetation including topsoil, any saturated/inundated and disturbed soil, and any non-soil or incompetent materials encountered at the time of construction should be removed. If any deep root systems or tree trunks are removed, then the excavated areas should be filled with densely compacted on-site silt soil or imported crushed rock.

In wet season, to protect moisture sensitive soils during construction activities, a 3-inch to 6-inch thick crushed rock layer should be placed immediately on any exposed subgrades after site grading and topsoil removal. For construction truck traffic areas, at least 12-inch thick granular working base is generally recommended with thicker sections and/or geotextile fabrics for heavily traveled areas.

Fill Placement: Granular backfill materials for the proposed retaining wall should be placed in layers that, when compacted, do not exceed 12 inches. At your request, depending upon weather conditions during construction, we may provide specific fill recommendations, especially for wet weather conditions.

Excavation/Cuts and Dewatering: In general, excavations should be performed in accordance with Department of Labor Occupational Safety and Health Administration (OSHA) guidelines for Type C soils. Deeper excavations may be excavated at grade steeper than the recommended OSHA grades provided the excavations are monitored and certified by a qualified geotechnical engineer. Please note that site safety is the sole responsibility of the project contractor and/or the owners.

The use of a standard excavator may be adequate for this site. Groundwater seepage in excavations should be anticipated during wet season of the year. For most of the excavations on this project, pumping from sumps outside the limits of the excavation should control groundwater seepage and surface water ponding.

Soils exposed in excavated areas should be protected from rain, freezing, and excessive loading along edges. Surface water run-off should be intercepted and drained away from excavated areas. Ideally, in structural areas, concrete should be poured within 24 hours of the completion of excavation.

Wet Weather Construction: In our opinion, the site is not suitable for wet weather construction. Earthwork done during summer months will be most likely more economical. In any case, during construction in wet or cold weather, grade the site such that surface water can drain readily away from the building areas. Promptly pump out or otherwise remove any water that may accumulate in excavations or on subgrade surfaces and allow these areas to dry before resuming construction. Berms, ditches and similar means may be used to prevent storm water from entering the work area and to convey any water off-site efficiently. Wet weather construction will require the implementation of best management erosion and sedimentation control practices to reduce the chances of off-site sediment transport, including but not limited to covering the excavated slopes with plastic sheets, using silt fences, bales of straws, and prompt subgrade preparation and concrete pour.

All excavations during wet weather should be covered with plastic sheeting and adequate drainage should be provided to avoid cut/excavation instability due to soil saturation. It is important to understand that, if proper precautions are not taken, sudden cut or excavation failures can occur without warning during wet weather, which can be fatal.

LIMITATIONS

Historically, with construction in sloping areas such as the site vicinity, there is an inherent risk associated with ground movement and/or settlements and related structural damage due to osil movement. Therefore, the owner is completely responsible for taking all risks associated with any future potential for instability at the site or in the site vicinity. Although there was no global slope instability observed in the immediate vicinity of the site, the potential for future slope failure associated with movement of the global/ancient landslide and related local slope movement is low and the owner is responsible for any risks associated with any future potential for instability at the site or in the site vicinity. It should be noted that the detailed evaluation of the impact of any ancient or global landslides in the site vicinity area was beyond our scope of services.

Recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory evaluation, and our present knowledge of the proposed construction. It is possible that soil conditions could vary between or beyond the points explored. If soil conditions are encountered during construction that differ from those described herein, we should be notified so that we can review and make any supplemental recommendations necessary. If the scope of the proposed construction, including the proposed loads or structural locations, changes from that described in this report, our recommendations should also be reviewed and revised by AGS.

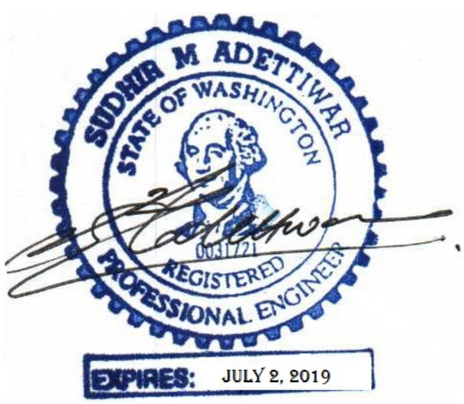
Our Scope of Work for this project did not include research, testing, or assessment relative to past or present contamination of the site by any source. If such contamination were present, it is very likely that the exploration and testing conducted for this report would not reveal its existence. If the Owner is concerned about the potential for such contamination, additional studies should be undertaken. We are available to discuss the scope of such studies with you. No tests were performed to detect the existence of mold or other environmental hazards as it was beyond Scope of Work.

Local regulations regarding land or facility use, on and off-site conditions, or other factors may change over time, and additional work may be required with the passage of time. Based on the intended use of the report within one year from the date of report preparation, AGS may recommend additional work and report updates. Non-compliance with any of these requirements by the client or anyone else will release AGS from any liability resulting from the use of this report by any unauthorized party. Client agrees to defend, indemnify, and hold harmless AGS from any claim or liability associated with such unauthorized use or non-compliance.

In this report, we have presented judgments based partly on our understanding of the proposed construction and partly on the data we have obtained. This report meets professional standards expected for reports of this type in this area. Our company is not responsible for the conclusions, opinions or recommendations made by others based on the data we have presented. Refer to American Society of Foundation Engineers (ASFE) general conditions included in an appendix.

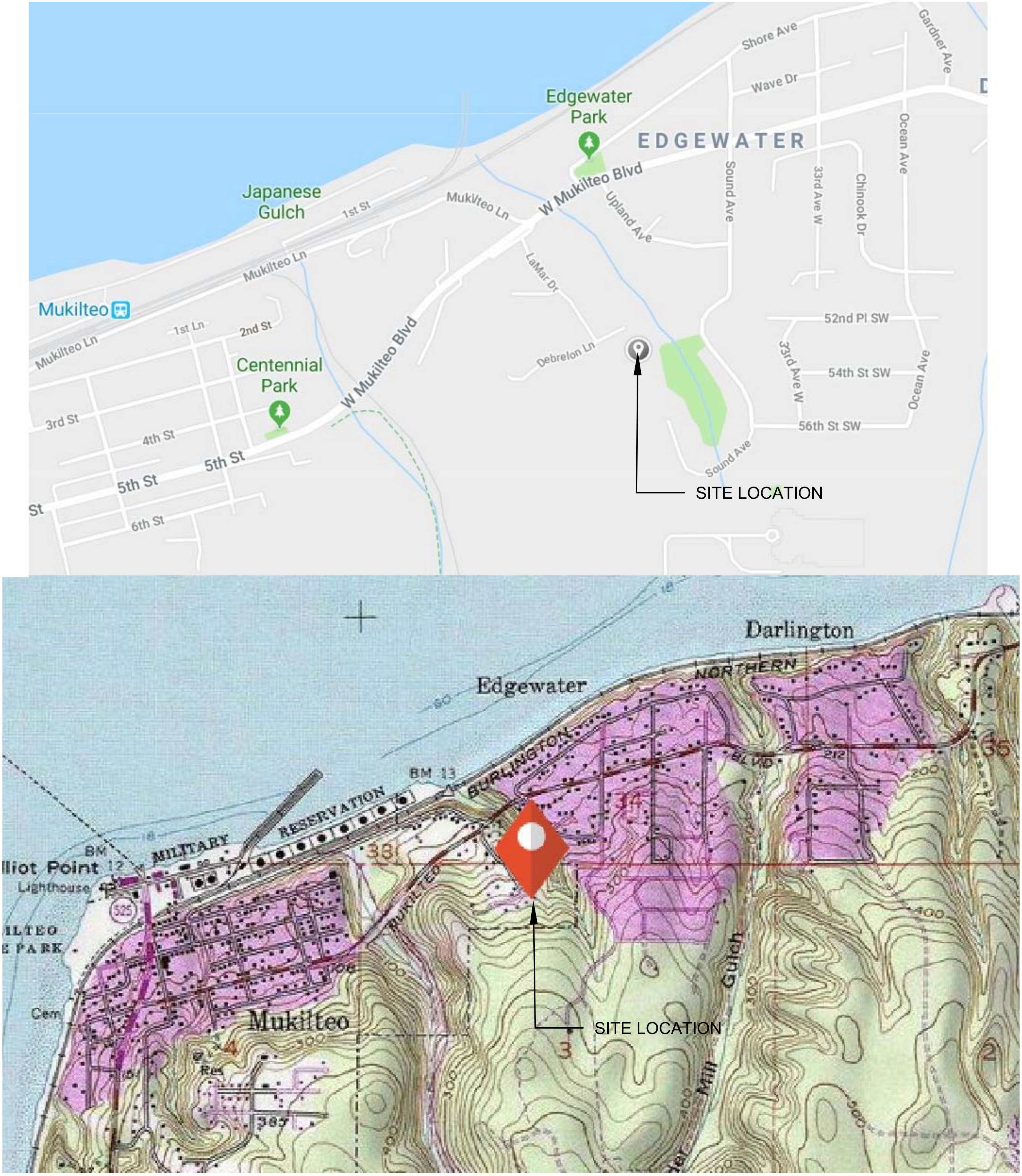
This report has been prepared exclusively for the client, its' engineers and subcontractors for design and construction of the proposed structure. No other engineer, consultant, or contractor shall be entitled to rely on information, conclusions or recommendations presented in this document without the prior written approval of AGS. We appreciate the opportunity to be of service to you on this project. If we can provide additional assistance or observation and testing services during design and construction phases, please call us at 1 888 276 4027.

Sincerely,



Sam Adettiwar, MS, PE, GE, P. Eng, M. ASCE
Senior Engineer
Attachments

FIGURES



REFERENCE:
GOOGLE MAPS
USGS TOPO MAP

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FIGURE 1: SITE LOCATION MAP

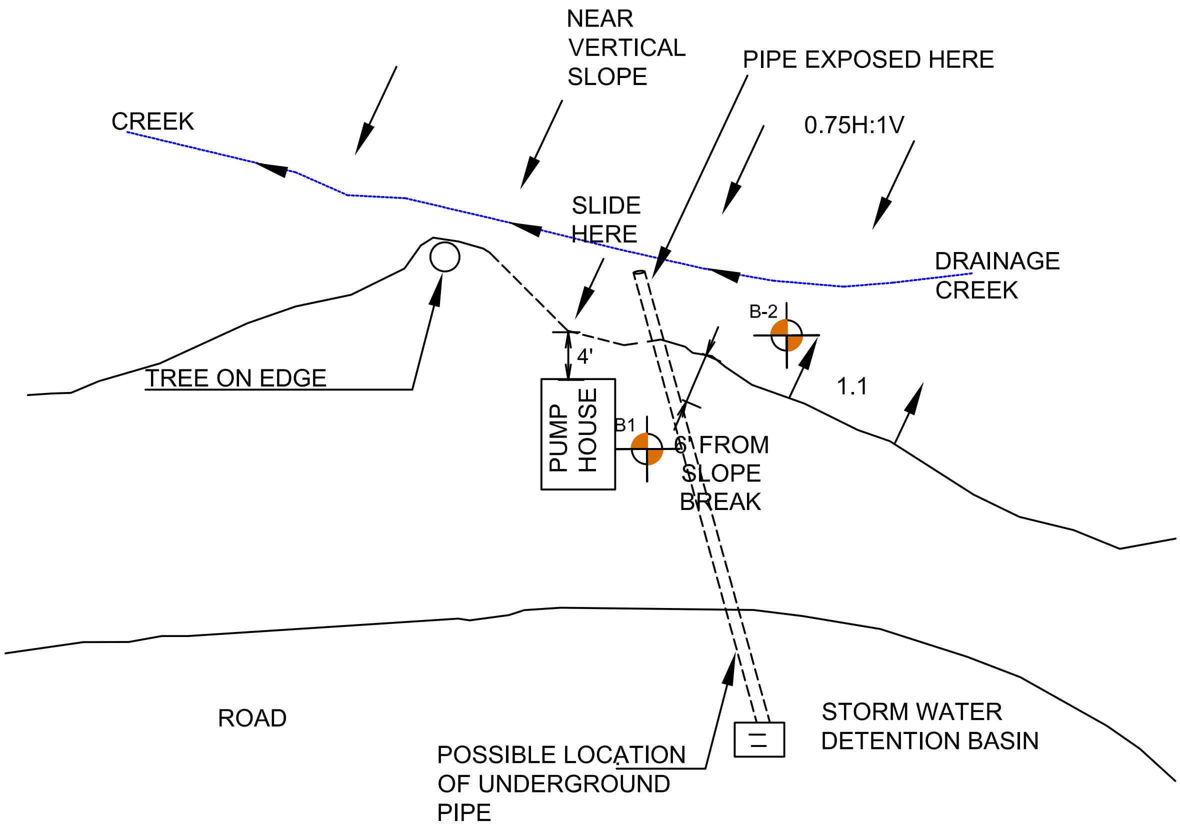

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FIGURE 2A: SCHEMATIC SITE PLAN

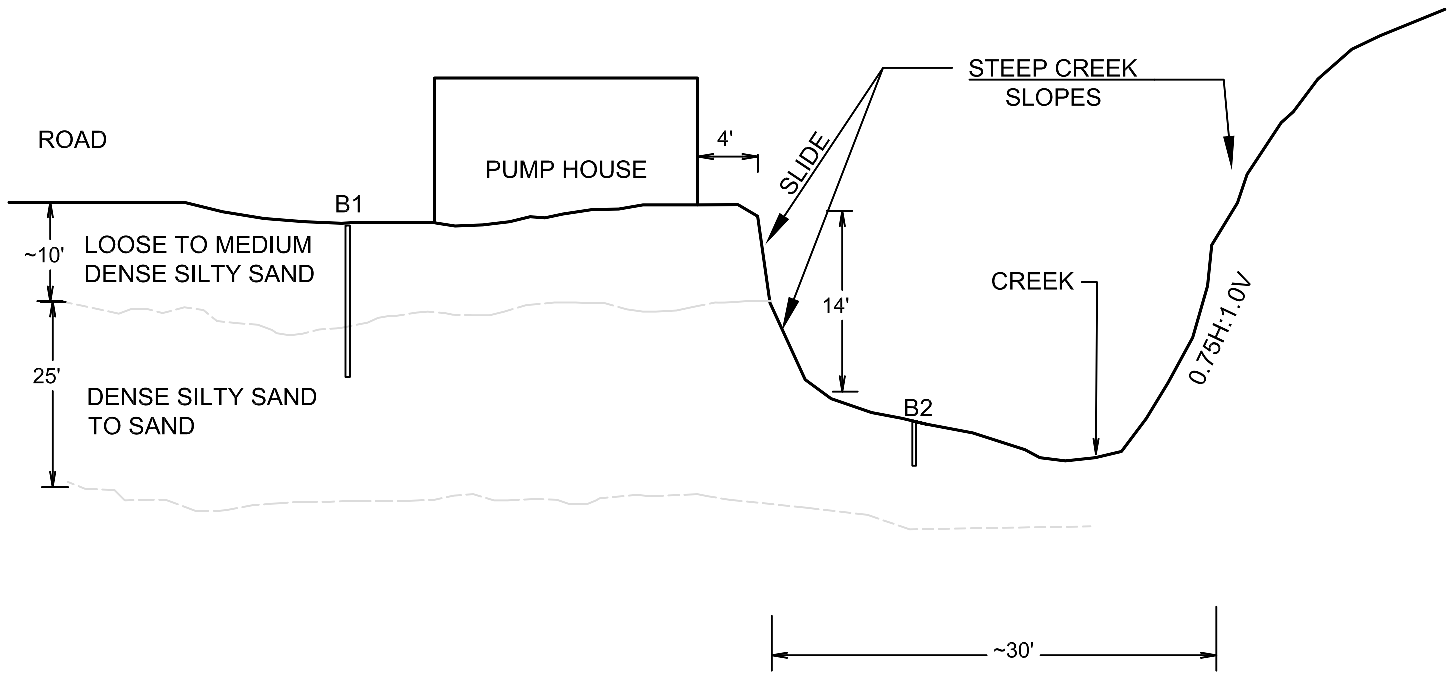

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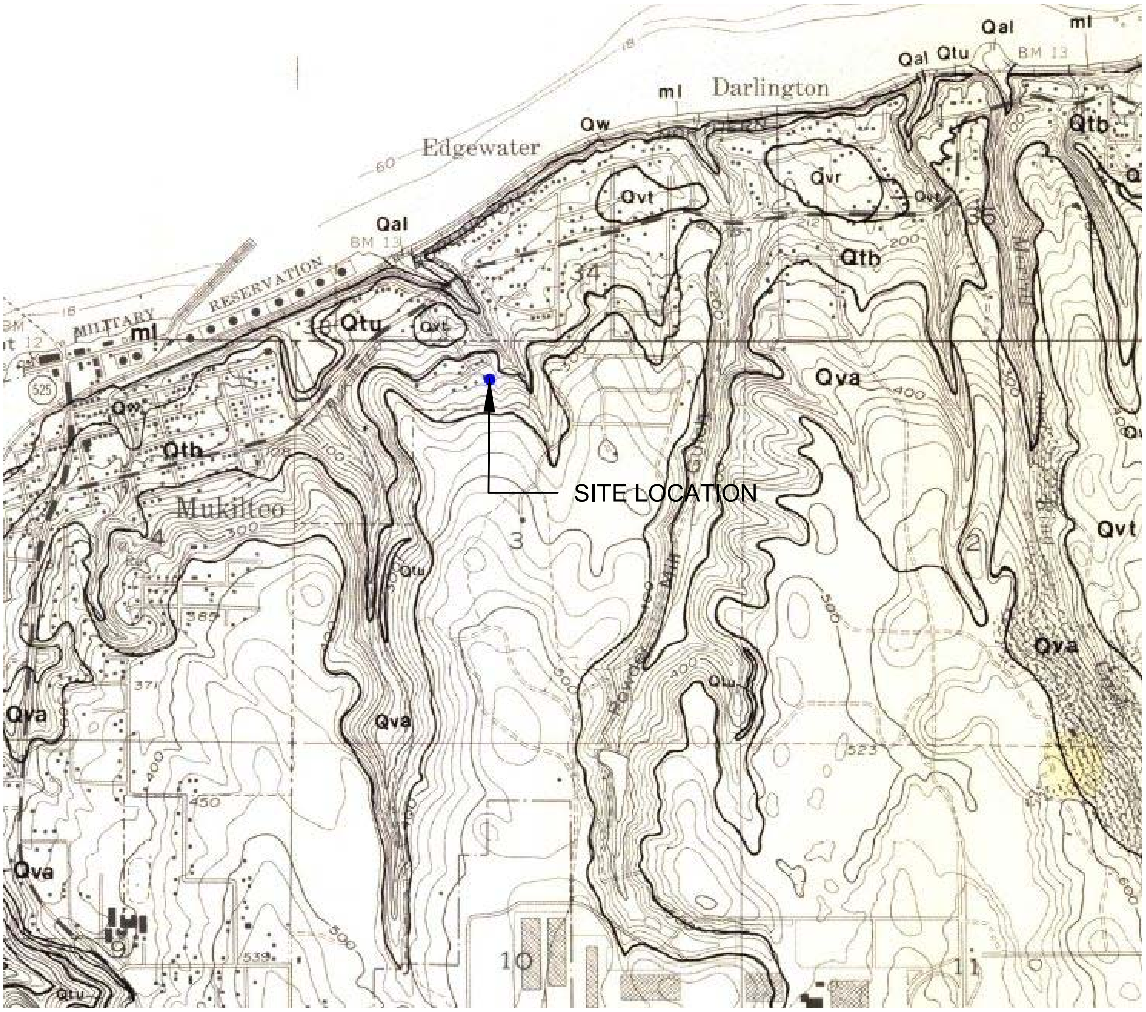
FIGURE 2B: GENERALIZED CROSS-SECTION



LEGEND

Qva ADVANCE OUTWASH (Fraser Glaciation) - Advance outwash underlies the till. The advance outwash typically is a thick section of mostly clean, gray, pebbly sand with increasing amounts of gravel higher in the section. Distinctive features of the outwash are its well developed horizontal and cross stratification, and cut and fill structures. Locally some of these sediments are stained by iron oxide precipitated from the ground water. Fine grained sand and some silt are common in the lower part of the unit and also locally occur sparingly in the upper part. The advance outwash was deposited by meltwater flowing from the advancing front of the glacier, partly, at least, as braided stream deposits. Some also was deposited as deltas in ponded water. Clast composition is varied[1]; the clasts were derived from adjacent older units and ice transported debris.

The sediments typically coarsen upward, partly because of local channel deposits, but also because of the increasing proximity of the advancing ice. The closer the ice, generally the steeper the gradient of the meltwater and the greater the capability to carry coarser material. This coarsening upward contrasts with the fining upward of the recessional outwash deposits, which is due to the withdrawal instead of advance of the sediment source.

Because the till overlies an irregular topographic surface formed on the advance outwash, different thicknesses of outwash are present in outcrop at different points. In many places the till completely overlaps the advance outwash downslope and rests directly on the next lower unit or units.

Because large expanses of the advance outwash have been exposed to subaerial weathering in slopes along the sound for a considerable amount of time, much of it has been oxidized to shades of brown in contrast to its usual gray color where protected by the overlying till.

The advance outwash underlies much of the upper slopes around the periphery of the till-covered uplands. It is as much as 80 m thick in the quadrangle. It is a source of excellent, clean sand and gravel and is one of the thickest and most extensive aquifers in the region. The unit occupies the stratigraphic interval of the Esperance Sand Member of Newcomb (1952, p. 40) and the Esperance Sand and advance outwash of Smith (1976). It is generally well drained and mostly provides a stable foundation where not close to a steep slope.

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FIGURE 3: GEOLOGIC MAP

LEGEND

Snohomish County Area, Washington (WA661)

Snohomish County Area, Washington (WA661)			
Map Unit Symbol	**Map Unit Name**	**Acres in AOI**	**Percent of AOI**
1	Alderwood gravelly sandy loam, 0 to 8 percent slopes	0.5	0.7%
3	Alderwood gravelly sandy loam, 15 to 30 percent slopes	7.2	8.7%
4	Alderwood-Everett gravelly sandy loams, 25 to 70 percent slopes	72.0	87.6%
5	Alderwood-Urban land complex, 2 to 8 percent slopes	2.5	3.0%
Totals for Area of Interest		**82.3**	**100.0%**

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REFERENCE:

WEB SOIL SURVEY

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FIGURE 4: SOIL SURVEY MAP

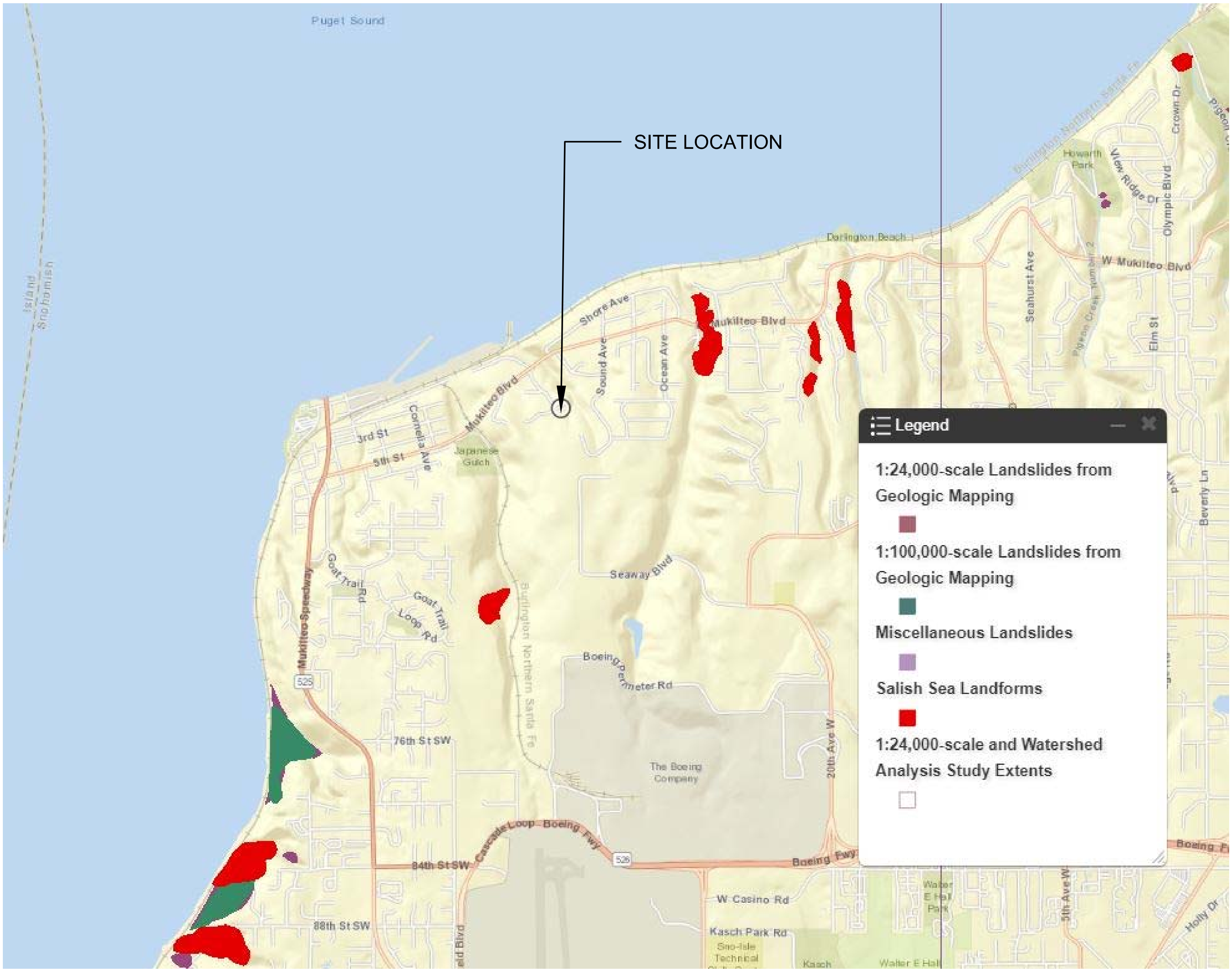

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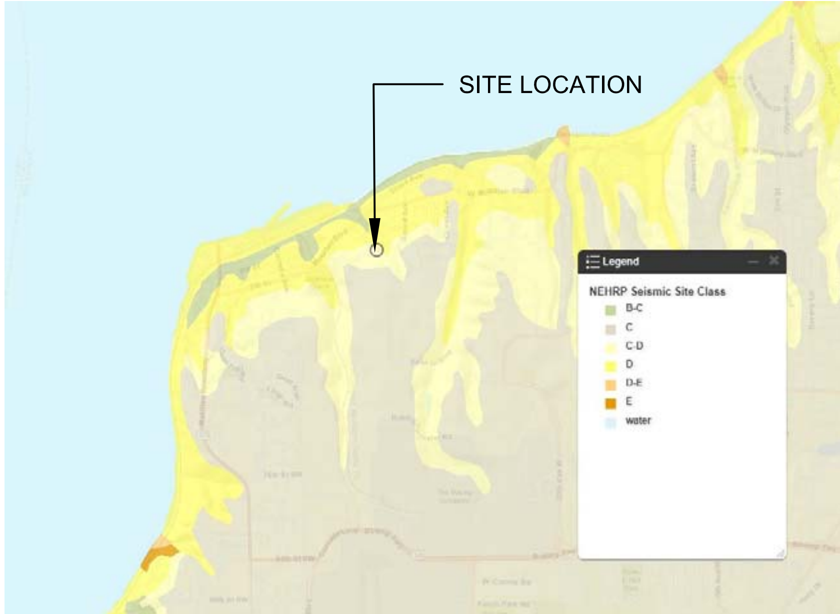
FIGURE 5: LANDSLIDE HAZARD MAP



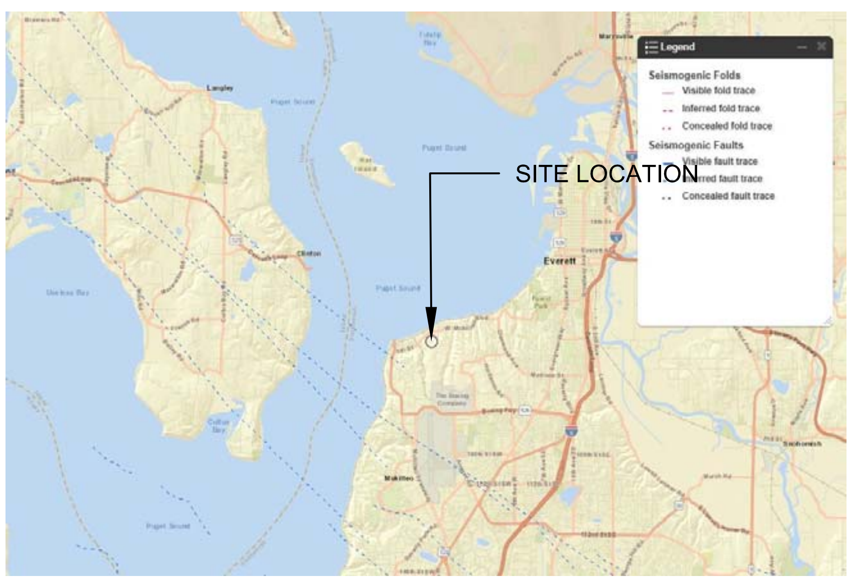


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FIGURE 6:EARTHQUAKE HAZARD MAP

APPENDIX

B1

Snohomish County Washington Parcel No. 28040300203200

Project Number	0178-WA18	Drill Rig: CME55 Solid Stem Auger, 4" Diameter	
Geologist/Engineer	SMA	Ground Elevation	See Figures
Date Drilled	3-24-2018	Total Depth of Borehole: 25 Feet	
Borehole Diameter	4 OD Inches	Depth to Water:	Not Encountered

Graphic Log	Description / Lithology	Depth (feet)	Sample	SPT Blow Count	Recovery (%)	Moisture (%)	DD (pcf)	LL (%), PL (%)	Swell (%)	Completion
SM	SILTY SAND, some gravel medium grain, light brown, loose to medium dense, moist	5	X	5-8-9	40					
SM/ SP	SILTY SAND to SAND, medium dense, gray, very moist	10	X	7-12-14	60					
SM/ SP	SILTY SAND to SAND, some gravel, dense, gray, very moist	15	X	15-12-18	20					
SP/ SM		20								
	End of Borehole at 25 feet. Groundwater was encountered at 18 feet. At completion, borehole was backfilled with soil cuttings.									

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B2

Snohomish County Washington Parcel No. 28040300203200

Project Number	0178-WA18	Drill Rig: Soil Auger & Williamson Drive Probe (WDP)	
Geologist/Engineer	SMA	Ground Elevation	See Figures
Date Drilled	3-24-2018	Total Depth of Borehole:	6 Feet
Borehole Diameter	4 OD Inches	Depth to Water:	Not Encountered

Graphic Log		Description / Lithology	Depth (feet)	Sample	SPT Blow Count	Recovery (%)	Moisture (%)	DD (pcf)	LL (%), PL (%)	Swell (%)	Completion
	SM	SILTY SAND, some gravel medium grained, light brown, loose to medium dense, moist to very moist		X	6-28-39	40					
	SP/ SM	More sand and gravel, dense below 3 feet	5								
		End of Borehole at 6 feet. Groundwater was not encountered during exploration. At completion, borehole was backfilled with soil cuttings. SPT blowcounts are derived from WDP corelations.	10 15 20								

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DESCRIPTIVE TERMINOLOGY & SOIL CLASSIFICATION
UNIFIED SOIL CLASSIFICATION SYSTEM

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size.)		
	Clean Gravels (Less than 5% fines)	
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	Gravels with fines (More than 12% fines)	
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
	Clean Sands (Less than 5% fines)	
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	SW	Well-graded sands, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with fines (More than 12% fines)	
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more of material is smaller than No. 200 sieve size.)		
SILTS AND CLAYS Liquid limit less than 50%	ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
	CH	Inorganic clays of high plasticity, fat clays
	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils

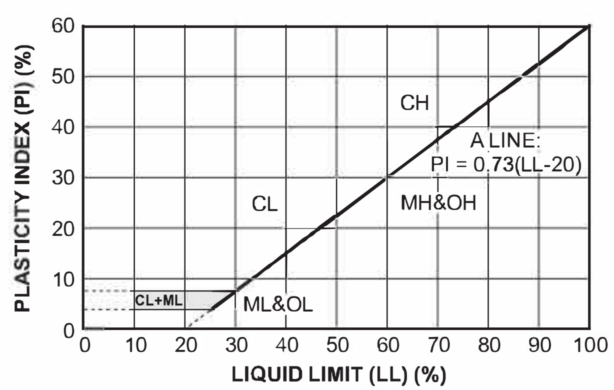
LABORATORY CLASSIFICATION CRITERIA

GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limits above "A" line with P.I. greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for GW	
SM	Atterberg limits below "A" line or P.I. less than 4	Limits plotting in shaded zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
SC	Atterberg limits above "A" line with P.I. greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent .... GW, GP, SW, SP
More than 12 percent .... GM, GC, SM, SC
5 to 12 percent .... Borderline cases requiring dual symbols

PLASTICITY CHART



DESCRIPTIVE TERMINOLOGY & SOIL CLASSIFICATION

LABORATORY/FIELD TESTING DEFINITIONS FOR EXPLORATION LOGS

DD = DRY DENSITY (PCF)

WD = WET DENSITY (PCF)

MC = MOISTURE CONTENT (%)

PL = PLASTIC LIMIT (%)

LL = LIQUID LIMIT (%)

PI = PLASTICITY INDEX

OC = ORGANIC CONTENT (%)

S = SATURATION PERCENT (%)

SG = SPECIFIC GRAVITY

C = COHESION

Φ = ANGLE OF INTERNAL FRICTION

QU = UNCONFINED COMPRESSION STRENGTH

#200 = PERCENT PASSING THE #200 SIEVE

CBR = CALIFORNIA BEARING RATIO

VS = VANE SHEAR

PP = POCKET PENETROMETER

DP = DRIVE PROBE

SPT = STANDARD PENETRATION TEST

BPF = BLOWS PER FOOT (N VALUE)

SH = SHELBY TUBE SAMPLE

GW = GROUND WATER

RQD = ROCK QUALITY DESIDNATION

TP = TEST PIT

B = BORING

HA = HAND AUGER

GROUNDWATER LEVEL/SEEPAGE ENCOUNTERED DURING EXPLORATION

STATIC GROUNDWATER LEVEL WITH DATE MEASURED

CONSISTENCY OF COHESIVE SOILS

CONSISTENCY	STP (BPF)	PP (TSF)
VERY SOFT	0-1	LESS THAN 0.25
SOFT	2 - 4	0.25 - 0.5
MEDIUM STIFF	5 - 8	0.5 - 1.0
STIFF	9 - 15	1.0 - 2.0
VERY STIFF	16 - 30	2.0 - 4.0
HARD	30+	OVER 4.0

RELATIVE DENSITY OF COHESIONLESS SOILS

DENSITY	SPT (BPF)
VERY LOOSE	0 – 4
LOOSE	5 – 10
MEDIUM DENSE	11 – 30
DENSE	31 – 50
VERY DENSE	50+

PARTICLE SIZE IDENTIFICATION

NAME	DIAMETER (INCHES)	SIEVE NO.
ROCK BLOCK	>120	
BOULDER	12-120	
COBBLE	3-12	
GRAVEL		
COURSE	3/4 - 3	
FINE	1/4 – 3/4	NO. 4
SAND		
COARSE	4.75 MM	NO. 10
MEDIUM	2.0MM	NO. 40
FINE	.425 MM	NO. 200
SILT	.075 MM	
CLAY	<0.005 MM	

GRAIN SIZE

FINE GRAINED	<0.04 INCH	FEW GRAINS ARE DISTINGUISHABLE IN THE FIELD OR WITH HAND LENS.
MEDIUM GRAINED	0.04-0.2 INCH	GRAINS ARE DISTINGUISHABLE WITH THE AID OF A HAND LENS.
COARSE GRAINED	0.04-0.2 INCH	MOST GRAINS ARE DISTINGUISHABLE WITH THE NAKED EYE.

DESCRIPTIVE TERMINOLOGY & SOIL CLASSIFICATION

SPT EXPLORATIONS:

STANDARD PENETRATION TESTING IS PERFORMED BY DRIVING A 2 – INCH O.D. SPLIT-SPOON INTO THE UNDISTURBED FORMATION AT THE BOTTOM OF THE BORING WITH REPEATED BLOWS OF A 140 – POUND PIN GUIDED HAMMER FALLING 30 INCHES. NUMBER OF BLOWS (N VALUE) REQUIRED TO DRIVE THE SAMPLER A GIVEN DISTANCE WAS CONSIDERED A MEASURE OF SOIL CONSISTENCY.

SH SAMPLING:

SHELBY TUBE SAMPLING IS PERFORMED WITH A THIN WALLED SAMPLER PUSHED INTO THE UNDISTURBED SOIL TO SAMPLE 2.0 FEET OF SOIL.

AIR TRACK EXPLORATION:

TESTING IS PERFORMED BY MEASURING RATE OF ADVANCEMENT AND SAMPLES ARE RETRIEVED FROM CUTTINGS.

HAND AUGUR EXPLORATION:

TESTING IS PREFORMED USING A 3.25" DIAMETER AUGUR TO ADVANCE INTO THE EARTH AND RETRIEVE SAMPLES.

DRIVE PROBE EXPLORATIONS:

THIS "RELATIVE DENSITY" EXPLORATION DEVICE IS USED TO DETERMINE THE DISTRIBUTION AND ESTIMATE STRENGTH OF THE SUBSURFACE SOIL AND DECOMPRESSED ROCK UNITS. THE RESISTANCE TO PENETRATION IS MEASURED IN BLOWS-PER-1/2 FOOT OF AN 11-POUND HAMMER WHICH FREE FALLS ROUGHLY 3.5 FEET DRIVING THE 0.5 INCH DIAMETER PIPE INTO THE GROUND. FOR A MORE DETAILED DESCRIPTION OF THIS GEOTECHNICAL EXPLORATION METHOD, THE SLOPE STABILITY REFERENCE GUIDE FOR NATIONAL FORESTS IN THE UNITED STATES, VOLUME I, UNITED STATES DEPARTMENT OF AGRICULTURE, EM-7170-13, AUGUST 1994, P. 317-321.

CPT EXPLORATION:

CONE PENETROMETER EXPLORATIONS CONSIST OF PUSHING A PROBE CONE INTO THE EARTH USING THE REACTION OF A 20-TON TRUCK. THE CONE RESISTANCE (QC) AND SLEEVE FRICTION (FS) ARE MEASURED AS THE PROBE WAS PUSHED INTO THE EARTH. THE VALUES OF QC AND FS (IN TSF) ARE NOTED AS THE LOCALIZED INDEX OF SOIL STRENGTH.

ANGULARITY OF GRAVEL & COBBLES

ANGULAR	COARSE PARTICLES HAVE SHARP EDGES AND RELATIVELY PLANE SIDES WITH UNPOLISHED SURFACES.
SUBANGULAR	COARSE GRAINED PARTICLES ARE SIMILAR TO ANGULAR BUT HAVE ROUNDED EDGES.
SUBROUNDED	COARSE GRAINED PARTICLES HAVE NEARLY PLANE SIDES BUT HAVE WELL ROUNDED CORNERS AND EDGES.
ROUNDED	COARSE GRAINED PARTICLES HAVE SMOOTHLY CURVED SIDES AND NO EDGES.

SOIL MOISTURE MODIFIER

DRY	ABSENCE OF MOISTURE; DUSTY, DRY TO TOUCH
MOIST	DAMP BUT NO VISIBLE WATER
WET	VISIBLE FREE WATER

WEATHERED STATE

FRESH	NO VISIBLE SIGN OF ROCK MATERIAL WEATHERING; PERHAPS SLIGHT DISCOLORATION IN MAJOR DISCONTINUITY SURFACES.
SLIGHTLY WEATHERED	DISCOLORATION INDICATES WEATHERING OF ROCK MATERIAL AND DISCONTINUITY SURFACES. ALL THE ROCK MATERIAL MAY BE DISCOLORED BY WEATHERING AND MAY BE SOMEWHAT WEAKER EXTERNALLY THAN ITS FRESH CONDITION.
MODERATELY WEATHERED	LESS THAN HALF OF THE ROCK MATERIAL IS DECOMPOSED AND/OR DISINTEGRATED TO SOIL. FRESH OR DISCOLORED ROCK IS PRESENT EITHER AS A CONTINUOUS FRAMEWORK OR AS CORE STONES.
HIGHLY WEATHERED	MORE THAN HALF OF THE ROCK MATERIAL IS DECOMPOSED AND/OR DISINTEGRATED TO SOIL. FRESH OR DISCOLORED ROCK IS PRESENT EITHER AS DISCONTINUOUS FRAMEWORK OR AS CORE STONE.
COMPLETELY WEATHERED	ALL ROCK MATERIAL IS DECOMPOSED AND/OR DISINTEGRATED TO SOIL. THE ORIGINAL MASS STRUCTURE IS STILL LARGELY INTACT.
RESIDUAL SOIL	ALL ROCK MATERIAL IS CONVERTED TO SOIL. THE MASS STRUCTURE AND MATERIAL FABRIC IS DESTROYED. THERE IS A LARGE CHANGE IN VOLUME, BUT THE SOIL HAS NOT BEEN SIGNIFICANTLY TRANSPORTED.

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT

As the client of a consulting geotechnical engineer, you should know that site subsurface conditions cause more construction problems than any other factor. ASFE/the Association of Engineering Firms Practicing in the Geosciences offers the following suggestions and observations to help you manage your risks.

A GEOTECHNICAL ENG.NEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS Your geotechnical engineering report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. These factors typically include: the general nature of the structure involved, its size, and configuration; the location of the structure on the site; other improvements, such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask your geotechnical engineer to evaluate how factors that change subsequent to the date of the report may affect the report's recommendations.

Unless your geotechnical engineer indicates otherwise, do not use your geotechnical engineering report:

MOST GEOTECHNICAL FINDINGS ARE PROFESSIONAL JUDGMENTS
Site exploration identifies actual subsurface conditions only at those points where samples are taken. The data were extrapolated by your geotechnical engineer who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates, Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations. you and your geotechnical engineer can work together to help minimize their impact. Retaining your geotechnical engineer to observe construction can be particularly beneficial in this respect.

- when the nature of the proposed structure is changed. for example, if an office building will be erected instead of a parking garage, or a refrigerated warehouse will be built instead of an unrefrigerated one;
- when the size, elevation. or configuration of the proposed structure is altered;
- when the location or orientation of the proposed structure is modified;
- when there is a change of ownership; or .for application to an adjacent site.

Geotechnical engineers cannot accept responsibility for problems that may occur if they are not consulted after factors considered in their report's development have changed.

A REPORT'S RECOMMENDATIONS CAN ONLY BE PRELIMINARY
The construction recommendations included in your geotechnical engineer's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site.

Because actual subsurface conditions can be discerned only during earthwork, you should retain your geo- technical engineer to observe actual conditions and to finalize recommendations. Only the geotechnical engineer who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations are valid and whether or not the contractor is abiding by applicable recommendations. The geotechnical engineer who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

SUBSURFACE CONDITIONS CAN CHANGE A geotechnical engineering report is based on conditions that existed at the time of subsurface exploration. Do not base construction decisions on a geotechnical engineering report whose adequacy may have been affected by time. Speak with your geotechnical consult- ant to learn if additional tests are advisable before construction starts. Note, too, that additional tests may be required when subsurface conditions are affected by construction operations at or adjacent to the site, or by natural events such as floods, earthquakes, or ground water fluctuations. Keep your geotechnical consultant apprised of any such events.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS
Consulting geotechnical engineers prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your geotechnical engineer prepared your report expressly for you and expressly for purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the geotechnical engineer. No party should apply this report for any purpose other than that originally contemplated without first conferring with the geotechnical engineer.

GEOENVIRONMENTAL CONCERNS ARE NOT AT ISSUE
Your geotechnical engineering report is not likely to relate any findings, conclusions, or recommendations

about the potential for hazardous materials existing at the site. The equipment, techniques, and personnel used to perform a geoenvironmental exploration differ substantially from those applied in geotechnical engineering. Contamination can create major risks. If you have no information about the potential for your site being contaminated. you are advised to speak with your geotechnical consultant for information relating to geoenvironmental issues.

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION Costly problems can occur when other design profes- sionals develop their plans based on misinterpretations of a geotechnical engineering report. To help avoid misinterpretations, retain your geotechnical engineer to work with other project design professionals who are affected by the geotechnical report. Have your geotechnical engineer explain report implications to design professionals affected by them. and then review those design professionals' plans and specifications to see how they have incorporated geotechnical factors. Although certain other design professionals may be fam- iliar with geotechnical concerns, none knows 'as much about them as a competent geotechnical engineer.

BORING LOGS SHOULD NOT BE SEPARATED FROM THE REPORT Geotechnical engineers develop final boring logs based upon their interpretation of the field logs (assembled by site personnel) and laboratory evaluation of field samples. Geotechnical engineers customarily include only final boring logs in their reports. Final boring logs should not under any circumstances be redrawn for inclusion in architectural or other design drawings. because drafters may commit errors or omissions in the transfer process. Although photographic reproduction eliminates this problem, it does nothing to minimize the possibility of contractors misinterpreting the logs during bid preparation. When this occurs. delays. disputes. and unanticipated costs ara the all-too-frequent result.

To minimize the likelihood of boring log misinterpretation, give contractors ready access to the complete geotechnical engineering report prepared or authorized for their use. (If access is provided only to the report prepared for you, you should advise contractors of the report's limitations. assuming that a contractor was not one of the specific persons for whom the report was prepared and that developing construction cost estimates was not one of the specific purposes for which it was prepared. In other words. while a contractor may gain important knowledge from a report prepared for another party, the contractor would be well-advised to discuss the report with your geotechnical engineer and to perform the additional or alternative work that the contractor believes may be needed to obtain the data specifically appropriate for construction cost estimating purposes.) Some clients believe that it is unwise or unnecessary to give contractors access to their geo- technical engineering reports because they hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems. It also helps reduce the adversarial attitudes that can aggravate problems to disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY
Because geotechnical engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against geotechnical engineers. To help prevent this problem, geotechnical engineers have developed a number of clauses for use in their contracts, reports, and other documents. Responsibility clauses are not exculpatory clauses designed to transfer geotechnical engineers' liabilities to other parties. Instead, they are definitive clauses that identify where geotechnical engineers' responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your geotechnical engineering report. Read them closely. Your geotechnical engineer will be pleased to give full and frank answers to any questions.

RELY ON THE GEOTECHNICAL ENGINEER FOR ADDITIONAL ASSISTANCE
Most ASFE-member consulting geotechnical engineering firms are familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a construction project, from design through construction. Speak with your geotechnical engineer not only about geotechnical issues, but others as well, to learn about approaches that may be of genuine benefit. You may also wish to obtain certain ASFE publications. Contact a member of ASFE of ASFE for a complimentary directory of ASFE publications.

ASFE

8811 Colesville Road/Suite G106/Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017

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