AMERICAN GEOSERVICES

Geotechnical Evaluation Report

3632 90th Ave SE, Mercer Island, WA 98040

Date: September 15, 2022; Project No: 0531-WA21

September 15, 2022

PROJECT NO: 0531-WA21

CLIENT: Mr. Ananta
Seattle, WA

Reference: Lot-Specific Geotechnical Investigation, 3632 90th Avenue SE, Mercer Island, WA

Dear Mr. Ananta,

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 site is a developed lot covered with a residential building in a residential neighborhood. Site topography is illustrated in Figure 2. It should be noted that our scope of services did not include any structural design or foundation engineering of any kind. Our scope of services did not include forensic evaluation or condition survey of the existing structures on the site. Our scope of work was limited to geotechnical evaluation of the property.

SCOPE OF WORK

Our scope of work included following specific items:

- Detailed site reconnaissance to evaluate surface conditions, and slope / landslide characteristics in the vicinity of existing house.
- 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, and seismically induced slope instability.

- 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. This exploration method was used due to accessibility issues. 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. The subsurface exploration results are shown on the individual Boring Logs included in an Appendix. The Legend and Notes necessary to interpret our Boring Logs are also included in an appendix. After the review of subsurface exploration data, it was concluded that soil conditions were competent and additional exploration was not deemed necessary.

  The WDP is a “relative density” exploration device which is used to determine the distribution and to estimate strength of the subsurface soil and decomposed 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 ½ inch diameter pipe into the ground. This measure of resistance to penetration can be used to estimate relative density of soils. For a more detailed description of this geotechnical exploration method, please refer to 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. A representative schematic of WDP is included in appendix along with the published data on correlation between WDP and SPT. The penetration test results are shown on the individual Borehole Log included the appendix. The Legend and Notes necessary to interpret our Borehole Logs are also included in the appendix.

- 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 and engineering design parameters.

It should be noted that our scope of services did not include any structural design or foundation engineering of any kind. Our scope of services did not include forensic evaluation or condition survey of the existing structures on the site. Our scope of work was limited to geotechnical evaluation of the area for existing onsite house structure/foundations. Again, our scope of services did not include structural evaluation of any kind for the existing building. Our scope of services did not include evaluation of any global slope stability or ancient landslide stability evaluation, or detailed seismic slope stability or hazards evaluation or civil engineering evaluation of any kind.

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:

Topsoil: Silty sand, gravel, and organics is present in upper up to 6 inches.

Silty Sand to Sandy Silt: The site is underlain by roughly 2.5 feet of SM/ML in the western portion to 7.5 feet of SM/ML in the eastern portion.

Vashon Drift: Below topsoil, the site is generally underlain by medium dense to dense mixtures of sand, silt, and gravel (SM, SW, ML, GM) extending to the maximum explored depth of 10 feet below ground surface (BGS) where refusal to exploration was encountered due to the presence of gravels and dense relative densities. Our review of available geologic literature (USGS) and water well logs (Department of Ecology, WA) indicated that the Vashon Drift deposit extends to a few tens of feet below the ground surface at the site. See Figure 3 for specific geologic descriptions.

Groundwater: Stable groundwater table was not encountered during explorations 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 Maps
- Geologic Map of Mercer Island, UW, WA
- Environmental Critical Areas Maps.
- City of Mercer Island GIS
- King County GIS
- Soil Survey Maps
- Washington Geologic Hazards Explorer
- Geologic Information Portal, USGS

Landslides: Our review of available geologic maps and landslide hazard maps did not indicate that landslides or debris blow had occurred at the site or in the immediate building area. During our site reconnaissance, we did not notice scarps, crevices, depressions, tension cracks in the ground surface, irregular slope toes, exposed surfaces of ruptures without vegetation, presence of distinct fast-growing vegetation, undrained depressions, etc., that are generally indicative of local active and/or inactive landslides or slope instability that would adversely impact the on-site structure at this time. During our reconnaissance, there was no visual evidence of local slope instability or local active landslides that would adversely impact building stability at this time, however, a detailed landslide evaluation of any kind or slope stability evaluation under seismic conditions was beyond our scope of services.

Notwithstanding, our desk study, site reconnaissance, subsurface exploration, and initial slope stability evaluation (as discussed in following paragraphs) reveal that there is significant potential for slow ground movement (soil creep) in upper 7-8 feet in the eastern portion located east of the setback line shown in Figure 2, especially if the site is disturbed without proper geotechnical design and well-monitored construction activities. There is some potential for global landslides (which includes several properties and neighborhoods) or activation of global ancient landslide mass (if any) due to the geologic conditions present in the site vicinity area present adjacent to the eastern property line. This is especially true during a significant earthquake event. A detailed evaluation of global landslide hazards was beyond our scope of services.

There are potentially mapped landslides and/or ancient landslide deposits very close to the eastern site boundaries. There is also moderate to high potential for the presence of dormant and/or unknown historic landslides, deep-seated ancient landslides, or geologically-recently developed dormant landslides in the site vicinity close to the eastern site boundaries (Figure 5).

The site itself is not mapped as being situated within the existing active or ancient landslide mass or an ancient global landslide. However, the site vicinity area is mapped as having moderate to high landslide hazard (Figure 5). Considering these findings, the site topography, and site geologic conditions, it is our opinion that the immediate site and the vicinity area have ‘high site-specific landslide hazards’ and has some ‘inherent’ risk associated with slope instability and structural impact from the movement of any global/ancient landslide and local slope movements.

Moreover, historically, with construction in such areas, there is always an inherent risk associated with ground movement and/or settlements and related structural damage. The owner should understand these inherent risks. Since this report and preliminary recommendations contained herein have been prepared to maintain a low degree of risk for future structural damage, all our recommendations should be strictly followed. If the owner wants to better understand the risks and to eliminate the site-specific landslide hazard risks, then a detailed and comprehensive geotechnical evaluation including deep drilling, detailed slope stability modeling, and a detailed geologic hazards assessment (including global landslide hazards evaluation) should be performed to quantify the abovementioned risks and to provide detailed geotechnical design recommendations for comprehensive mitigation measures. Unless these recommended studies are performed, the owner is completely responsible for taking all risks associated with any future potential for instability at the site or in the site vicinity.

Initial Slope Stability Evaluation: Based on the results of our initial analyses (as discussed in following paragraphs), in our opinion, at present there are no site-specific slope instability hazards at the site impacting the stability of the existing slope, provided site drainage is properly maintained at the site including all the uphill and downhill areas, during the design life of the structure.

Using the results of geologic and soils literature review (as attached in the appendix) and site reconnaissance data, we analyzed on-site slopes by performing preliminary slope stability analysis. We used the software SLOPE/W to model on-site slopes, subsurface soil conditions, and the impact of existing construction on the stability of the site.

We used several methods (Bishop, Janbu, Spencer, etc.) in order 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 this 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 in the range of 1.5-2.0 is generally acceptable to assure slope stability in residential applications.

Preliminary slope stability analysis was performed using various input soil parameters derived from the results of our preliminary geotechnical evaluation, in order to evaluate the stability of a slope. Of importance were surface and subsurface profiles (slope geometry), soil strength parameters, and groundwater conditions. Based on our experience with past slope stability evaluations in similar geologic conditions, soil strength parameters can vary considerably. Notwithstanding, we used soil strength values typical of on-site soils and native soils/bedrock based on our experience with soil strength testing, as well as back-calculation of soil strength parameters for failed slopes in similar geologic conditions.

- Based on the results of our initial analyses, in our opinion, there is an adequate slope stability safety factor (>1.5).

- There will always be a possibility of localized shallow slides in the easternmost portion at the site slopes and/or localized soil creep in upper 7.5 feet resulting from topography and drainage conditions during wet season of the year, which is typical of many such properties in the Mercer Island area.

- Storm water disposal regulations of King County and City of Mercer Island, and general drainage recommendations given in following sections should be strictly followed.

- In general, areas with moderate to steep slopes present greater construction difficulties. These areas can easily become unstable as the result of poorly planned or non-engineered construction activities such as cuts and fill. Therefore, these areas should not be considered for development or disturbed without a detailed review of site grading plans and house plans by the project geotechnical engineer, and slope stability analysis and foundation design as required once the site grading plans and house plans are completed.

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 landsliding during a major earthquake (Figure 6). Ground acceleration more than 0.3g 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 "D" 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 may occur on a localized scale in the steep slope area or on a global scale impacting the site; however, such an evaluation was beyond our scope of services. A detailed seismic hazards evaluation of the site was beyond our scope of services.

CONCLUSIONS AND RECOMMENDATIONS

Our scope of services did not include any detailed slope stability evaluation, flood hazard evaluation, civil engineering evaluation including any assessment of stormwater or drainage, etc. Our scope of services also did not include the evaluation of the impact of adjacent structures including any retaining walls on the site stability. Structural evaluation or foundation engineering assessment or forensic study of any kind was beyond our scope of services.

Based on the results of limited geotechnical evaluation, in our opinion, the site is suitable for the proposed underpinning construction provided following recommendations are strictly followed and provided all the risks associated with landslides and slope instability are clearly understood. It should be noted that our conclusions and recommendations are intended as design guidance. They are based on our interpretation of the geotechnical data obtained during our evaluation and following assumptions:

- Proposed/Final foundation and/or site mitigation plans will be reviewed and approved by us;
- Proposed foundation mitigation elements will be constructed on level ground with the consideration of the results of our geologic hazards evaluation as described earlier; and
- Structural loads will be typical of residential construction.

Construction recommendations are provided to highlight aspects of construction that could affect the design of the project. Entities requiring information on various aspects of construction must make their own interpretation of the subsurface conditions to determine construction methods, cost, equipment, and work schedule.

OPTION I: CONVENTIONAL SHALLOW FOUNDATION OR SHORT PIERS (NEW CONSTRUCTION)

Any conventional shallow spread foundations, pad footings, short piers, or grade beams may be designed using the following recommendations:

- Excavate footing areas and compact the exposed footing subgrade with a hoepack or hand-held plate compactor.
- Pour concrete only after all foundation subgrades and foundation drains are inspected and approved by a registered geotechnical engineer from our office.

We recommend conventional shallow foundations be designed and constructed in accordance with the following criteria:

- Foundations should be designed for a maximum allowable bearing capacity of **2,000 psf** (pounds per square foot), provided all footings are placed at least 3 feet below existing grades. For higher bearing capacity, foundation should be extended to a depth of at least 6 feet, and a maximum allowable bearing capacity of **3,000 psf** may be used. For lateral load resistance, passive earth pressure value of 300 pcf equivalent fluid density may be used. A coefficient of friction value of 0.4 (unfactored) may be used for concrete foundation against sandy compacted subgrade.

- Estimated final structural loads will dictate the final form and size of foundations to be constructed. However, as a minimum, we recommend bearing walls be supported by continuous footings of at least 12 inches in width. Isolated columns should be supported on pads with minimum dimensions of 12 inches square.

- Continuous foundation walls should be reinforced in the top and bottom to span an unsupported length of at least 8 feet to further aid in resisting differential movement. The project structural engineer should design the foundations as per the project needs.

- Exterior footings and footings in unheated areas should extend at least 18 inches below finished exterior grades shallower than 5H:1V.

We estimate total settlement for foundations designed and constructed as discussed in this section will be one inch or less, with differential settlements on the order of one-half to three-fourths of the total settlement, not accounting for any movement related to possible slope creep.

OPTION II – DEEP FOUNDATION

Driven Steel Pipe Piles: This type of deep foundation is widely used in the Seattle Metro area for residential construction due to light loading, availability of specialty contractors, and the ease of construction. We recommend the use of steel pipe piles with the following driving and design capacity criteria. These piles should not be used for lateral loading capacity unless the project structural engineer and AGS review and discuss project characteristics together. AGS will modify design recommendations as necessary to provide lateral capacity, once the project plans are finalized and made available for our review.

- Pile Diameter and Type: 4-inch diameter, SCH 40, Grade A, galvanized. Black pipe may be used if soil corrosion testing is performed and soils are confirmed as non-corrosive.

- Maximum Pile Spacing: Minimum 12 inches for pile group and maximum 5 feet.

- Critical locations: Install piles at all building corners, individual columns, and all other stress points.

- Minimum Pile Length: 10 feet below the bottom of footing; Minimum 5 feet embedment into dense glacial till. Pile lengths can be modified after review of site grading plans and during pile inspections in the field. Actual pile lengths may vary.
- Footing Embedment: At least 3 inches of a pin pile should be embedded into the footing base. Additional embedment is recommended for post or pier foundations and for lateral stability of the concrete foundation element.
- Pile driving / refusal criteria and Pile Capacity: See table below. Refusal criteria may be revised in the field by AGS representative.

Hammer Size (Pound)	Pipe Refusal Criteria (sec/inch)	Allowable Compression Capacity (kips)
650	20	12
800	15	12
1100	10	12

- Pile Installation Approval: All piles should be proof loaded in the presence of AGS representative. Load tests should be performed on 3% of installed piles or a maximum of 5 piles. At least one load test should be performed. Full time pile load testing and pile installation inspection should be performed by AGS representative and all piles should be approved by AGS prior to the placement of concrete.

  If piles are driven using other types of hammers, then a load test should be performed on 20 percent of piles. Load test capacity should be 200% of design capacity.
- Pile Caps and Grade Beam: Use to avoid eccentric loading and to combine two piles or more.
- Pile Couplings: Use threaded or slip couplers if they fit tightly and transfer load without loss of capacity. Welding of pipe sections may also be used.
- Lateral support: All piles should be designed for adequate lateral support at all locations (especially in the easternmost portion) as per our geotechnical design once the project plans are finalized.

At your request, we can provide recommendations for other types of deep foundation such as straight-shaft drilled piers or driven H piles.

Pin piles can be installed beyond the setback provided additional analyses are performed to assure slope stability. AGS should review and approve final project plans to assure site stability.

SLAB-ON-GRADE (NEW CONSTRUCTION)

Slab-on-grade (if used) should not be constructed without a geotechnical design for the site after site grading plans and project design is completed and made available for our review. For design of floor slabs, a modulus of subgrade reaction of 200 pounds per cubic inch (pci) may be used provided slab-on-grade is placed on properly prepared subgrades after the completion of soil modification procedures described earlier. Based on the results of our analyses, we believe that interior floor slabs designed as recommended above and constructed as recommended in following paragraphs could result in "total" movement of approximately up to 1-inch with "differential" movement on the order of half the total movement.

We recommend that the construction measures outlined in the following paragraphs be followed to reduce potential damage to floor slabs:

- Frequent control joints should be provided at about 10 feet spacing in the floor slab to reduce problems with shrinkage and cracking according to ACI specifications. Control joint spacing is a function of slab thickness, aggregate size, slump and curing conditions. The requirements for concrete slab thickness, joint spacing, and reinforcement should be established by the designer, based on experience, recognized design guidelines and the intended slab use. Placement and curing conditions will have a strong impact on the final concrete slab integrity. Floor slabs should be adequately reinforced.
- The need for a vapor barrier will depend on the sensitivity of floor coverings to moisture. If moisture sensitive floor coverings are proposed for portions of the proposed structure, a capillary break material, typically consisting of a "clean" gravel, should be considered. We can provide additional recommendations if this is the case.
- Provided gravel is desired below the slab, a layer of 4 to 6 inches can be used. Plumbing passing through slabs should be isolated from the slabs and provided with flexible connections to allow for movement. Under slab plumbing should be avoided if possible and should be brought above the slab as soon as possible.
- Where mechanical equipment and HVAC equipment are supported on slabs, we recommend provision of a flexible connection between the furnace and ductwork with a minimum of 1.5 inches of vertical movement.
- Sidewalks and other exterior flatwork should be separated from the slab and the slab should be designed as an independent unit.

STRUCTURAL FLOOR & CRAWL SPACE (NEW CONSTRUCTION)

Structural floors should not be constructed without a geotechnical design for the site after site grading plans and project design is completed and made available for our review. If structural floor is used, then the grade beams (if used) and floor system should be physically isolated from the underlying materials with crawl-space type construction. The void or crawl space of minimum of 6 inches or whatever is the minimum current International Building Code (UBC) requirement.

For crawl-space construction, various items should be considered in the design and construction that are beyond the scope of geotechnical scope of work for this project and require specialized expertise. Some of these include design considerations associated with clearance, ventilation, insulation, standard construction practice, and local building codes. If not properly drained and constructed, there is the potential for moisture to develop in crawl-spaces through transpiration of the moisture/groundwater within native soils underlying the structure, water intrusion from snowmelt and precipitation, and surface runoff or infiltration of water through irrigation of lawns and landscaping. In crawl space, excessive moisture or sustained elevated humidity can increase the potential for mold to develop on organic building materials. A qualified professional engineer in building systems should address moisture and humidity issues.

For the crawl space to remain free of moisture, it is important that drainage recommendations are properly implemented, and adequate inspections are performed prior to the placement of concrete.

- As a minimum, subgrade beneath a structural floor system should be graded so that water does not pond. Perimeter drains, and under-slab drains should be installed in conjunction with a sump pump system to eliminate the potential for ponding and any subsequent damage to foundation and slab elements. The lot-specific perimeter dewatering and underdrain systems should be properly designed and connected to the area underdrain system or a sump-pump system for suitable discharge from the lot.

- The underdrain system should consist of adequate lateral drains and a main drain, regular clean out and inspection locations, and proper connections to the sump-pump system for discharge into suitable receptacles located away from the site.

- The entire design and construction team should evaluate, within their respective field of expertise, the current and potential sources of water throughout the life of the structure and provide any design/construction criteria to alleviate the potential for moisture changes. If recommended drain systems are used, the actual design/layout, outlets, locations, and construction means, and methods should be observed by a representative of AGS.

RETAINING WALLS (NEW CONSTRUCTION)

Any new retaining walls should not be constructed without a geotechnical design for the site after site grading plans and project design is completed and made available for our review. In general, for preliminary design, retaining walls for at-rest conditions can be designed to resist an equivalent fluid density of 65 pcf for on-site granular materials. Retaining walls for unrestrained conditions (free lateral movement) can be designed to resist an equivalent fluid density of 50 pcf for on-site granular materials. For passive resistance of unrestrained walls, we recommend passive resistance of 300 psf per foot of wall height. A coefficient of friction value of 0.35 may be used for contact between the prepared soil surface and concrete base.

The above recommended values do not include a factor of safety or allowances for surcharge loads such as adjacent foundations, sloping backfill, vehicle traffic, or hydrostatic pressure. We should be contacted to provide additional recommendations for any specific site retaining conditions.

Retaining walls should be designed for seismic loading conditions. We recommend the addition of uniform lateral pressure under seismic conditions of 9H psf, where H is the design height of the retaining wall in feet. The safety factor of 1.25 may be used for retaining wall design under seismic loading conditions.

Retaining wall backfill should be placed in strict accordance with our earthwork recommendations given below. Backfill should not be over-compacted to minimize excessive lateral pressures on the walls. As a precautionary measure, a drainage collection system (drains or geosynthetic drains) should be included in the wall design to minimize hydrostatic pressures. A prefabricated drainage composite or drain board such as the MiraDrain 2000 or an engineer-approved equivalent may be installed along the backfilled side of the basement foundation wall.

SUBSURFACE DRAINAGE

We strongly recommend preparing a detailed drainage plan and its implementation for the site to assure proper subsurface and surface drainage and long-term site stability. AGS can prepare a detailed drainage plan at an additional cost.

Proper subsurface drainage is critical for long-term performance of the mitigated structures. As a minimum, recommendations given below should be strictly followed.

- A perimeter drain/dewatering system should be installed to reduce the potential for groundwater entering foundation and slab areas.

- The subgrade beneath any new structural floor system should be graded so that water does not pond. In addition, drain laterals that span the crawl space are recommended to prevent ponding of water within the crawlspace.

- As a minimum, the subsurface drainage system should consist typically of 4-inch minimum diameter perforated rigid PVC pipe surrounded by at least one pipe diameter of free draining gravel. The pipe should be wrapped in a geosynthetic to prevent fine soils from clogging the system in the future. The pipe should drain by gravity to a suitable all-weather outlet or to a properly designed area underdrain system. Surface cleanouts of the perimeter drain should be installed at minimum serviceability distances around the addition. A properly constructed drain system can result in a reduction of moisture infiltration of the subsurface soils. Drains which are improperly installed can introduce settlement or heave of the subsurface soils and could result in improper surface grading only compounding the potential issues.

- The entire design and construction team should evaluate, within their respective field of expertise, the current and potential sources of water throughout the life of the structure and provide any design/construction criteria to alleviate the potential for moisture changes. If recommended drain systems are used, the actual design/layout, outlets, and location should be designed by AGS. The construction means, and methods should be observed by a representative of AGS.

SURFACE DRAINAGE

A detailed drainage plan should be prepared by us once the site mitigation plans and project foundation mitigation design are completed. In general, proper surface drainage should be maintained at this site during and after completion of construction operations. The ground surface adjacent to buildings should be sloped to promote rapid run-off of surface water. We recommend a minimum slope of six inches in the first five horizontal feet for landscaped or graveled areas. These slopes should be maintained during the service life of buildings.

Landscaping should be limited around building areas. 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 building. Ponding of water should not be allowed immediately adjacent to the building.

It is important to follow these recommendations to minimize settling of the foundation 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 foundations and flatwork and should be provided during the life of the structure. Paved areas within 10 feet of structures should slope at a minimum of 2 percent away from foundations, and landscape areas within 10 feet of structures should slope away at a minimum of 8 percent. 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 should be created such that water is diverted away from building sites and away from backfill areas of adjacent buildings.

EARTHWORK CONSTRUCTION (NEW CONSTRUCTION)

Once the grading plans are finalized for any new construction, we should be contacted for specific earthwork recommendations, especially the stability of adjacent structures, shoring requirements, cut slopes, and construction dewatering. In any case, site grading should be carefully planned so that positive drainage away from all structures is achieved. As a minimum, following earthwork recommendations should be followed for all aspects of the project.

Fill Placement: Fill material should be placed in uniform horizontal layers (lifts) not exceeding 12 inches before compacting to the required density and before successive layers are placed. If the contractor's equipment is not capable of properly moisture conditioning and compacting 8-inch lifts, then the lift thickness shall be reduced until satisfactory results are achieved.

Import soils should be approved by AGS prior to placement. *Fill placement observations and fill compaction tests should be performed by AGS Engineering to minimize the potential for future problems.* Fill material should not be placed on frozen ground. Vegetation, roots, topsoil, the existing fill materials, and other deleterious material to depth of approximately 6 inches should be removed before new fill material is placed.

On-site fill to be placed should be moisture treated to within 2 percent of optimum moisture content (OMC) for sand fill. Fill to be placed in wall backfill areas and driveway areas and all other structural areas should be compacted to 95% of Modified Proctor (ASTM D1557) maximum dry density or greater. Compaction in landscape areas should be 85% or greater.

Imported structural fill should consist of sand or gravel material with a maximum particle size of 3 inches or less. In addition, this material shall have a liquid limit less than 30 and a plasticity index of 15 or less. Structural fill should also have a percent fine between 15 to 30 percent passing the No. 200 sieve. Structural fill should be moisture conditioned to within 2 percent of OMC and compacted to at least 95 percent of Modified Proctor (ASTM D1557) maximum dry density.

Excavation: In our opinion, the materials encountered at this site may be excavated with conventional mechanical excavating equipment. Although our borings did not encounter "buried" foundation elements or other structures or debris, these materials will most likely be encountered during excavation activities. Debris materials such as brick, wood, concrete, and abandoned utility lines, if encountered, should be removed from structural areas when encountered in excavations and either wasted from the site or placed in landscaped areas.

Temporary excavations should comply with OSHA and other applicable federal, state, and local safety regulations. In our opinion, OSHA Type B/C soils will be encountered at this site during excavation. OSHA recommends maximum allowable unbraced temporary excavation slopes of 1.25:1(H:V) for Type B/C soils for excavations up to 10 feet deep. Permanent cut and fill slopes are anticipated to be stable at slope ratios as steep as 2H:1V (horizontal to vertical) under dry conditions. New slopes should be revegetated as soon as possible after completion to minimize erosion.

We recommend a minimum of 15 feet of clearance between the top of excavation slopes and soil stockpiles or heavy equipment or adjacent structures (subject to approval of AGS). If braced excavations are to be used, they should be reviewed and designed by AGS. It should be noted that near-surface soils encountered at the site will be susceptible to some sloughing and excavations should be periodically monitored by AGS's representative.

Once the grading plans and construction sequencing are finalized, we should be contacted to evaluate the need for shoring or underpinning or overall site stability. If shoring is deemed necessary, we should be contacted to provide detailed shoring plans to assure on-site stability during construction. Note that is the responsibility of the contractor and/or owner to assure site stability during and after construction.

Wet Weather: 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.

Cold Weather: If earthwork is performed during the cold winter months when freezing might become a factor, no grading fill, structural fill or other fill should be placed on frosted or frozen ground, nor should frozen material be placed as fill. Frozen ground should be allowed to thaw or be completely removed prior to placement of fill. A good practice is to cover the compacted fill with a "blanket" of loose fill to help prevent the compacted fill from freezing overnight. The "blanket" of loose fill should be removed the next morning prior to resuming fill placement.

During cold weather, foundations, concrete slabs-on-grade, or other concrete elements should not be constructed on frozen soil. Frozen soil should be completely removed from beneath the concrete elements, or thawed, scarified and re-compacted. The amount of time passing between excavation or subgrade preparation and placing concrete should be minimized during freezing conditions to prevent the prepared soils from freezing. Blankets, soil cover or heating as required may be utilized to prevent the subgrade from freezing.

CONCRETE CONSTRUCTION

Concrete sidewalks and any other exterior concrete flatwork around the proposed structure may experience some differential movement and cracking. While it is not likely that the exterior flatworks can be economically protected from distress, we recommend following techniques to reduce the potential long-term movement:

- Scarify and re-compact at least 12 inches of subgrade material located immediately beneath structures.
- Avoid landscape irrigation adjacent to structures.
- Thicken or structurally reinforce the structures.

We recommend Type I-II cement for all concrete in contact with the soil on this site. Calcium chloride should not be added. Concrete must be protected from low temperatures and properly cured.

LIMITATIONS

Historically, with construction in areas extremely close to the landslide hazards and seismicity, there is an inherent risk associated with ground movement and/or settlements and related structural damage due to poor drainage, an earthquake, or otherwise. The owner is completely responsible for taking all risks associated with any future potential for instability at the site or in the site vicinity, unless a detailed geotechnical evaluation is performed with a lot more exploration and comprehensive landslide and slope stability assessment.

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.

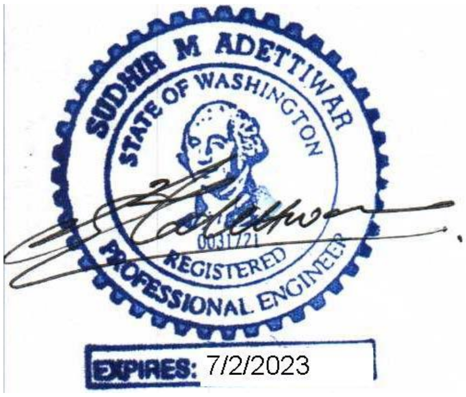
Our Scope of Work for this project did not include research, testing, or assessment relative to past or present contamination of the site by any source. If such contamination were present, it is very likely that the exploration and testing conducted for this report would not reveal its existence. If 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.

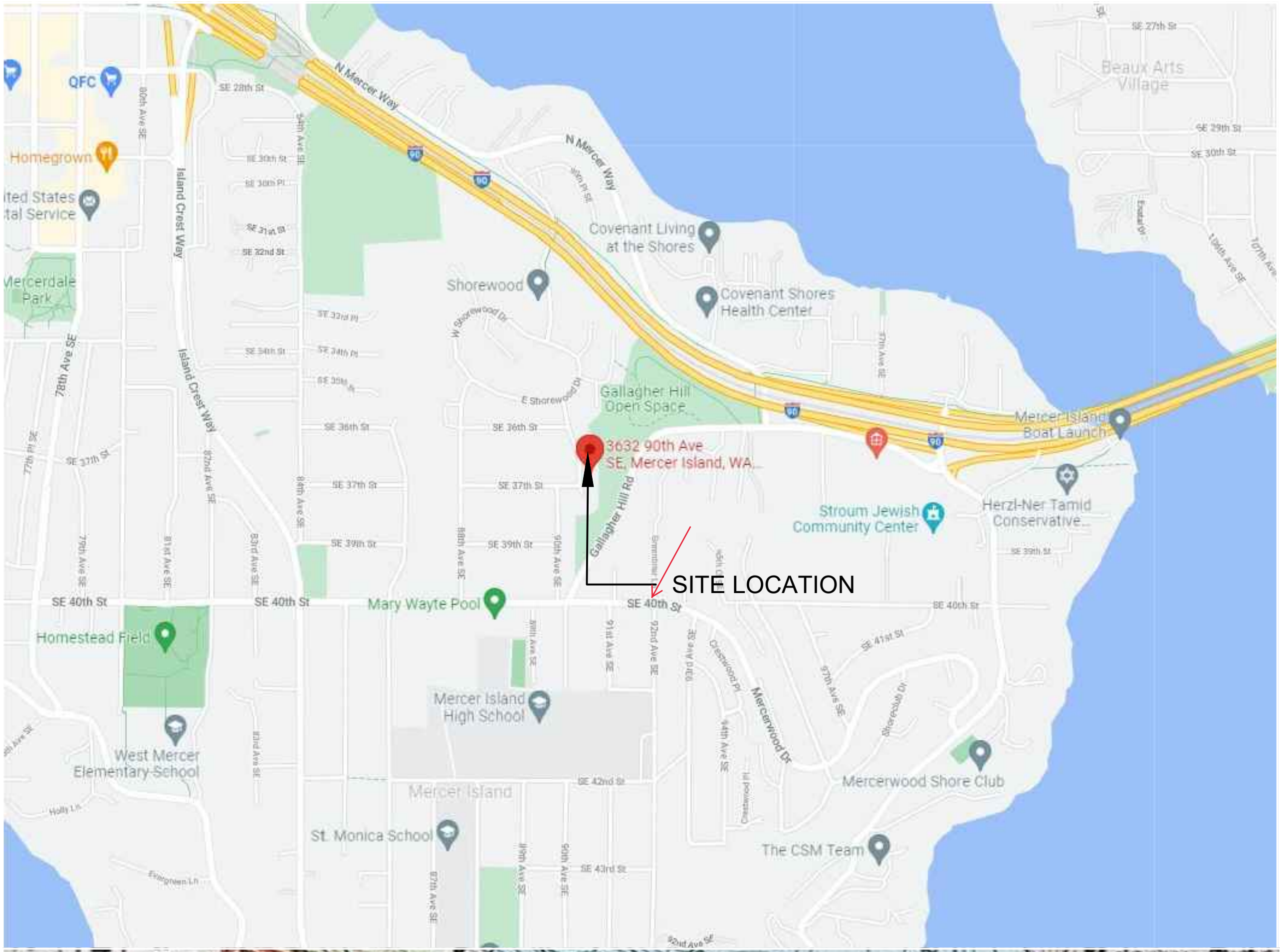
We appreciate the opportunity to be of service to you on this project. If we can provide additional assistance or observation and testing services during design and construction phases, please call us at 1 888 276 4027.

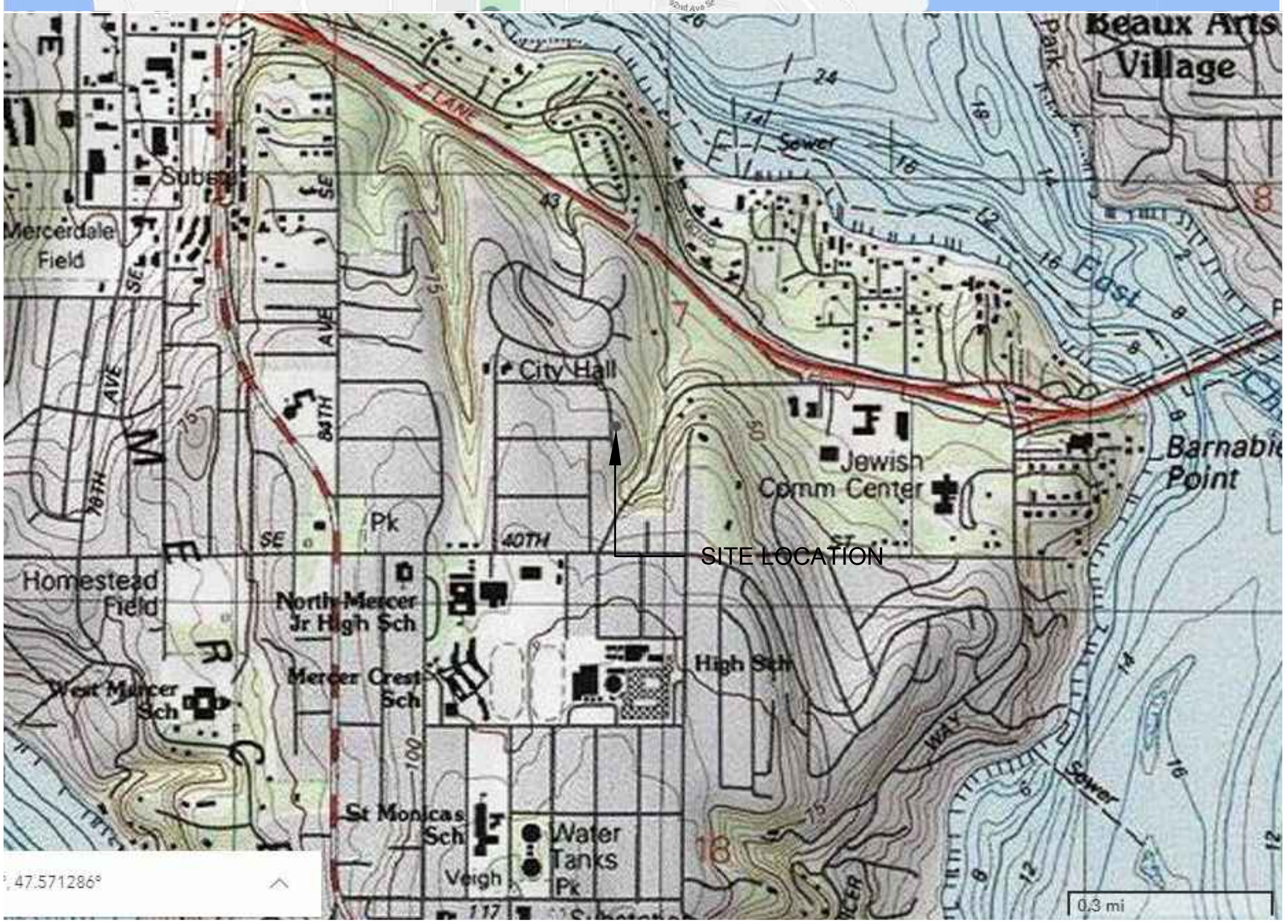
Sincerely,



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

FIGURES



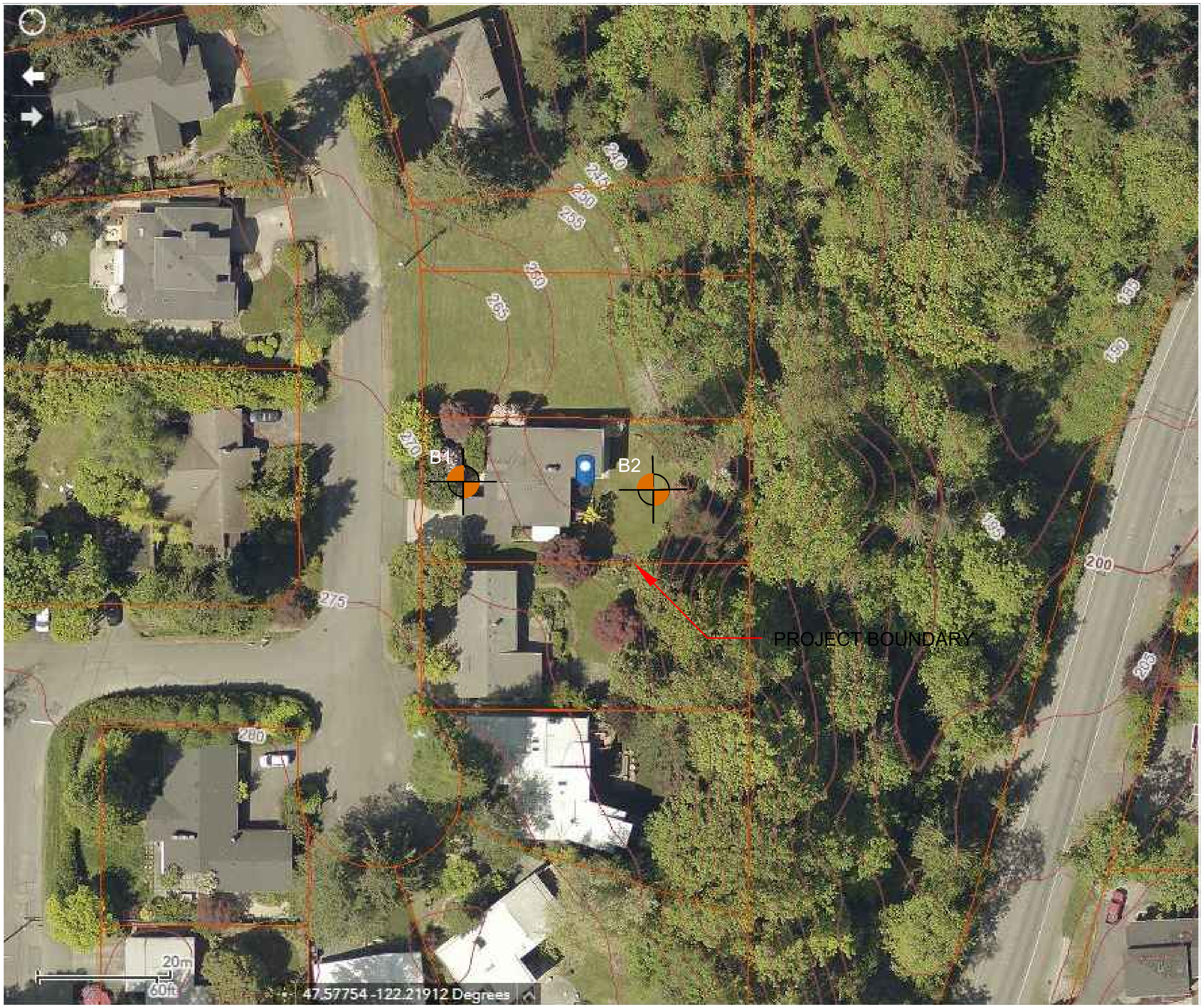


REFERENCE:
GOOGLE MAPS
USGS TOPOGRAPHIC MAPS

N

AMERICAN GEOSERVICES
888.276.4027 - americangeoservices.com

FIGURE 1: SITE LOCATION MAP



NOTE:
SCHEMATIC PLAN TO SHOW APPROXIMATE SUBSURFACE EXPLORATION LOCATION ONLY; NOT SURVEYED.

N

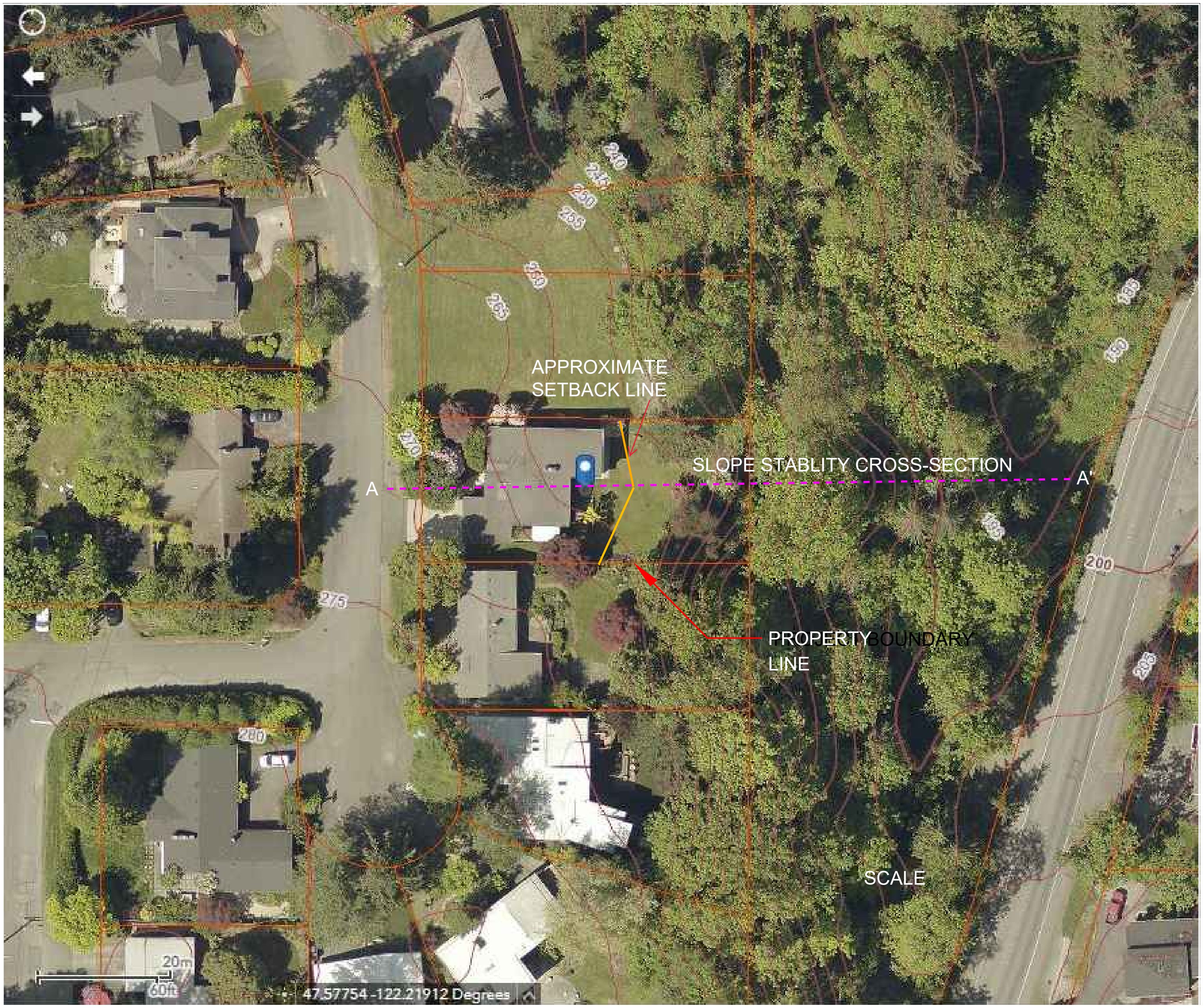
LEGEND:

DESIGNATES SUBSURFACE EXPLORATION LOCATION, BY AMERICAN GEOSERVICES, LLC. , DECEMBER 2021 SEE EXPLORATION LOG IN APPENDIX FOR FURTHER DETAILS.

REFERENCE:
KING COUNTY
PROPERTY MAP

AMERICAN GEOSERVICES
888.276.4027 - americangeoservices.com

FIGURE 2: SCHEMATIC SITE PLAN



NOTES: SETBACK LINE CAN BE MODIFIED BASED ON ADDITIONAL ANALYSES.

NOTE:
SCHEMATIC PLAN TO SHOW APPROXIMATE SUBSURFACE EXPLORATION LOCATION ONLY; NOT SURVEYED.

N

LEGEND:
DESIGNATES SUBSURFACE EXPLORATION LOCATION, BY AMERICAN GEOSERVICES, LLC. , DECEMBER 2021 SEE EXPLORATION LOG IN APPENDIX FOR FURTHER DETAILS.

REFERENCE:
KING COUNTY
PROPERTY MAP

AMERICAN GEOSERVICES
888.276.4027 - americangeoservices.com

FIGURE 2: SCHEMATIC SITE PLAN

LEGEND

Landslide deposits Ql	Includes intimately mixed material from various units, and discrete blocks of various units

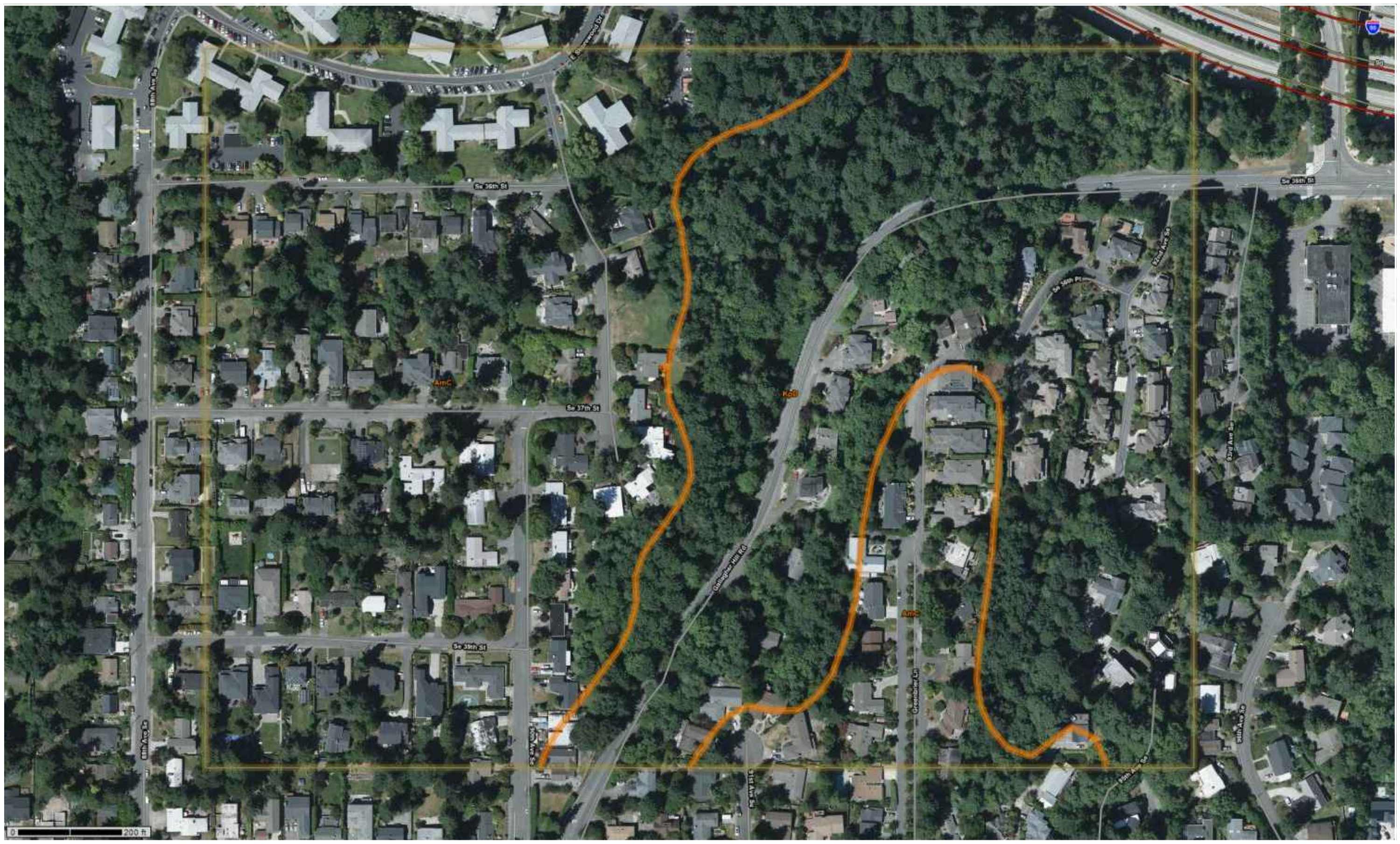
Older sand Qos	Generally uncemented. Physical characteristics consistent vertically and laterally over broad areas. Where unit overlies impermeable material, springs near base cause erosion

REFERENCE
U.S. GEOLOGIC SURVEY

N

AMERICAN GEOSERVICES
888.276.4027 - americangeoservices.com

FIGURE 3: GEOLOGIC MAP



LEGEND

King County Area, Washington (WA633)

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
AmC	Arents, Alderwood material, 6 to 15 percent slopes	33.9	55.9%
KpD	Kitsap silt loam, 15 to 30 percent slopes	26.8	44.1%
Totals for Area of Interest		**60.6**	**100.0%**

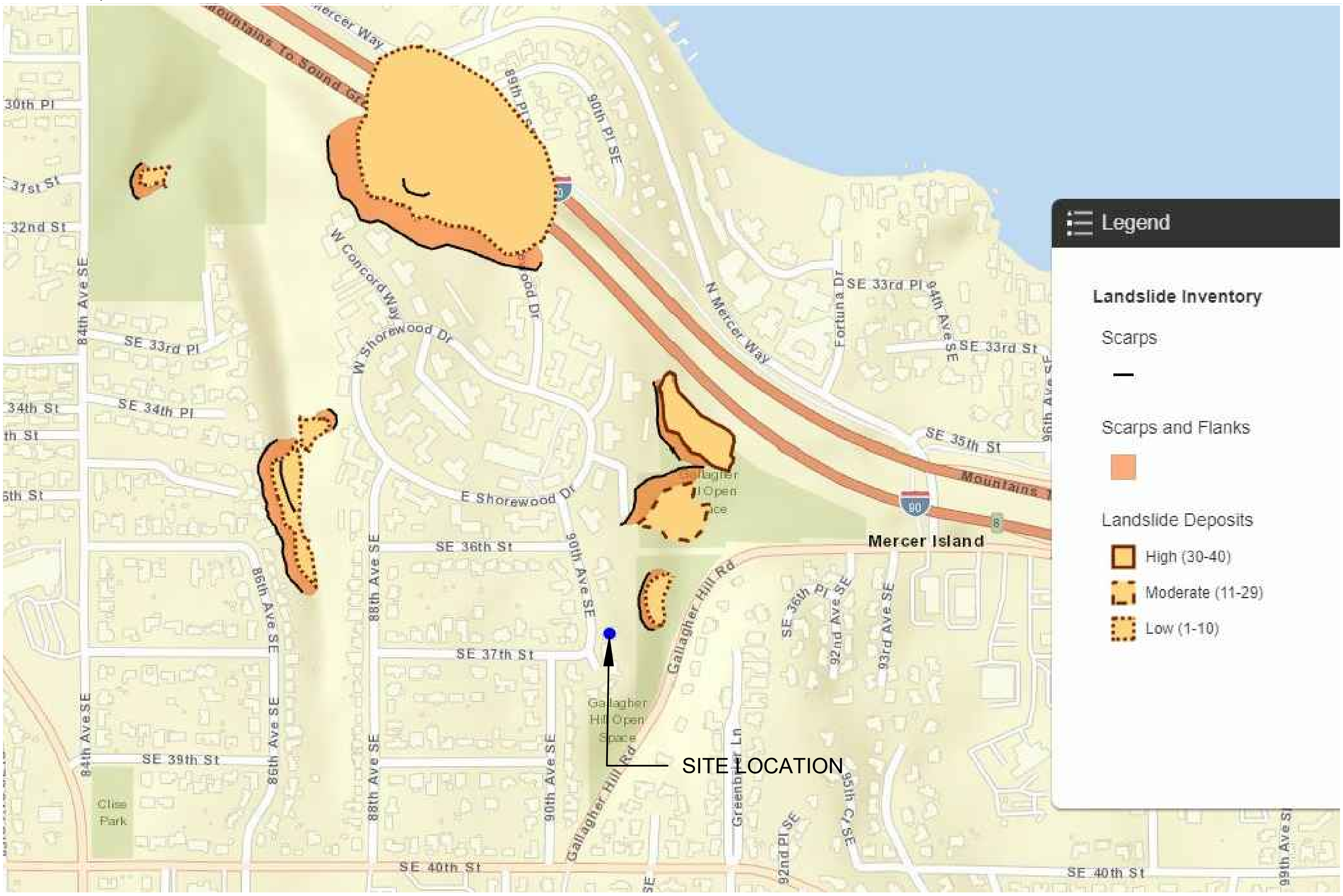
N

REFERENCE:

WEB SOIL SURVEY

AMERICAN GEOSERVICES
888.276.4027 - americangeoservices.com

FIGURE 4: SOIL SURVEY MAP

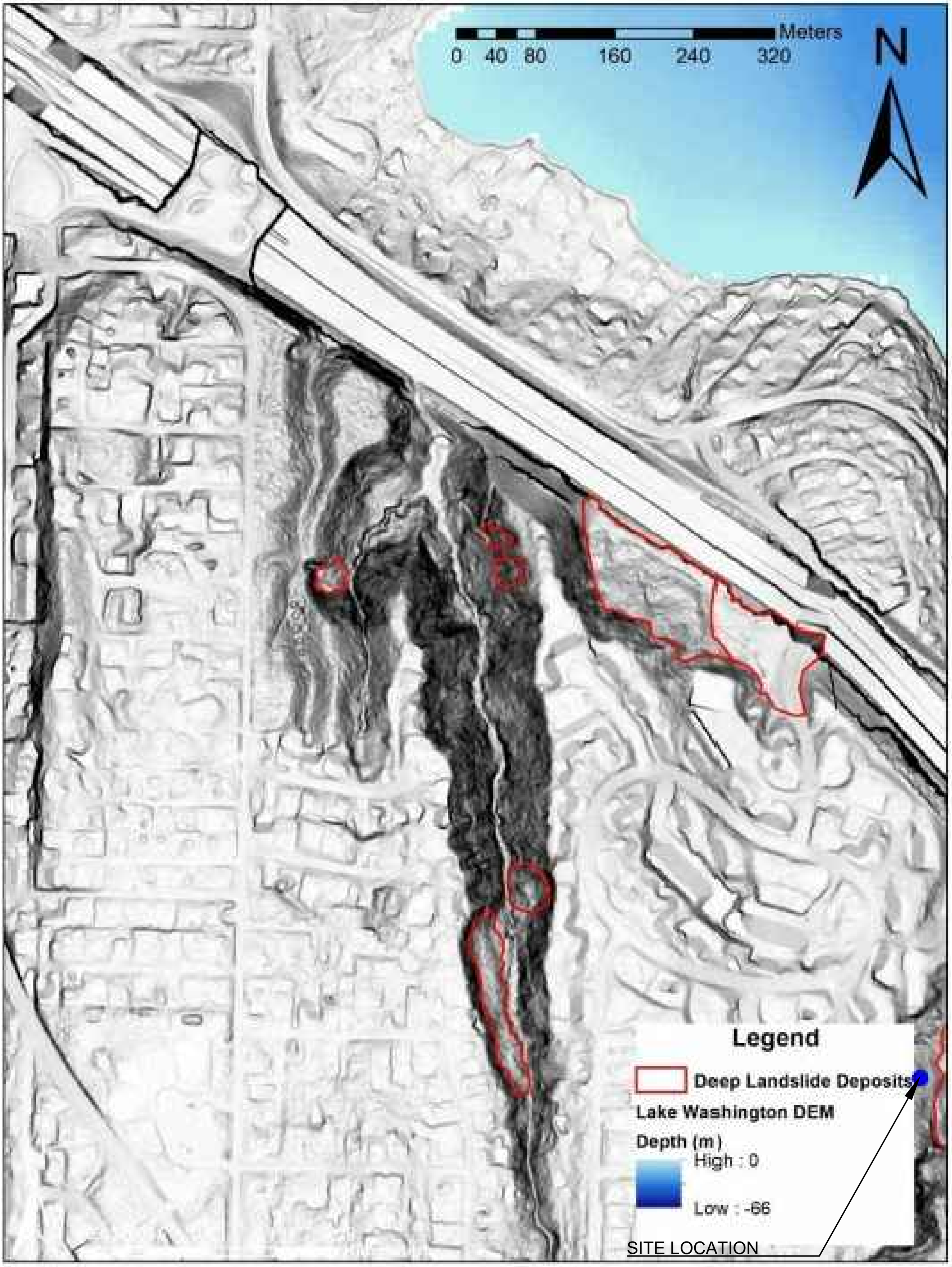


N

REFERENCE:
WASHINGTON GEOLOGIC INFORMATION PORTAL,

AMERICAN GEOSERVICES
888.276.4027 - americangeoservices.com

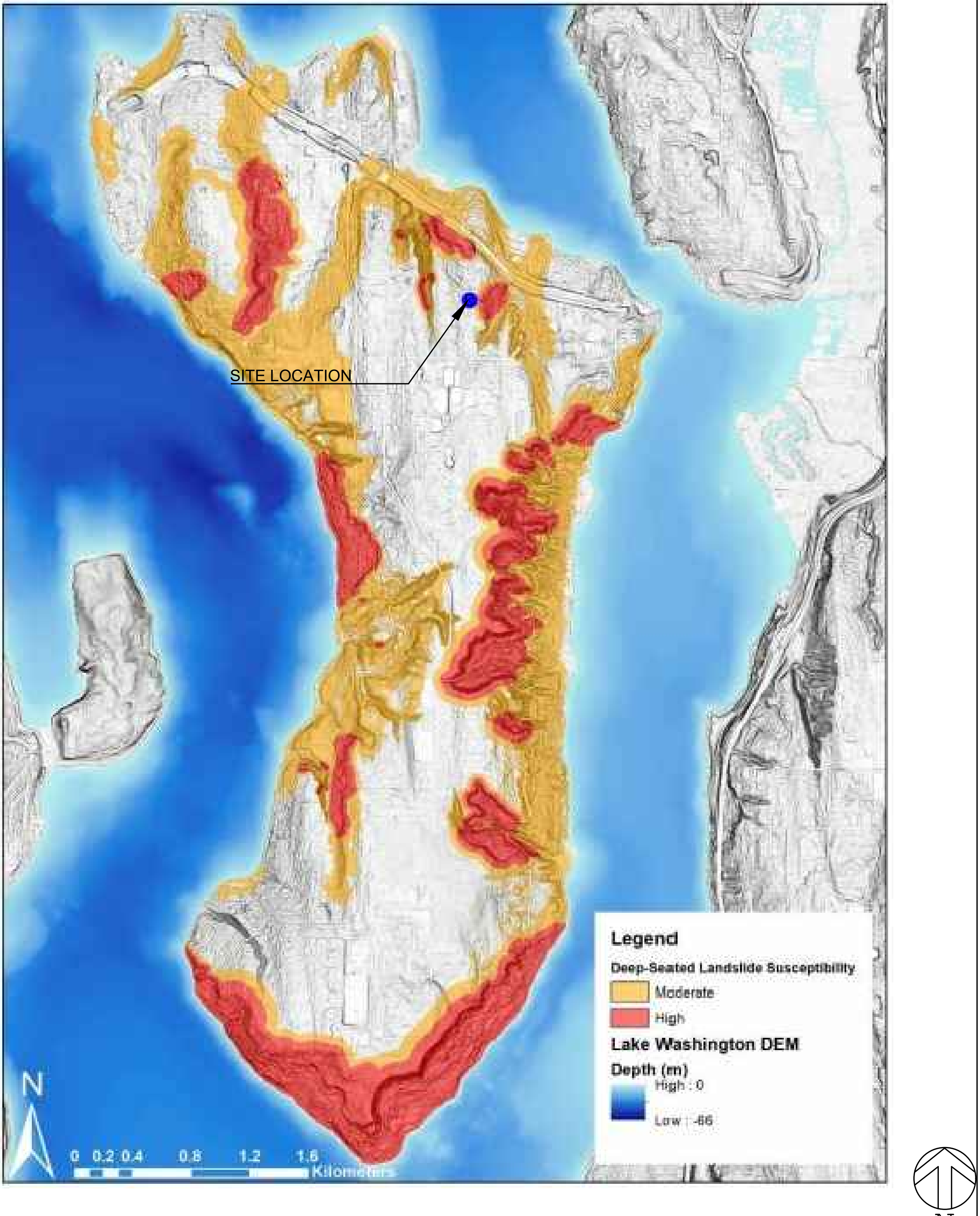
FIGURE 5: LANDSLIDE HAZARD MAP



REFERENCE:
LANDSLIDES HAZARD
STUDY MERCER ISLAND
WASHINGTON

AMERICAN GEOSERVICES
888.276.4027 - americangeoservices.com

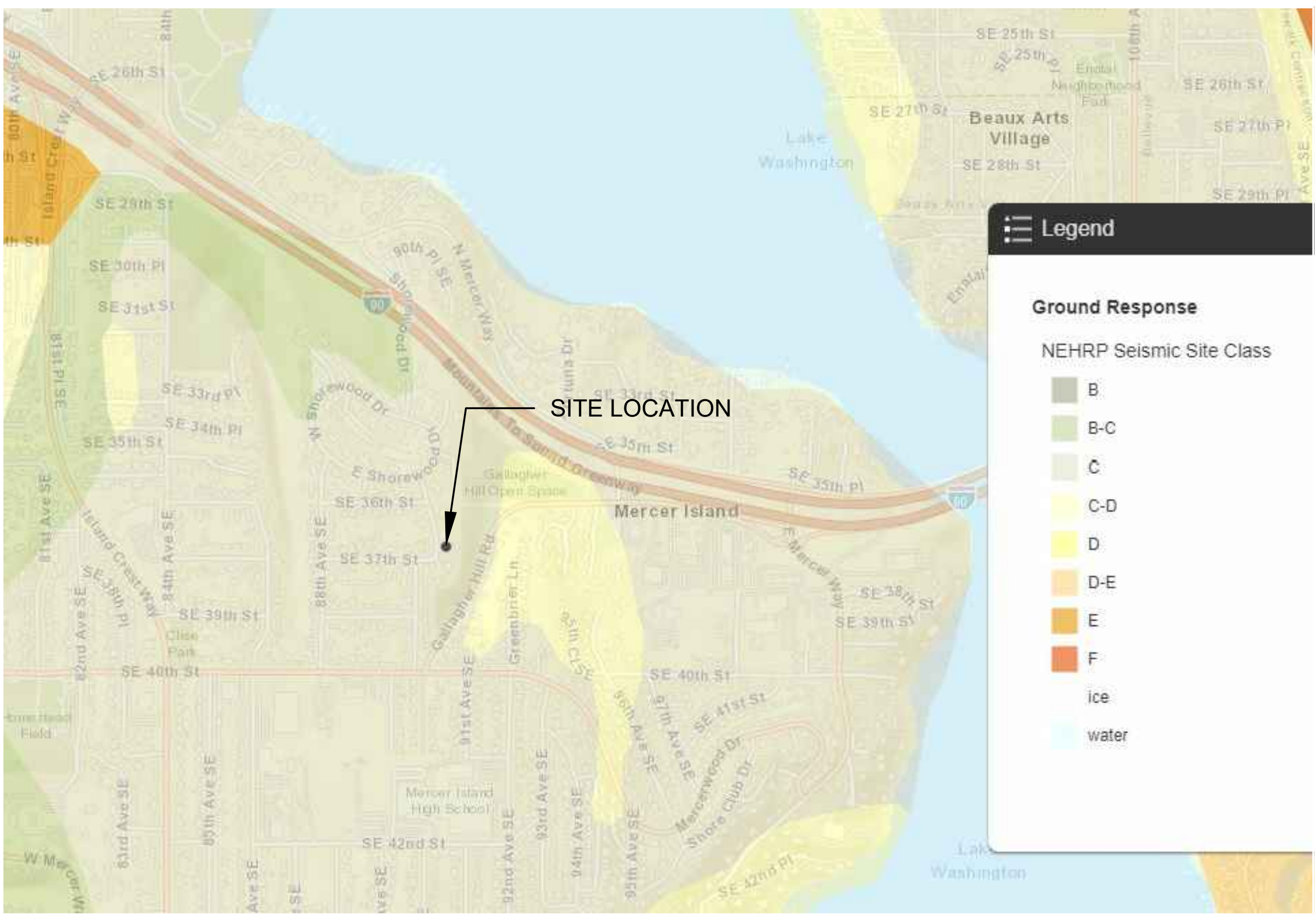
FIGURE 6: DEEP LANDSLIDES DEPOSIT



REFERENCE:
LANDSLIDES HAZARD STUDY MERCER ISLAND WASHINGTON

AMERICAN GEOSERVICES
888.276.4027 - americangeoservices.com

FIGURE 7: DEEP SEATED LANDSLIDES SUSCEPTIBILITY



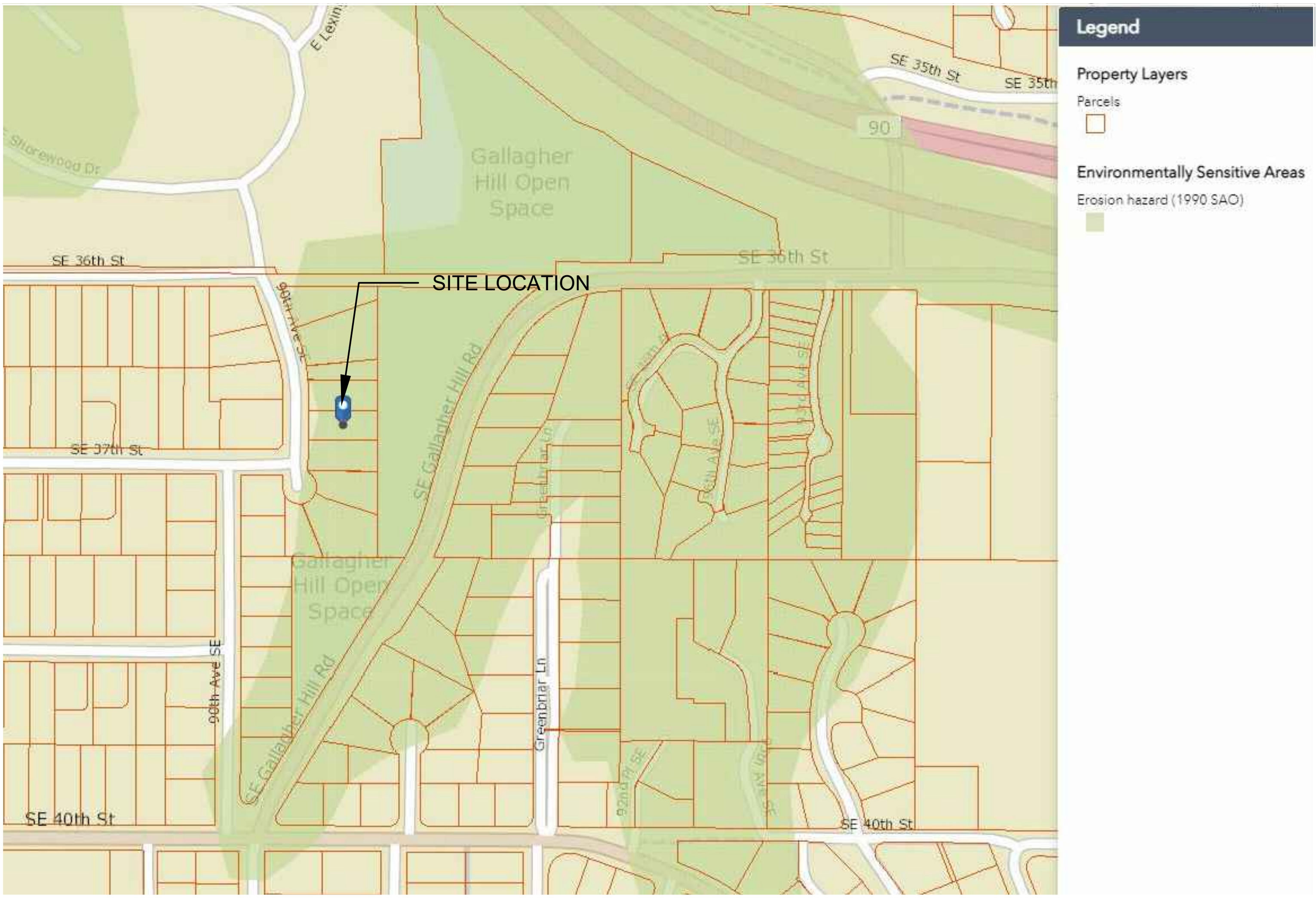
N

REFERENCE:

WASHINGTON GEOLOGIC INFORMATION PORTAL

AMERICAN GEOSERVICES
888.276.4027 - americangeoservices.com

FIGURE 8:EARTHQUAKE HAZARD MAP



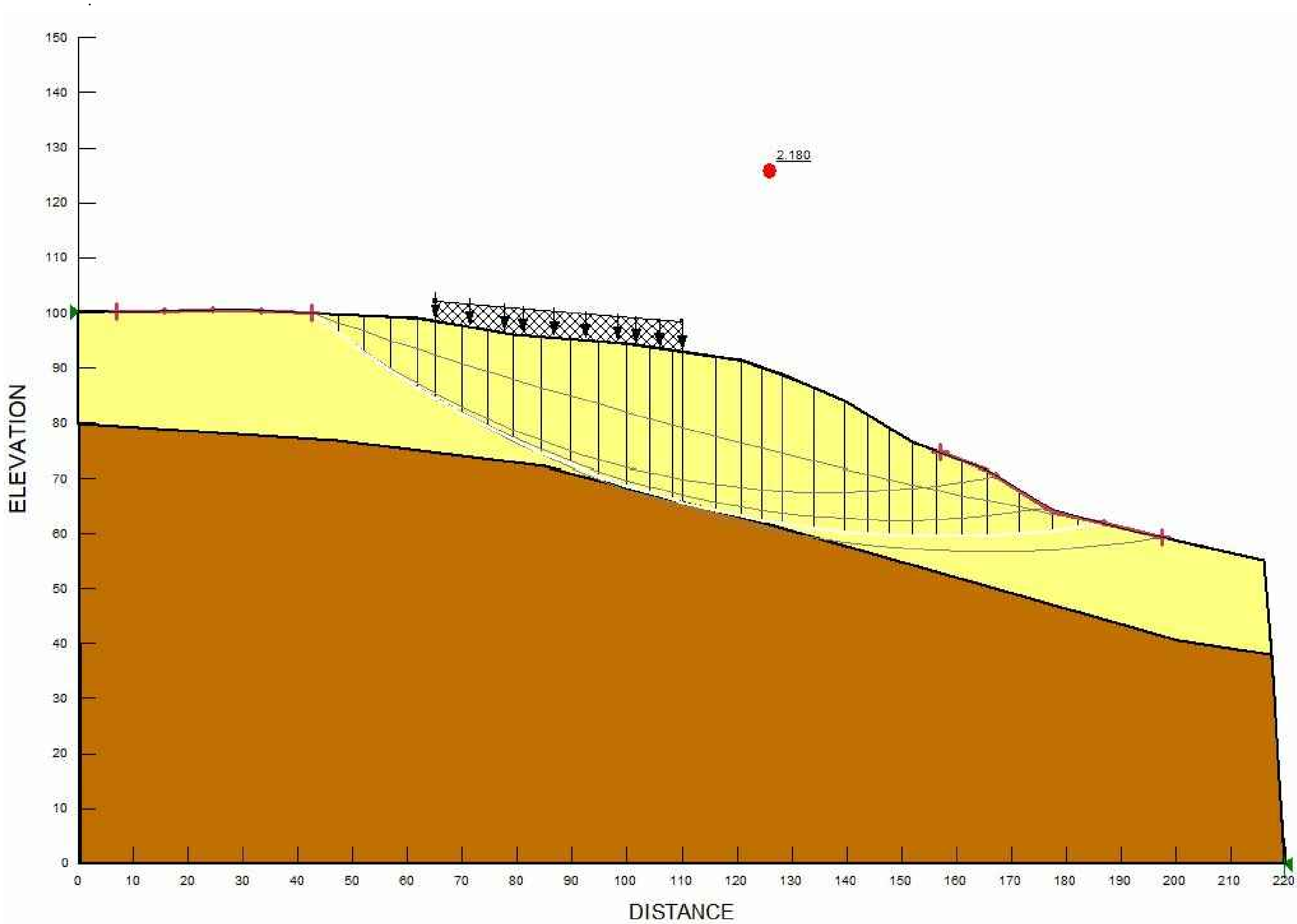
N

REFERENCE:

WASHINGTON GEOLOGIC INFORMATION PORTAL

AMERICAN GEOSERVICES
888.276.4027 - americangeoservices.com

FIGURE 9:EROSION HAZARD MAP

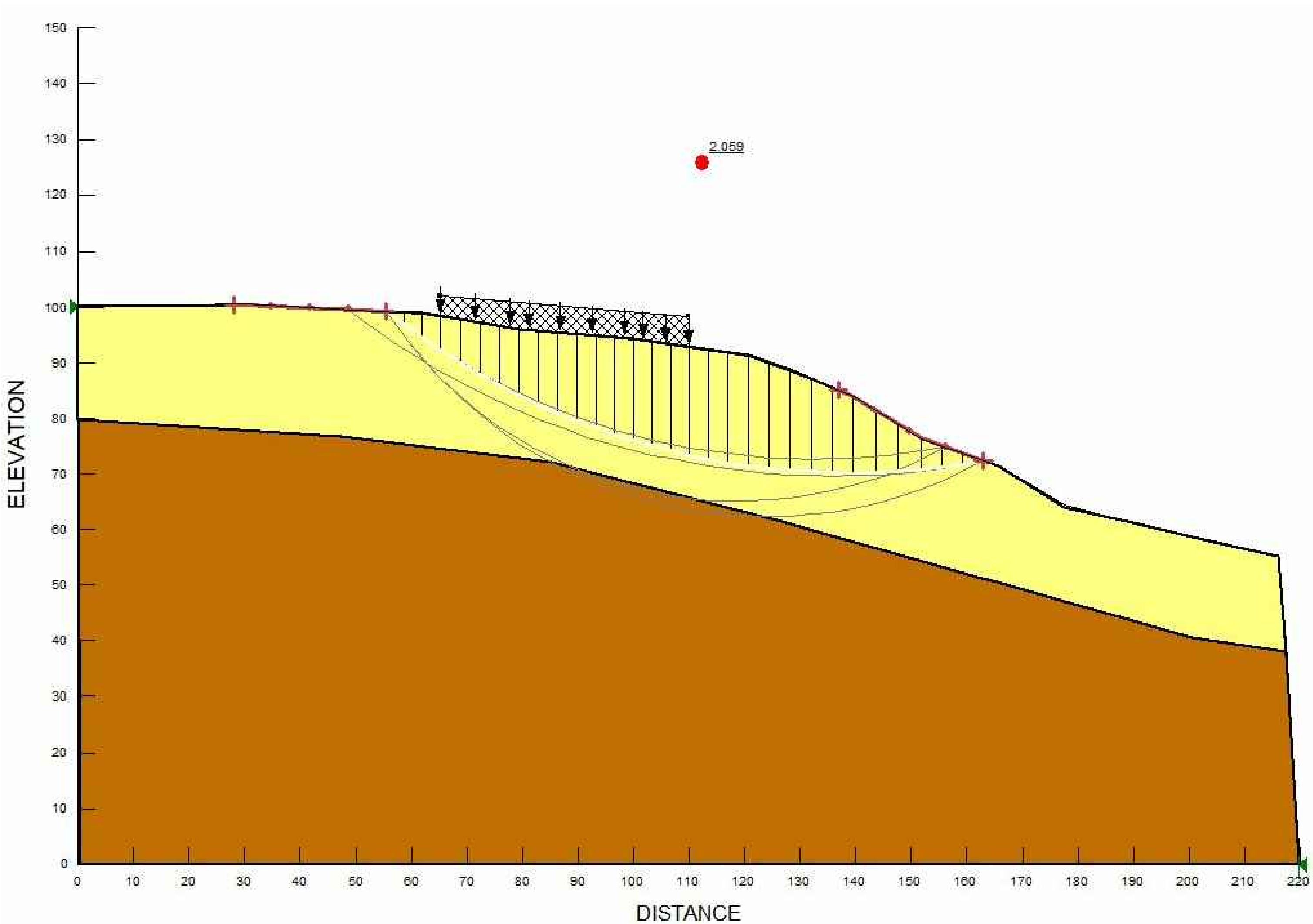


REFERENCE:

SLOPE/W SOFTWARE

AMERICAN GEOSERVICES
888.276.4027 - americangeoservices.com

FIGURE :SLOPE STABILITY ANALYSIS



N

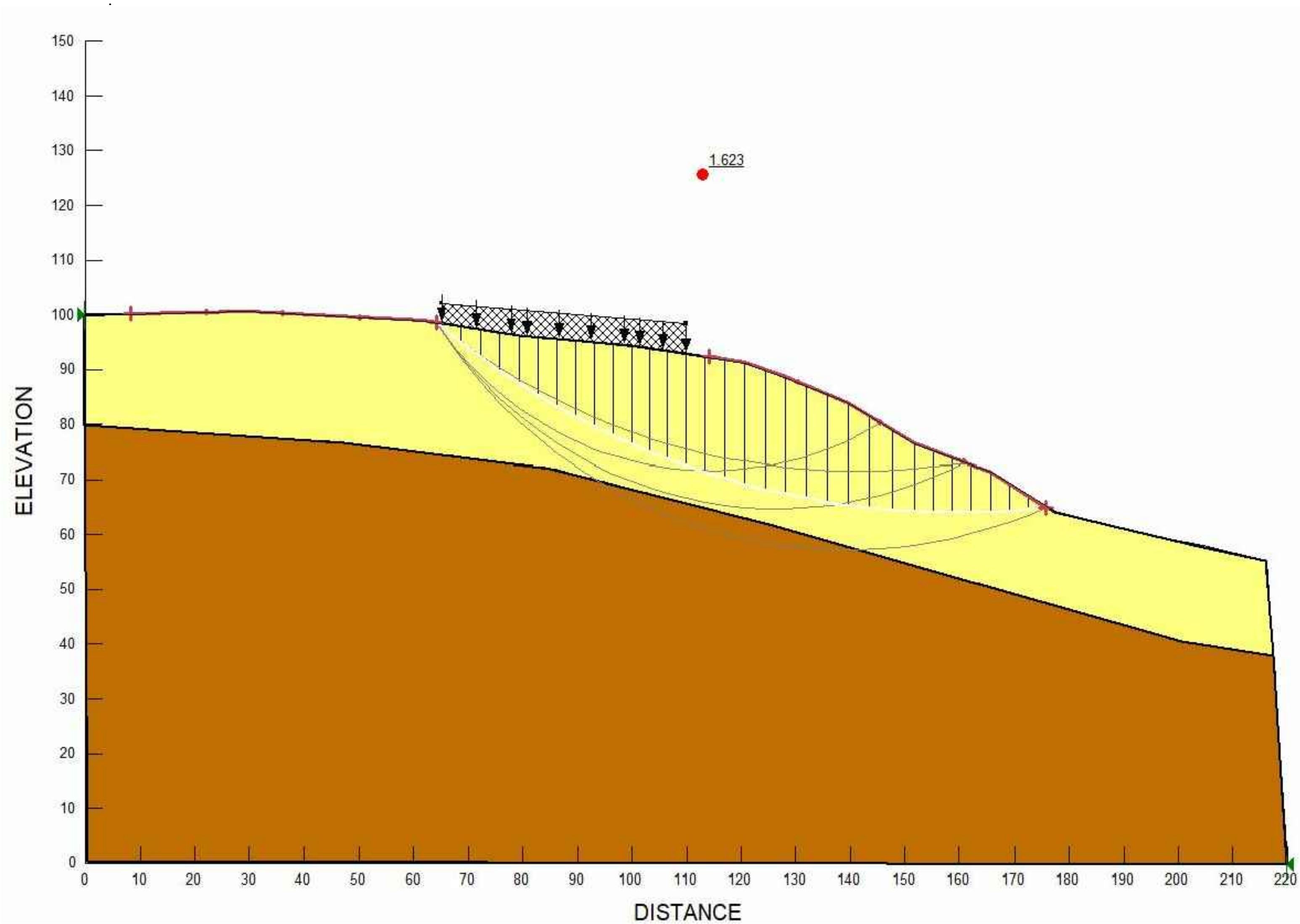
REFERENCE:

SLOPE/W SOFTWARE

AMERICAN GEOSERVICES

888.276.4027 - americangeoservices.com

FIGURE :SLOPE STABILITY ANALYSIS



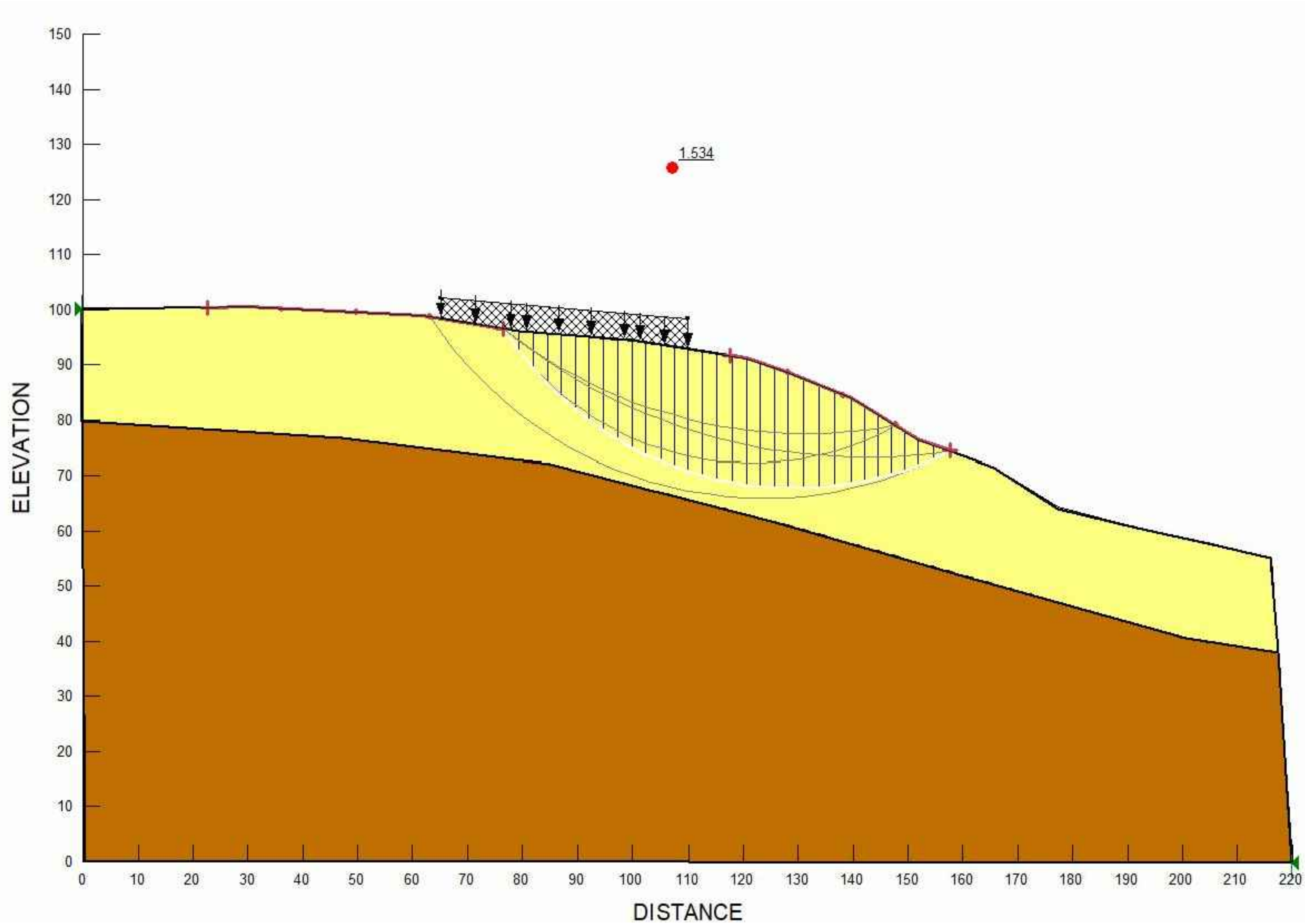
N

REFERENCE:

SLOPE/W SOFTWARE

AMERICAN GEOSERVICES
888.276.4027 - americangeoservices.com

FIGURE :SLOPE STABILITY ANALYSIS



REFERENCE:

SLOPE/W SOFTWARE

AMERICAN GEOSERVICES
888.276.4027 - americangeoservices.com

FIGURE :SLOPE STABILITY ANALYSIS

APPENDIX

B1

Project Number	0531-WA21	Drill Rig: Soil Auger and Williamson Drive Probe (WDP)	
Geologist/Engineer	SMA	Ground Elevation	See Figures
Date Drilled	01-22-2022	Total Depth of Borehole	5.5 Feet
Borehole Diameter	1 OD Inches	Depth to Water	Not encountered

Graphic Log		Description / Lithology	Depth (feet)	Sample	SPT Blows*	Recovery (%)	Moisture (%)	DD (pcf)	LL (%), PL (%)	Swell (%)	Completion
	OL	Surface topsoil 6.0"									
	SM/ ML	SILTY SAND to SANDY SILT, fine grained, very loose to medium dense or soft to medium stiff, moist to very moist		X	2-2-2						
	SM/ GM	SILTY SAND with some GRAVEL, gray-brown, medium dense to dense, moist (Vashon Till)	2.5	X	50+						
		End of Borehole at 5.5 feet due to hard drilling/augering on gravel. Perched groundwater was NOT noted during exploration. Stable groundwater table was not encountered during or at the completion of drilling. At completion, borehole was backfilled with soil cuttings.	5.0								
			7.5								
			10.0								

B2

Project Number	0531-WA21	Drill Rig: Soil Auger and Williamson Drive Probe (WDP)	
Geologist/Engineer	SMA	Ground Elevation	See Figures
Date Drilled	01-22-2022	Total Depth of Borehole	10 Feet
Borehole Diameter	1 OD Inches	Depth to Water	Not encountered

Graphic Log		Description / Lithology	Depth (feet)	Sample	SPT Blows*	Recovery (%)	Moisture (%)	DD (pcf)	LL (%), PL (%)	Swell (%)	Completion
	ML/ SM	SILTY SAND to SANDY SILT, fine grained, loose to medium dense or medium stiff	2.5	X	2-3-4						
	SM/ ML	SILTY SAND to SANDY SILT, fine grained, medium stiff to stiff	5.0	X	4-5-46						
			7.5	X	11-12-20						
	SW/ GM	SILTY SAND with some GRAVEL, gray-brown, medium dense to dense, moist (Vashon Till)	10.0	X	50+						
		End of Borehole at 10.0 feet due to hard drilling/augering on gravel. Perched groundwater was noted at 3 feet during exploration. Stable groundwater table was not encountered during or at the completion of drilling. At completion, borehole was backfilled with soil cuttings.									

SLOPE STABILITY OUTPUTS

SLOPE/W Analysis

File Information

- Created By: Admin
- Last Edited By: Admin
- Revision Number: 1
- File Version: 8.3
- Tool Version: 8.13.4.13430
- Date: 24-01-2022
- Time: 10:46:01
- File Name: ANALYSIS.gsz
- Directory: G:\My Drive\AGS ENGINEERING\AGS 2021 ENG PROJECTS\0531-WA21 Ananta 3632 90th Avenue SE, Mercer Island, WA\Slope Stability Analysis\
- Last Solved Date: 24-01-2022
- Last Solved Time: 10:46:03

Project Settings

- Length(L) Units: Feet
- Time(t) Units: Seconds
- Force(F) Units: Pounds
- Pressure(p) Units: psf
- Strength Units: psf
- Unit Weight of Water: 62.4 pcf
- View: 2D
- Element Thickness: 1

Analysis Settings

SLOPE/W Analysis

- Kind: SLOPE/W
- Method: Morgenstern-Price
- Settings
  - Side Function
    - Interslice force function option: Half-Sine
  - Lambda
    - Lambda 1: -1
    - Lambda 2: -0.8
    - Lambda 3: -0.6
    - Lambda 4: -0.4
    - Lambda 5: -0.2
    - Lambda 6: 0
    - Lambda 7: 0.2
    - Lambda 8: 0.4
    - Lambda 9: 0.6
    - Lambda 10: 0.8
    - Lambda 11: 1
  - PWP Conditions Source: (none)
- Slip Surface
  - Direction of movement: Left to Right
  - Use Passive Mode: No
  - Slip Surface Option: Entry and Exit
  - Critical slip surfaces saved: 1
  - Optimize Critical Slip Surface Location: No

Tension Crack
Tension Crack Option: (none)
F of S Distribution
F of S Calculation Option: Constant
Advanced
Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2,000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

SOIL 1

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 250 psf
Phi': 32 °
Phi-B: 0 °

SOIL 2

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion': 750 psf
Phi': 34 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (6.8544, 100.19273) ft
Left-Zone Right Coordinate: (42.4, 99.91038) ft
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (157.02572, 74.63089) ft
Right-Zone Right Coordinate: (197.6, 59.34063) ft
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (-0.1, 100.1) ft
Right Coordinate: (219.9, -0.2) ft

Surcharge Loads

Surcharge Load 1

Surcharge (Unit Weight): 1,500 pcf
Direction: Vertical

Coordinates

	X (ft)	Y (ft)
	65.1	102.1
	109.9	98.4

Points

	X (ft)	Y (ft)
Point 1	-0.1	100.1
Point 2	29.9	100.5
Point 3	61.7	99
Point 4	79.3	96.1
Point 5	120.6	91.3
Point 6	128.1	88.7
Point 7	100	94.4
Point 8	139.6	84
Point 9	151.8	76.6
Point 10	165.6	71.4
Point 11	177.4	64
Point 12	191.8	60.7
Point 13	204.6	57.7
Point 14	216	55.1
Point 15	219.9	-0.2
Point 16	-0.1	79.8
Point 17	47.1	76.7
Point 18	85.1	72.1
Point 19	125.9	61.7
Point 20	162.8	51.2
Point 21	200.6	40.5
Point 22	217.5	37.9
Point 23	0.3	0.2

Regions

	Material	Points	Area (ft²)
Region 1	SOIL 1	1,16,17,18,19,20,21,22,14,13,12,11,10,9,8,6,5,7,4,3,2	4,889.6
Region 2	SOIL 2	16,23,15,22,21,20,19,18,17	13,758

Current Slip Surface

Slip Surface: 117
F of S: 2.180
Volume: 2,491.0244 ft³
Weight: 2,98,923.64 lbs
Resisting Moment: 7,66,48,393 lbs-ft
Activating Moment: 3,51,63,506 lbs-ft
Resisting Force: 3,91,451.13 lbs
Activating Force: 1,79,604.03 lbs
F of S Rank: 1
Exit: (186.92805, 61.816488) ft
Entry: (42.4, 99.910377) ft
Radius: 184.24867 ft
Center: (157.58725, 243.71395) ft

Slip Slices

	X (ft)	Y (ft)	PWP	Base Normal Stress	Frictional Strength	Cohesive Strength

			(psf)	(psf)	(psf)	(psf)
Slice 1	44.8125	98.042685	0	101.21758	63.247764	250
Slice 2	49.6375	94.430468	0	439.80529	274.82085	250
Slice 3	54.4625	91.056229	0	754.20911	471.28216	250
Slice 4	59.2875	87.904475	0	1,048.4305	655.13209	250
Slice 5	63.4	85.371223	0	1,267.367	791.93879	250
Slice 6	67.466667	83.032628	0	6,238.5984	3,898.3089	250
Slice 7	72.2	80.467345	0	6,940.8234	4,337.1078	250
Slice 8	76.933333	78.077039	0	7,654.1125	4,782.8203	250
Slice 9	81.8875	75.758413	0	8,163.1873	5,100.9256	250
Slice 10	87.0625	73.519988	0	8,463.7067	5,288.7109	250
Slice 11	92.2375	71.466102	0	8,771.1544	5,480.8255	250
Slice 12	97.4125	69.590217	0	9,083.2091	5,675.819	250
Slice 13	102.00737	68.060844	0	9,534.4983	5,957.8158	250
Slice 14	106.0221	66.840454	0	10,127.217	6,328.1872	250
Slice 15	108.96474	65.999345	0	10,573.588	7,131.9749	750
Slice 16	113.02417	64.964587	0	3,116.8948	2,102.3721	750
Slice 17	118.37417	63.700853	0	3,241.6686	2,025.6193	250
Slice 18	122.475	62.851982	0	3,288.651	2,054.9772	250
Slice 19	126.225	62.164071	0	3,270.9959	2,043.9451	250
Slice 20	130.975	61.420461	0	3,204.2784	2,002.2553	250
Slice 21	136.725	60.673072	0	3,073.8294	1,920.7418	250
Slice 22	141.63333	60.168652	0	2,882.4957	1,801.1832	250
Slice 23	145.7	59.860445	0	2,640.5169	1,649.9781	250
Slice 24	149.76667	59.642588	0	2,376.5876	1,485.0567	250
Slice 25	154.1	59.512656	0	2,138.0807	1,336.0211	250
Slice 26	158.7	59.483005	0	1,923.8353	1,202.1457	250
Slice 27	163.3	59.56825	0	1,686.8109	1,054.0364	250
Slice 28	168.55	59.815461	0	1,296.7131	810.27628	250
Slice 29	174.45	60.262482	0	751.96828	469.88193	250
Slice 30	179.78201	60.822713	0	363.30485	227.01807	250
Slice 31	184.54604	61.464135	0	135.75181	84.827143	250

SLOPE/W Analysis

File Information

Created By: Admin
Last Edited By: Admin
Revision Number: 2
File Version: 8.3
Tool Version: 8.13.4.13430
Date: 24-01-2022
Time: 10:47:36
File Name: ANALYSIS.gsz
Directory: G:\My Drive\AGS ENGINEERING\AGS 2021 ENG PROJECTS\0531-WA21 Ananta 3632 90th Avenue SE, Mercer Island, WA\Slope Stability Analysis\
Last Solved Date: 24-01-2022
Last Solved Time: 10:47:38

Project Settings

Length(L) Units: Feet
Time(t) Units: Seconds
Force(F) Units: Pounds
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D
Element Thickness: 1

Analysis Settings

SLOPE/W Analysis

Kind: SLOPE/W
Method: Morgenstern-Price
Settings
- Side Function
  - Interslice force function option: Half-Sine
- Lambda
  - Lambda 1: -1
  - Lambda 2: -0.8
  - Lambda 3: -0.6
  - Lambda 4: -0.4
  - Lambda 5: -0.2
  - Lambda 6: 0
  - Lambda 7: 0.2
  - Lambda 8: 0.4
  - Lambda 9: 0.6
  - Lambda 10: 0.8
  - Lambda 11: 1
- PWP Conditions Source: (none)

Slip Surface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Entry and Exit
- Critical slip surfaces saved: 1
- Optimize Critical Slip Surface Location: No

Tension Crack
Tension Crack Option: (none)
F of S Distribution
F of S Calculation Option: Constant
Advanced
Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2,000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

SOIL 1

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 250 psf
Phi': 32 °
Phi-B: 0 °

SOIL 2

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion': 750 psf
Phi': 34 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (28.0278, 100.47504) ft
Left-Zone Right Coordinate: (55.3, 99.30189) ft
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (137.07302, 85.03277) ft
Right-Zone Right Coordinate: (163.2, 72.30435) ft
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (-0.1, 100.1) ft
Right Coordinate: (219.9, -0.2) ft

Surcharge Loads

Surcharge Load 1

Surcharge (Unit Weight): 1,500 pcf
Direction: Vertical

Coordinates

	X (ft)	Y (ft)
	65.1	102.1
	109.9	98.4

Points

	X (ft)	Y (ft)
Point 1	-0.1	100.1
Point 2	29.9	100.5
Point 3	61.7	99
Point 4	79.3	96.1
Point 5	120.6	91.3
Point 6	128.1	88.7
Point 7	100	94.4
Point 8	139.6	84
Point 9	151.8	76.6
Point 10	165.6	71.4
Point 11	177.4	64
Point 12	191.8	60.7
Point 13	204.6	57.7
Point 14	216	55.1
Point 15	219.9	-0.2
Point 16	-0.1	79.8
Point 17	47.1	76.7
Point 18	85.1	72.1
Point 19	125.9	61.7
Point 20	162.8	51.2
Point 21	200.6	40.5
Point 22	217.5	37.9
Point 23	0.3	0.2

Regions

	Material	Points	Area (ft²)
Region 1	SOIL 1	1,16,17,18,19,20,21,22,14,13,12,11,10,9,8,6,5,7,4,3,2	4,889.6
Region 2	SOIL 2	16,23,15,22,21,20,19,18,17	13,758

Current Slip Surface

Slip Surface: 122
F of S: 2.059
Volume: 1,366.4296 ft³
Weight: 1,63,971.55 lbs
Resisting Moment: 4,30,69,180 lbs-ft
Activating Moment: 2,09,18,492 lbs-ft
Resisting Force: 2,97,820.45 lbs
Activating Force: 1,44,668.62 lbs
F of S Rank: 1
Exit: (163.2, 72.304348) ft
Entry: (55.3, 99.301887) ft
Radius: 135.50089 ft
Center: (139.24185, 205.67038) ft

Slip Slices

	X (ft)	Y (ft)	PWP	Base Normal Stress	Frictional Strength	Cohesive Strength

			(psf)	(psf)	(psf)	(psf)
Slice 1	56.9	98.077393	0	38.935404	24.329541	250
Slice 2	60.1	95.70142	0	263.60443	164.71833	250
Slice 3	63.4	93.401526	0	460.61492	287.82415	250
Slice 4	66.875	91.131647	0	5,273.659	3,295.3479	250
Slice 5	70.425	88.963543	0	5,796.6497	3,622.1487	250
Slice 6	73.975	86.941166	0	6,328.1815	3,954.2867	250
Slice 7	77.525	85.057173	0	6,872.1216	4,294.1782	250
Slice 8	81.025	83.328074	0	7,257.3917	4,534.9216	250
Slice 9	84.475	81.744894	0	7,479.5431	4,673.7372	250
Slice 10	87.925	80.276609	0	7,710.9866	4,818.3592	250
Slice 11	91.375	78.919224	0	7,950.6045	4,968.0891	250
Slice 12	94.825	77.669206	0	8,196.6271	5,121.821	250
Slice 13	98.275	76.523436	0	8,446.6027	5,278.0232	250
Slice 14	101.65	75.499745	0	8,835.7769	5,521.2062	250
Slice 15	104.95	74.591582	0	9,367.2073	5,853.2807	250
Slice 16	108.25	73.772224	0	9,901.2115	6,186.9636	250
Slice 17	111.68333	73.014051	0	2,308.7889	1,442.6914	250
Slice 18	115.25	72.322718	0	2,390.3307	1,493.6444	250
Slice 19	118.81667	71.729913	0	2,456.3135	1,534.875	250
Slice 20	122.475	71.224132	0	2,460.6266	1,537.5701	250
Slice 21	126.225	70.809324	0	2,398.2459	1,498.5904	250
Slice 22	130.01667	70.497541	0	2,294.137	1,433.5359	250
Slice 23	133.85	70.290397	0	2,146.4376	1,341.243	250
Slice 24	137.68333	70.192012	0	1,970.8872	1,231.547	250
Slice 25	141.63333	70.205859	0	1,710.606	1,068.9053	250
Slice 26	145.7	70.338788	0	1,366.1867	853.68821	250
Slice 27	149.76667	70.594253	0	999.44692	624.52375	250
Slice 28	153.7	70.956603	0	684.57702	427.7712	250
Slice 29	157.5	71.41892	0	426.00678	266.19858	250
Slice 30	161.3	71.990834	0	158.26214	98.893159	250

SLOPE/W Analysis

File Information

Created By: Admin
Last Edited By: Admin
Revision Number: 8
File Version: 8.3
Tool Version: 8.13.4.13430
Date: 06-02-2022
Time: 22:13:24
File Name: ANALYSIS.gsz
Directory: G:\My Drive\AGS ENGINEERING\AGS 2021 ENG PROJECTS\0531-WA21 Ananta 3632 90th Avenue SE, Mercer Island, WA\Slope Stability Analysis\
Last Solved Date: 06-02-2022
Last Solved Time: 22:13:25

Project Settings

Length(L) Units: Feet
Time(t) Units: Seconds
Force(F) Units: Pounds
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D
Element Thickness: 1

Analysis Settings

SLOPE/W Analysis

Kind: SLOPE/W
Method: Morgenstern-Price
Settings
- Side Function
  - Interslice force function option: Half-Sine
- Lambda
  - Lambda 1: -1
  - Lambda 2: -0.8
  - Lambda 3: -0.6
  - Lambda 4: -0.4
  - Lambda 5: -0.2
  - Lambda 6: 0
  - Lambda 7: 0.2
  - Lambda 8: 0.4
  - Lambda 9: 0.6
  - Lambda 10: 0.8
  - Lambda 11: 1
- PWP Conditions Source: (none)

Slip Surface
- Direction of movement: Left to Right
- Use Passive Mode: No
- Slip Surface Option: Entry and Exit
- Critical slip surfaces saved: 1
- Optimize Critical Slip Surface Location: No

Tension Crack
Tension Crack Option: (none)
F of S Distribution
F of S Calculation Option: Constant
Advanced
Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2,000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

SOIL 1

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 250 psf
Phi': 32 °
Phi-B: 0 °

SOIL 2

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion': 750 psf
Phi': 34 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (8.28675, 100.21182) ft
Left-Zone Right Coordinate: (64.12759, 98.6) ft
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (114.23258, 92.2582) ft
Right-Zone Right Coordinate: (175.8, 65.00339) ft
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (-0.1, 100.1) ft
Right Coordinate: (219.9, -0.2) ft

Surcharge Loads

Surcharge Load 1

Surcharge (Unit Weight): 1,500 pcf
Direction: Vertical

Coordinates

	X (ft)	Y (ft)
	65.1	102.1
	109.9	98.4

Points

	X (ft)	Y (ft)
Point 1	-0.1	100.1
Point 2	29.9	100.5
Point 3	61.7	99
Point 4	79.3	96.1
Point 5	120.6	91.3
Point 6	128.1	88.7
Point 7	100	94.4
Point 8	139.6	84
Point 9	151.8	76.6
Point 10	165.6	71.4
Point 11	177.4	64
Point 12	191.8	60.7
Point 13	204.6	57.7
Point 14	216	55.1
Point 15	219.9	-0.2
Point 16	-0.1	79.8
Point 17	47.1	76.7
Point 18	85.1	72.1
Point 19	125.9	61.7
Point 20	162.8	51.2
Point 21	200.6	40.5
Point 22	217.5	37.9
Point 23	0.3	0.2

Regions

	Material	Points	Area (ft²)
Region 1	SOIL 1	1,16,17,18,19,20,21,22,14,13,12,11,10,9,8,6,5,7,4,3,2	4,889.6
Region 2	SOIL 2	16,23,15,22,21,20,19,18,17	13,758

Current Slip Surface

Slip Surface: 122
F of S: 1.623
Volume: 1,524.4438 ft³
Weight: 1,82,933.25 lbs
Resisting Moment: 4,80,09,646 lbs-ft
Activating Moment: 2,95,88,091 lbs-ft
Resisting Force: 2,93,354.49 lbs
Activating Force: 1,80,810.2 lbs
F of S Rank: 1
Exit: (175.8, 65.00339) ft
Entry: (64.12759, 98.599999) ft
Radius: 148.56401 ft
Center: (159.32996, 212.65163) ft

Slip Slices

	X (ft)	Y (ft)	PWP	Base Normal Stress	Frictional Strength	Cohesive Strength

			(psf)	(psf)	(psf)	(psf)
Slice 1	64.613795	98.197641	0	-65.687433	-41.046064	250
Slice 2	66.875	96.383822	0	4,371.2337	2,731.4499	250
Slice 3	70.425	93.646316	0	4,873.1788	3,045.1001	250
Slice 4	73.975	91.074112	0	5,378.6851	3,360.9755	250
Slice 5	77.525	88.656904	0	5,893.7106	3,682.7991	250
Slice 6	81.025	86.415826	0	6,263.5632	3,913.9087	250
Slice 7	84.475	84.339463	0	6,487.4256	4,053.7934	250
Slice 8	87.925	82.387562	0	6,726.0153	4,202.8808	250
Slice 9	91.375	80.554598	0	6,979.8954	4,361.5227	250
Slice 10	94.825	78.83568	0	7,249.05	4,529.7092	250
Slice 11	98.275	77.226459	0	7,532.8195	4,707.028	250
Slice 12	101.65	75.753559	0	7,955.6376	4,971.2341	250
Slice 13	104.95	74.409301	0	8,521.8957	5,325.0715	250
Slice 14	108.25	73.156055	0	9,106.2104	5,690.1918	250
Slice 15	111.68333	71.947972	0	2,150.5897	1,343.8376	250
Slice 16	115.25	70.789954	0	2,314.0749	1,445.9945	250
Slice 17	118.81667	69.730314	0	2,469.9334	1,543.3857	250
Slice 18	122.475	68.744608	0	2,576.5441	1,610.0034	250
Slice 19	126.225	67.83579	0	2,627.3888	1,641.7748	250
Slice 20	130.01667	67.021367	0	2,642.212	1,651.0373	250
Slice 21	133.85	66.301871	0	2,616.2362	1,634.8058	250
Slice 22	137.68333	65.685867	0	2,560.0615	1,599.704	250
Slice 23	141.63333	65.159594	0	2,417.6444	1,510.7119	250
Slice 24	145.7	64.728272	0	2,184.3116	1,364.9094	250
Slice 25	149.76667	64.409744	0	1,914.2395	1,196.1496	250
Slice 26	153.525	64.211112	0	1,686.9571	1,054.1278	250
Slice 27	156.975	64.116305	0	1,511.5629	944.52933	250
Slice 28	160.425	64.101671	0	1,318.2201	823.71531	250
Slice 29	163.875	64.167189	0	1,110.2323	693.75014	250
Slice 30	167.3	64.311328	0	838.63431	524.03688	250
Slice 31	170.7	64.533163	0	509.49383	318.36708	250
Slice 32	174.1	64.833525	0	180.75594	112.94885	250

SLOPE/W Analysis

File Information

- Created By: Admin
- Last Edited By: Admin
- Revision Number: 15
- File Version: 8.3
- Tool Version: 8.13.4.13430
- Date: 06-02-2022
- Time: 22:19:23
- File Name: ANALYSIS.gsz
- Directory: G:\My Drive\AGS ENGINEERING\AGS 2021 ENG PROJECTS\0531-WA21 Ananta 3632 90th Avenue SE, Mercer Island, WA\Slope Stability Analysis\
- Last Solved Date: 06-02-2022
- Last Solved Time: 22:19:24

Project Settings

- Length(L) Units: Feet
- Time(t) Units: Seconds
- Force(F) Units: Pounds
- Pressure(p) Units: psf
- Strength Units: psf
- Unit Weight of Water: 62.4 pcf
- View: 2D
- Element Thickness: 1

Analysis Settings

SLOPE/W Analysis

- Kind: SLOPE/W
- Method: Morgenstern-Price
- Settings
  - Side Function
    - Interslice force function option: Half-Sine
  - Lambda
    - Lambda 1: -1
    - Lambda 2: -0.8
    - Lambda 3: -0.6
    - Lambda 4: -0.4
    - Lambda 5: -0.2
    - Lambda 6: 0
    - Lambda 7: 0.2
    - Lambda 8: 0.4
    - Lambda 9: 0.6
    - Lambda 10: 0.8
    - Lambda 11: 1
  - PWP Conditions Source: (none)
- Slip Surface
  - Direction of movement: Left to Right
  - Use Passive Mode: No
  - Slip Surface Option: Entry and Exit
  - Critical slip surfaces saved: 1
  - Optimize Critical Slip Surface Location: No

Tension Crack
Tension Crack Option: (none)
F of S Distribution
F of S Calculation Option: Constant
Advanced
Number of Slices: 30
F of S Tolerance: 0.001
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2,000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

SOIL 1

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 250 psf
Phi': 32 °
Phi-B: 0 °

SOIL 2

Model: Mohr-Coulomb
Unit Weight: 125 pcf
Cohesion': 750 psf
Phi': 34 °
Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (22.68697, 100.40383) ft
Left-Zone Right Coordinate: (76.5, 96.56136) ft
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (117.65388, 91.74335) ft
Right-Zone Right Coordinate: (157.63846, 74.4) ft
Right-Zone Increment: 4
Radius Increments: 4

Slip Surface Limits

Left Coordinate: (-0.1, 100.1) ft
Right Coordinate: (219.9, -0.2) ft

Surcharge Loads

Surcharge Load 1

Surcharge (Unit Weight): 1,500 pcf
Direction: Vertical

Coordinates

	X (ft)	Y (ft)
	65.1	102.1
	109.9	98.4

Points

	X (ft)	Y (ft)
Point 1	-0.1	100.1
Point 2	29.9	100.5
Point 3	61.7	99
Point 4	79.3	96.1
Point 5	120.6	91.3
Point 6	128.1	88.7
Point 7	100	94.4
Point 8	139.6	84
Point 9	151.8	76.6
Point 10	165.6	71.4
Point 11	177.4	64
Point 12	191.8	60.7
Point 13	204.6	57.7
Point 14	216	55.1
Point 15	219.9	-0.2
Point 16	-0.1	79.8
Point 17	47.1	76.7
Point 18	85.1	72.1
Point 19	125.9	61.7
Point 20	162.8	51.2
Point 21	200.6	40.5
Point 22	217.5	37.9
Point 23	0.3	0.2

Regions

	Material	Points	Area (ft²)
Region 1	SOIL 1	1,16,17,18,19,20,21,22,14,13,12,11,10,9,8,6,5,7,4,3,2	4,889.6
Region 2	SOIL 2	16,23,15,22,21,20,19,18,17	13,758

Current Slip Surface

Slip Surface: 123
F of S: 1.534
Volume: 1,231.6017 ft³
Weight: 1,47,792.21 lbs
Resisting Moment: 1,65,92,931 lbs-ft
Activating Moment: 1,08,16,067 lbs-ft
Resisting Force: 2,26,499.81 lbs
Activating Force: 1,47,771.96 lbs
F of S Rank: 1
Exit: (157.63846, 74.400001) ft
Entry: (76.500001, 96.561364) ft
Radius: 63.109542 ft
Center: (129.46723, 130.87297) ft

Slip Slices

	X (ft)	Y (ft)	PWP	Base Normal Stress	Frictional Strength	Cohesive Strength

			(psf)	(psf)	(psf)	(psf)
Slice 1	77.9	94.572567	0	4,333.3837	2,707.7987	250
Slice 2	80.59375	90.99806	0	4,726.9029	2,953.6968	250
Slice 3	83.18125	88.015114	0	4,993.4775	3,120.271	250
Slice 4	85.76875	85.37529	0	5,261.2592	3,287.5996	250
Slice 5	88.35625	83.0212	0	5,539.6921	3,461.5838	250
Slice 6	90.94375	80.912121	0	5,835.6185	3,646.4991	250
Slice 7	93.53125	79.017965	0	6,154.1534	3,845.5418	250
Slice 8	96.11875	77.315804	0	6,499.089	4,061.0816	250
Slice 9	98.70625	75.787761	0	6,873.0358	4,294.7494	250
Slice 10	101.2375	74.446189	0	7,362.5785	4,600.6496	250
Slice 11	103.7125	73.273749	0	7,975.2088	4,983.4636	250
Slice 12	106.1875	72.229169	0	8,628.4291	5,391.6409	250
Slice 13	108.6625	71.305705	0	9,320.8291	5,824.3004	250
Slice 14	111.2375	70.469831	0	2,295.682	1,434.5014	250
Slice 15	113.9125	69.725936	0	2,536.2059	1,584.7974	250
Slice 16	116.5875	69.106798	0	2,773.1762	1,732.8728	250
Slice 17	119.2625	68.608689	0	3,000.6614	1,875.0213	250
Slice 18	121.85	68.237466	0	3,173.1707	1,982.8171	250
Slice 19	124.35	67.98374	0	3,280.9449	2,050.1619	250
Slice 20	126.85	67.830134	0	3,354.9782	2,096.4231	250
Slice 21	129.5375	67.779841	0	3,374.8453	2,108.8374	250
Slice 22	132.4125	67.84862	0	3,322.7943	2,076.3123	250
Slice 23	135.2875	68.048973	0	3,199.3179	1,999.1557	250
Slice 24	138.1625	68.382169	0	3,001.7678	1,875.7127	250
Slice 25	140.82	68.80534	0	2,718.7571	1,698.868	250
Slice 26	143.26	69.301785	0	2,368.7484	1,480.1583	250
Slice 27	145.7	69.89988	0	1,981.7248	1,238.3191	250
Slice 28	148.14	70.602656	0	1,568.3121	979.99016	250
Slice 29	150.58	71.41383	0	1,139.8785	712.27514	250
Slice 30	153.25962	72.441359	0	715.86437	447.3217	250
Slice 31	156.17885	73.717845	0	303.82151	189.84875	250

King County Area, Washington

AmC—Arents, Alderwood material, 6 to 15 percent slopes

Map Unit Setting

National map unit symbol: 1hmsq
Elevation: 50 to 660 feet
Mean annual precipitation: 35 to 60 inches
Mean annual air temperature: 50 degrees F
Frost-free period: 150 to 200 days
Farmland classification: Prime farmland if irrigated

Map Unit Composition

Arents, alderwood material, and similar soils: 100 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Arents, Alderwood Material

Setting

Landform: Till plains
Parent material: Basal till

Typical profile

H1 - 0 to 26 inches: gravelly sandy loam
H2 - 26 to 60 inches: very gravelly sandy loam

Properties and qualities

Slope: 6 to 15 percent
Depth to restrictive feature: 20 to 40 inches to densic material
Drainage class: Moderately well drained
Capacity of the most limiting layer to transmit water (Ksat): Very low to moderately low (0.00 to 0.06 in/hr)
Depth to water table: About 16 to 36 inches
Frequency of flooding: None
Frequency of ponding: None
Available water supply, 0 to 60 inches: Very low (about 2.3 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 4s
Hydrologic Soil Group: B/D
Hydric soil rating: No

Data Source Information

Soil Survey Area: King County Area, Washington
Survey Area Data: Version 17, Aug 23, 2021

King County Area, Washington

KpD—Kitsap silt loam, 15 to 30 percent slopes

Map Unit Setting

National map unit symbol: 1hmtc
Elevation: 0 to 590 feet
Mean annual precipitation: 37 inches
Mean annual air temperature: 50 degrees F
Frost-free period: 160 to 200 days
Farmland classification: Farmland of statewide importance

Map Unit Composition

Kitsap and similar soils: 97 percent
Minor components: 3 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Kitsap

Setting

Landform: Terraces
Parent material: Lacustrine deposits with a minor amount of volcanic ash

Typical profile

H1 - 0 to 5 inches: silt loam
H2 - 5 to 40 inches: silt loam
H3 - 40 to 60 inches: stratified silt to silty clay loam

Properties and qualities

Slope: 15 to 30 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Moderately well drained
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: About 18 to 36 inches
Frequency of flooding: None
Frequency of ponding: None
Available water supply, 0 to 60 inches: High (about 11.4 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 4e
Hydrologic Soil Group: C
Ecological site: F002XA004WA - Puget Lowlands Forest
Forage suitability group: Sloping to Steep Soils (G002XN702WA)
Other vegetative classification: Sloping to Steep Soils (G002XN702WA)
Hydric soil rating: No

Minor Components

Bellingham

Percent of map unit: 1 percent
Landform: Depressions
Other vegetative classification: Wet Soils (G002XN102WA)
Hydric soil rating: Yes

Seattle

Percent of map unit: 1 percent
Landform: Depressions
Other vegetative classification: Wet Soils (G002XN102WA)
Hydric soil rating: Yes

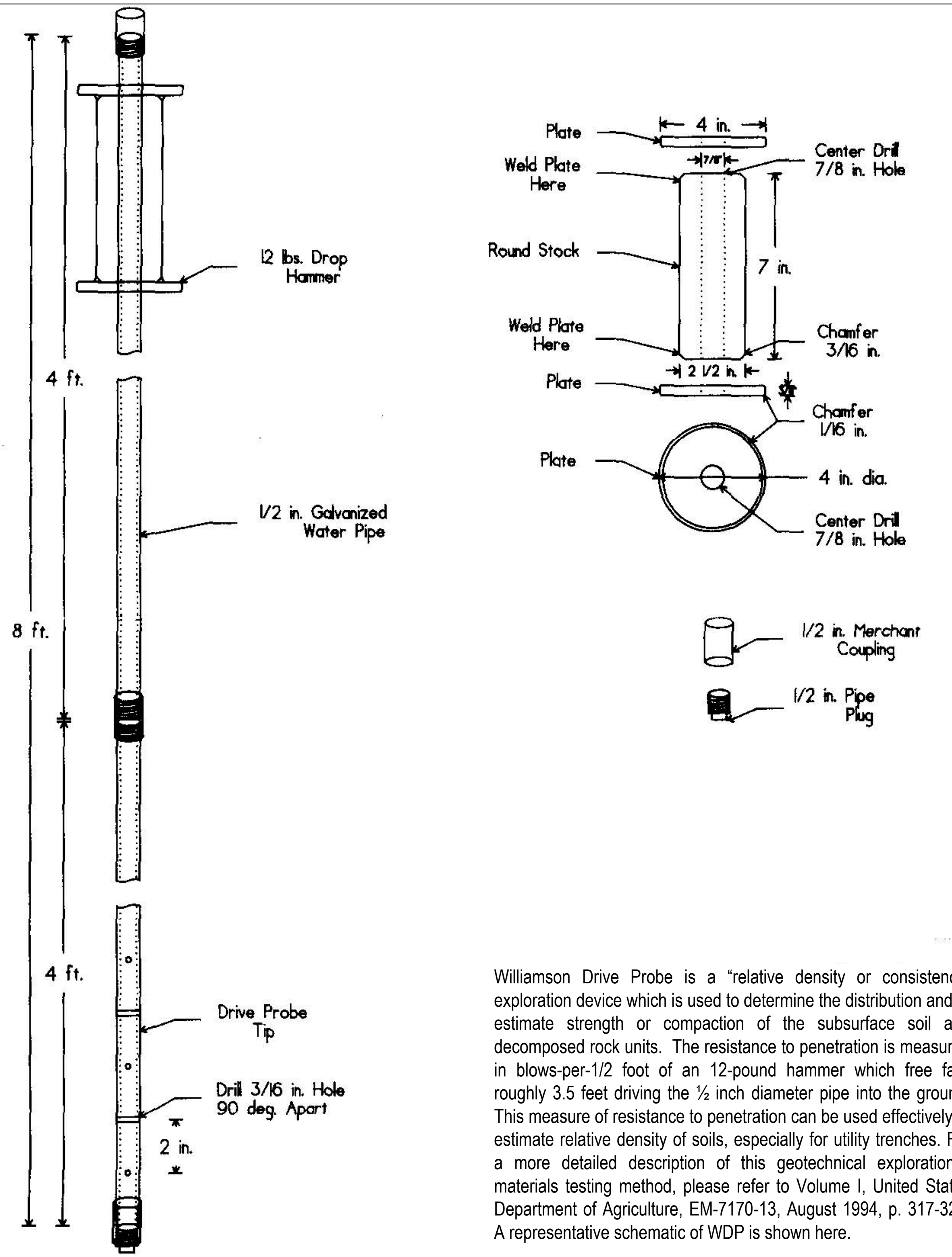
Tukwila

Percent of map unit: 1 percent
Landform: Depressions
Other vegetative classification: Wet Soils (G002XN102WA)
Hydric soil rating: Yes

Data Source Information

Soil Survey Area: King County Area, Washington
Survey Area Data: Version 17, Aug 23, 2021

Drive Prove Schematic

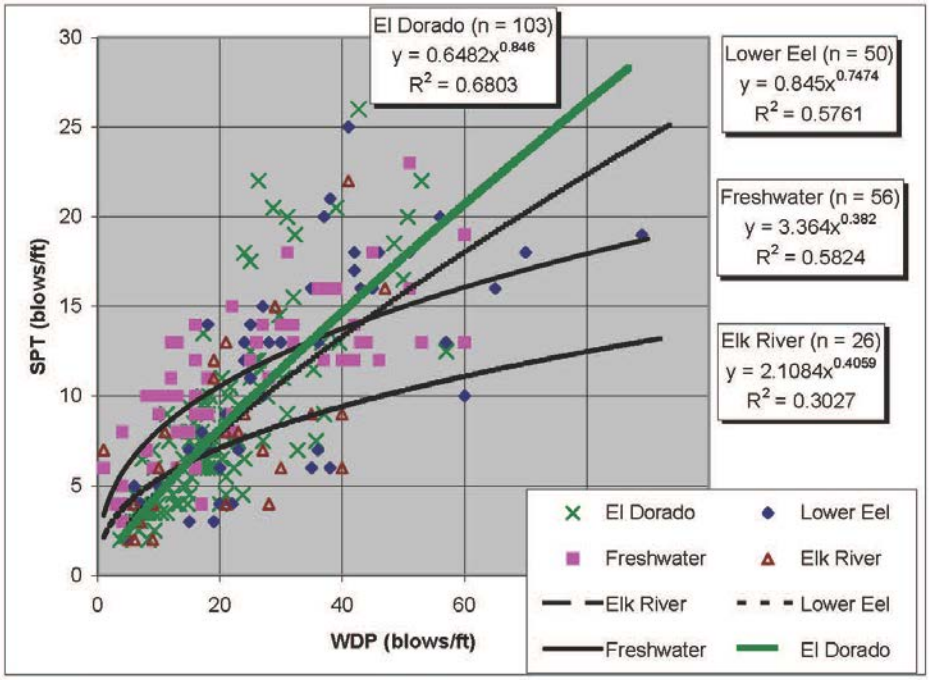


Williamson Drive Probe is a "relative density or consistency" exploration device which is used to determine the distribution and to estimate strength or compaction of the subsurface soil and decomposed rock units. The resistance to penetration is measured in blows-per-1/2 foot of an 12-pound hammer which free falls roughly 3.5 feet driving the ½ inch diameter pipe into the ground. This measure of resistance to penetration can be used effectively to estimate relative density of soils, especially for utility trenches. For a more detailed description of this geotechnical exploration / materials testing method, please refer to Volume I, United States Department of Agriculture, EM-7170-13, August 1994, p. 317-321. A representative schematic of WDP is shown here.

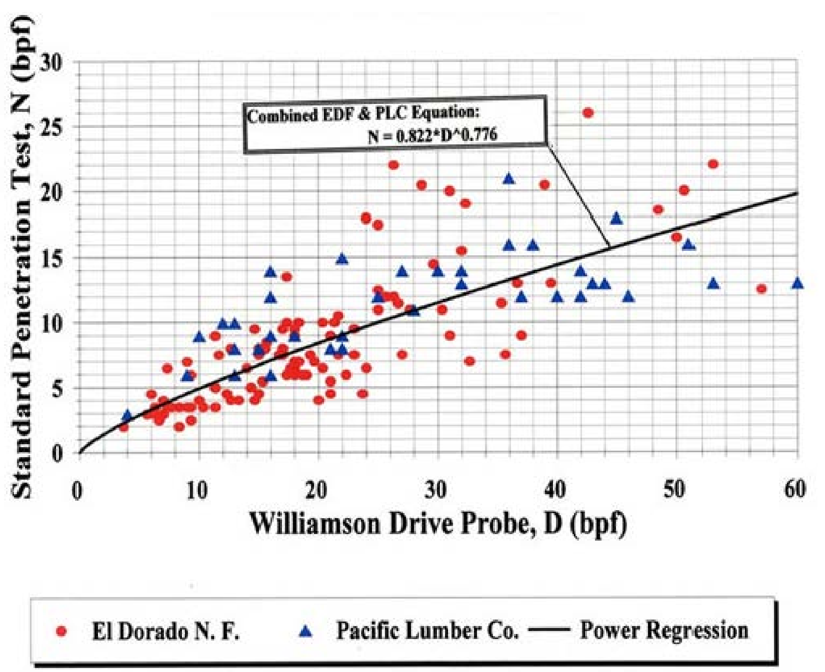
Reference: Figure 3.6.1, Appendix 3.6, Page No. 321, USDA EM-7170-13, August 1994, Volume I

CORRELATION

WILLIAMSON DRIVE PROBE (WDP)
STANDARD PENETRATION TEST (SPT)



SOURCE: Published literature by W.C. Adams (Hart Crowser, Inc.), R.W. Prellwitz (Bitterroot Geotechnical) & T.E. Koler (El Dorado National Forest).



SOURCE: United States Forest Service, Technology Development Program Website

(http://www.fs.fed.us/t-d/programs/im/williamson_drive/correlation.shtml)

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

Subsurface Explorations

Soil Testing

Earthwork Monitoring

Geotechnology

Foundation Engineering

Rock Mechanics

Earthquake Engineering

Geophysics

Retaining Wall Design

Geostrructural Design

Pavement Design

Drainage Evaluations

Groundwater Studies

Environmental Assets

Building Assessments

AMERICANGEOSERVICES.COM