EXHIBIT L

GEOTECHNICAL REPORT

Geotechnical Engineering Services Report

North Base Maintenance Facility
21711 Vetter Road NW
Poulsbo, Washington

for

TCF Architecture, PLLC

June 28, 2024

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GeoEngineers

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Poulsbo, Washington

File No. 06193-025-00
June 28, 2024

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1.0 Introduction

GeoEngineers is pleased to provide this report summarizing the results of our geotechnical engineering services for the Kitsap Transit (KT) North Base Maintenance Facility Design project (KT No. 23-820). The proposed project will add capability for vehicle maintenance and increased operational capacity to existing transit facilities located at 21711 Vetter Road NW in Poulsbo, Washington. A general overview of the project site in relation to the surrounding area is shown in the Vicinity Map, Figure 1.

The existing North Base facility consists of three parcels we consider as north, central and south. The southern parcel is developed with the North Viking Park & Ride and primarily contains paved parking and driveways. The northern parcel is developed with bus facilities, including a washing building, fueling island and bus parking. The center parcel includes passenger vehicle parking and a single-story operations building (4,000 square feet [sf]) consisting of driver facilities, dispatch and office space. We refer to this center parcel as the project site and/or project area herein as it relates to this report.

Our understanding of the project is based on our review of provided documents and our discussions with Kitsap Transit and the project team, including e-mails, phone calls and attending meetings. We have also reviewed previous geotechnical studies completed at the North Base facility for information regarding site conditions. Proposed improvements will include a new maintenance facility (15,400 sf) situated in the northeast corner of the project site and a new shared spaces addition (2,900 sf) in the north-central portion of the site. These new structures will be connected to the existing operations building, which will remain in its location and be remodeled. Paved personnel parking stalls will be located south of the existing and new buildings. Ruth Haines Road will extend east-west along the south site boundary. A covered storage outbuilding (1,100 sf) will be located along the adjacent north parcel boundary. Retaining walls are envisioned adjacent to the southern edge of the maintenance facility as site grades decrease to the south; walls may also be needed in other portions of the site. We were most recently provided open house drawings dated June 2024 and prepared by TCF Architecture PLLC (TCF) which include a conceptual site layout of the proposed improvements. The Site Plan, Figure 2, includes a more detailed view of existing site conditions and outlines of conceptual locations of the primary proposed improvements.

We were also provided and reviewed a preliminary topographic survey of the site dated June 20, 2023 and prepared by LDC, Inc. (project civil engineer). A vertical datum was not found on the plans but assumed to represent North American Vertical Datum 1988 (NAVD88). Elevations referenced herein are based on the provided survey and should be considered approximate.

2.0 Purpose and Scope of Services

The purpose of our geotechnical engineering services is to review relevant subsurface information in the vicinity and complete subsurface explorations (test pits) at the site to develop an understanding of soil and groundwater conditions. Based on our review and completed explorations, we provide geotechnical recommendations for design and construction primarily related to civil and structural elements. This report provides a summary of our review, completed explorations and geotechnical recommendations.

Our services have been provided in general accordance with the agreement between TCF Architecture, PLLC and GeoEngineers dated June 14, 2023. Details regarding our specific scope of services can be reviewed in the agreement or provided upon request.

The total of our services for this project consists of both geotechnical and environmental/ecological permitting tasks. A summary of our environmental permitting services and results will be provided in a separate report. Please refer to the environmental services report for additional information that may affect site layout, design and construction.

3.0 Site Conditions

3.1 SURFACE CONDITIONS

The site is generally bound by Vetter Road Northwest (north and east), Viking Avenue Northwest (west) and the North Viking Park & Ride (south). State Routes (SR) 3 and 305 NE are located within about a ¼ mile west and south of the site, respectively. Surrounding properties consist of light urban development, including single family residential homes and commercial businesses, however, some properties remain undeveloped and forested.

The approximate western half of the site is currently developed with the existing operations building (a one-story structure) surrounded by asphalt paved driveway and parking areas. The eastern portion of the site is surfaced with grass lawns and forested areas adjacent to Vetter Road Northwest. Overall surface grades across the site decrease from northwest (approximate Elevation 195 feet at the intersection of Viking Avenue and Vetter Road) to southeast (approximate Elevation 175 feet). Surface grades within the developed western site area (operations building and associate parking) varies between approximate Elevation 182 and 186 feet. Undeveloped areas in the south and eastern portions of the site include relatively gentle sloping swales/depressions with surfaces as low as about Elevation 170 feet. Intermittent tall coniferous trees, grass landscaping trails and areas of natural brush pockets are present in parts of the east quarter portion of the site.

3.2 LITERATURE REVIEW

3.2.1 Geologic Setting

Our understanding of the site geology is based, in part, on the review of published geologic maps and our experience. Mapped soils underlying the site and immediate surrounding area consist of glacial till or glacial ice-contact deposits. Glacial till typically consists of a cemented clay, silt and sand soil matrix with gravel, cobbles and boulders interspersed throughout. Ice-contact deposits are commonly described as subglacial till. These soils are each deposited directly below glacial ice and glacially overridden (glacially consolidated) and are generally characterized by relatively high strength, high allowable bearing pressures and low compressibility. Densities typically vary between dense to very dense; however, the upper few feet can be weathered and relatively less dense.

Other units mapped nearby include glacial recessional outwash, consisting of variably sorted silt, clay, sand and gravel deposited by meltwater at the terminus of a receding glacier. These deposits are generally medium dense to dense, but occasionally in a loose condition. Recessional outwash is typically underlain by glacial till or other glacially consolidated deposits.

3.2.2 Natural Resources Conservation Service Description

According to the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Web Soil Survey (WSS), the site is mapped as underlain by Poulsbo gravelly sandy loam. This soil type is described as a derivative of glacial till. Other units mapped nearby include derivatives of glacial outwash and alluvium (creek/river deposits).

3.2.3 Previous GeoEngineers Work

GeoEngineers has been involved with previous development at the North Base facility, including:

- "**Revised Infiltration Assessment, Proposed North Base Park and Ride**" (January 4, 2012). Our services included advancing 15 test pit explorations across the three Kitsap Transit owned parcels, including two test pits on the current site. Soil conditions encountered generally consisted of 2.5 to 7 feet of glacial outwash, underlain by dense glacial till deposits. Groundwater was not reported in the excavations.
- "**Kitsap Transit North Base Park and Ride, Stormwater Pond Monitoring Results**" (August 29, 2017). This report provides water level monitoring data and observed infiltration rates from four stormwater ponds extending along the eastern boundaries of the north and south parcels.

Approximate locations of previous site work (test pits and stormwater pond monitoring locations) are provided in Figure 3. Logs of test pits completed at the current project site (TP-10 and TP-11) are reproduced in Appendix B. Additional details of previous site work and conditions can be reviewed in the above studies.

3.2.4 Other Geotechnical Reports and Water Well Logs

Our document review also included the following:

- Zipper Zeman Associates, Inc. (July 6, 2001). "**Report of Geotechnical Services, SR3/SR305 Interchange Off-Site Improvements**". Includes summary logs of 11 borings completed to depths up to about 51 feet below grade, corresponding to approximate Elevation 202 feet. About 0.3 miles southwest of the North Base site.
- Ecology (1998). "**Water Well Report 73720**". Drilling advanced to 383 feet below grade, corresponding to approximate Elevation -193 feet. Approximately 0.45 miles southeast of the North Base site.
- Ecology (2011). "**Water Well Report 737027**". Drilling advanced to 248 feet below grade, corresponding to approximate Elevation 56 feet. About 0.7 miles northwest of the North Base site.

Overall, reviewed boring and well logs indicate soil conditions in the vicinity consist of dense to very dense glacial till soils at depth. Variable fill thicknesses, assumed to be associated with SR3 embankments, were reported in the Zipper Zeman borings. Static groundwater levels were reported at depths of 144 and 146 feet, corresponding to approximate Elevation 160 and 44 feet, respectively. Intermittent groundwater seepage was noted at shallower depths in some (but not all) of the borings, however, this seepage appears to be discontinuous and was observed to coincide with the transition from upper fill/topsoil layers to underlying glacial till.

3.3 SUBSURFACE CONDITIONS

3.3.1 Completed Subsurface Explorations

We explored subsurface conditions at the site by excavating ten test pits (TP-1 through TP-10) on July 19 and 20, 2023. Test pit locations were selected to target the proposed development areas and are shown in the Site Plan, Figure 2. Excavations were advanced to depths between approximately 5 and 9.5 feet below ground surface (bgs), corresponding to approximate Elevation 178 to 170.5, at which point practical digging refusal was encountered. A small-scale pilot infiltration test (PIT) was also completed at an intermediate depth in TP-2 and TP-5 to evaluate infiltration characteristics. Details regarding the subsurface exploration program, including summary logs of the explorations and results of our laboratory testing program are provided in Appendix A.

3.3.2 Soil Conditions

We observed what we interpret to be three general soil units in the completed test pits: fill, recessional outwash and glacial till deposits.

3.3.2.1 SURFACE LAYERS

Eight test pits (TP-1 through TP-8) were within landscaped grass lawn areas and encountered approximately 1 to 3 inches of sod at ground surface. Test pits TP-9 and TP-10 were located within undeveloped wooded areas of the site and encountered approximately 4 inches of forest duff at ground surface.

3.3.2.2 FILL

Soils we interpret to be fill were intermittently observed throughout the site in eight test pit excavations (TP-1, -3 through -8, and -10). Our determination of fill material was based on observations of soil type, layering and coloration. In general, fill soils consisted of loose to medium dense silty fine sand with variable gravel and organics (roots). Fill thickness was typically up to about 2 feet, except at TP-3 where approximately 5 feet of fill was encountered.

We interpret fill material encountered to be associated with previous site grading and development, fill depths could vary across the site and may be greater than those encountered at the exploration locations.

3.3.2.3 RECESSIONAL OUTWASH

We observed glacial recessional outwash deposits in three test pits (TP-3, -8, and -9) underneath fill soils. Outwash thickness varied between about 0.5 and 4 feet at test pit locations. Outwash consisted of medium dense sand with silt and gravel. These soils were relatively "cleaner" (fewer fine-grained silt and clay sized particles) than fill or glacial till deposits.

3.3.2.4 TILL

Soils we interpret to be glacial till deposits were observed at depth in all ten test pits, consisting of silty sand with variable gravel and cobble content. A boulder was observed within TP-4 at approximately 5.5 feet bgs. The top of glacial till soils was observed between about 0.25 and 6 feet bgs. In eight test pits, the upper approximate 2 to 3.5 feet of till was noted as brown to gray in coloration with occasional iron-oxide staining and densities ranging from stiff to medium dense; these observations are consistent with what we interpret to be weathered glacial till deposits. In two test pits (TP-3 and TP-8), no weathering was observed.

Gray dense to very dense glacial till with a cemented soil matrix was observed in all ten test pits, indicating an intact condition. The top of intact till was observed between about 2 to 6 feet bgs and extended to the depths explored.

3.3.3 Groundwater Conditions

We did not observe groundwater seepage and/or wet soils at the time of excavation. Occasional soil coloring and iron-oxide staining was observed in many of the explorations (as noted in the logs). In our opinion, these observations indicate the presence and fluctuation of groundwater seepage at various times of the year due to perched groundwater conditions. Based on the conditions encountered and our understanding of regional groundwater levels we do not anticipate static or regional groundwater levels will rise higher than the bottom of our explorations.

4.0 Geotechnical Design Recommendations

4.1 SUMMARY OF GEOTECHNICAL CONSIDERATIONS

Provided the recommendations in this report are included in design and construction, it is our opinion the proposed maintenance facility development is feasible at the site from a geotechnical engineering standpoint. We did not identify geotechnical related conditions that would prohibit or significantly constrain overall project design and construction, however, project layout and design will need to incorporate geotechnical considerations as discussed herein. A summary of key geotechnical considerations is provided in the bulleted list below. This summary is presented for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report.

- **Soil Conditions**: We interpret the site to be generally underlain by dense glacial till soils, however, relatively less dense layers are present in the upper few feet near existing grade.
  - Loose to medium dense fill, recessional outwash and/or weathered till layers were intermittently encountered throughout the site, with thicknesses up to about 6 feet at test pit locations. We anticipate depths and extents could vary across the site and may be greater than those encountered at the exploration locations.
  - Dense to very dense intact glacial till soils were encountered at depths below about 2 to 6 feet at test pit locations.
- **Groundwater Conditions**: We recommend perched groundwater conditions be considered during design and construction. We anticipate perched groundwater (if encountered) will be discontinuous, intermittent and will vary depending on a variety of conditions including season, rainfall events and infiltration of surface water that slows or terminates atop underlying glacial till. We do not expect static groundwater to be encountered within excavations completed for this project.
- **Geologic Critical Areas:** Based on our review, it is our opinion the risk of landslide and erosion hazards at the site are low. Seismic hazards are addressed below.
- **Seismic Design**: Based on the shallow depths observed to dense glacial till, we recommend using a seismic response spectrum for Site Class C. Recommended seismic design parameters are provided. In our opinion, the risk of soil liquefaction, lateral spreading, and/or surface rupture at the site is low.

- **Foundation Support:** Proposed new structures can be supported on shallow foundations and/or slabs-on-grade bearing on medium dense or denser soils and prepared as recommended herein. If relatively loose soils are present, some remediation or removal/replacement could be required.
- **Infiltration**: We anticipate limited infiltration capacity at the site due to relatively shallow depths to glacial till (low permeability layer). Recommended design infiltration rate is provided.
- **Earthwork:** Existing site soils generally contain a significant quantity of fine-grained (silt and clay size) particles and will be difficult or impossible to work with when wet. For general planning purposes, we recommended existing site soil not be considered for re-use as structural fill during wet weather; re-use during extended periods of dry weather can be considered upon review of a detailed work plan.

4.2 GEOLOGIC CRITICAL AREAS AND HAZARDS ASSESSMENT

4.2.1 Critical Areas

Chapter 19.400 of the Kitsap County Code defines geotechnical related critical areas as seismic (earthquake), landslide and erosion hazard areas. We address geotechnical critical areas below.

The potential presence of wetlands, habitat conservation areas and other ecological critical areas should also be addressed for the project, as needed. A summary of our environmental permitting services and results will be provided in a separate report. Please refer to the environmental services report, when it is available, for additional information that may affect site layout, design and construction.

4.2.2 Geological Hazards

Landslide and erosion hazard areas are not mapped at the site in the literature reviewed. Based on reviewed site topography, our observations while on site and completed subsurface explorations, it is our opinion that the risk of landslide and erosion hazards at this site is low. Existing slopes have shallow inclinations and heights are less than about 10 feet. Additionally, it appears that final grading will include removal of significant slopes or grade changes where new engineered retaining walls are not installed. We anticipate potential landslide and erosion hazards at this site can be managed through engineering design controls (e.g., site layout, site grading, horizontal setbacks). It is our opinion that landslide and erosion hazards as defined by Chapter 19.400 of the Kitsap County Code are not a limiting factor in the proposed development.

We envision standard code based seismic design (IBC and ASCE 7-16) is appropriate for this site. It is our opinion that site specific earthquake considerations, such as alternative foundations, ground improvements and/or alternative structural design will not be required. Detailed seismic hazards and design considerations are discussed in Section 4.3 below.

4.3 SEISMIC DESIGN CONSIDERATIONS

4.3.1 Seismic Design Parameters

The 2021 edition of the International Building Code (IBC) states structures shall be designed and constructed to resist the effects of earthquake ground motions in accordance with American Society of Civil Engineers (ASCE) 7-16. We evaluated seismic site response parameters in accordance with ASCE 7-16 using map-based methods as recommended by ASCE 7-16 and the United States Geologic Survey.

Completed test pits at the site encountered dense to very dense intact glacial till soils within about 6 feet of existing ground surface and were terminated within intact glacial till soils. We anticipate soils below our explorations and extending to depths of 100 feet bgs consist of similarly dense to very dense glacially consolidated deposits. Based on subsurface conditions observed in the test pits, our review of nearby subsurface information and data and our experience in the area, we recommend using a design response spectrum for Site Class C. Table 1 below provides recommended seismic design parameters.

TABLE 1. RECOMMENDED SEISMIC DESIGN PARAMETERS

2021 IBC (ASCE 7-16) SEISMIC DESIGN PARAMETERS[1]	RECOMMENDED VALUE
Site Class	C
Mapped Spectral Response Acceleration at Short Period, S_s (g)	1.357
Mapped Spectral Response Acceleration at 1-second Period, S_1 (g)	0.480
Mapped Peak Ground Acceleration, PGA (g)	0.500
Design Spectral Acceleration at Short Period, S_{DS} (g)	1.085
Design Spectral Acceleration at 1.0-second Period, S_{D1} (g)	0.480
Site Modified Peak Ground Acceleration, PGA_M (g)	0.600

4.3.2 Liquefaction Potential

Liquefaction refers to the condition by which vibration or shaking of the ground (e.g., earthquake forces) disturbs the soil structure and the arrangement of individual soil particles within saturated and unconsolidated soils. This rearrangement of particles results in the development of excess pore pressures and a subsequent loss of strength. In general, soils that are susceptible to liquefaction include very loose to medium dense, "clean" to silty sands located below the water table.

Reviewed liquefaction hazard maps indicate the site and immediate surrounding areas are mapped as low risk of liquefaction. Based on the subsurface conditions observed in our explorations, our interpretation of regional geology and review of hazard maps in the area, we also conclude the potential for liquefaction at the site is low.

4.3.3 Lateral Spreading Potential

Lateral spreading related to seismic activity typically involves lateral displacement of large, surficial blocks of non-liquefied soil overlying a liquefied soil layer that has lost strength during seismic shaking. Lateral spreading usually develops in areas where sloping ground or large grade changes are present. Based on our understanding of liquefaction potential, current site topography and proposed site grading, it is our opinion the risk of lateral spreading at the site is low.

4.3.4 Surface Rupture Potential

Reviewed geologic seismic feature maps indicate the nearest faults to the site are the Dabob Bay fault zone (approximately 5.4 miles southwest) and Southern Whidbey Island fault zone (approximately 9.5 miles northeast). The location of these fault zones has been inferred from geophysical studies and there are no known surface expressions of the faults. Based on our understanding of local geology (bedrock in the area is covered by thick glacial soils) and the proximity of the nearest fault to the project site, it is our opinion the risk for seismic surface rupture at the site is low.

4.4 SHALLOW FOUNDATION SUPPORT

Based on our understanding of the conceptual design, the proposed structures will consist of lightly loaded, single- to two-story, slab-on-grade structures. In our opinion, these proposed structures can be supported on shallow foundations provided bearing surfaces are prepared as recommended below. We anticipate this condition is appropriate for foundations bearing on medium dense or denser existing site soils, including existing fill, outwash and/or glacial till soils.

4.4.1 Bearing Surface Preparation

Forest duff, sod, roots and organic-rich soils must be removed from within structure footprint(s) and beneath footings. Existing pavements, hardscaping or other structural elements (if encountered) must also be removed from within structure footprint(s) and beneath footings.

Foundation bearing surfaces must be confirmed or compacted, as necessary to a uniformly firm and unyielding condition prior to foundation construction. If encountered, we recommend foundations not bear directly on relatively soft/loose and/or organic-rich soils to limit the potential for excessive settlement. If these soil types are present at proposed footing locations, improvements such as additional compaction or removal and replacement (overexcavation) may be necessary. In general, we would expect these materials are more likely to be encountered in the upper fill and outwash layers but could also be encountered in portions of the weathered till. The amount of overexcavation will depend on the building locations, time of year, depth of foundation or below-grade element, depth at which unsuitable soils are encountered and required foundation performance. As site layout is advanced and foundation elevations become available, we can assist with estimating overexcavation amounts for cost estimating purposes.

If structural fill is placed below foundations as either replacement of overexcavated soils or to establish a bearing pad, we recommend the structural fill extend laterally beyond the foundation perimeter a distance equal to the depth of structural fill (measured from the base of the footing), or 2 feet, whichever is less. In our opinion, overexcavation and replacement is not necessary, these issues should be evaluated on a case-by-case basis during construction.

Excavations should be performed using a smooth-edged bucket to limit bearing surface disturbance. Loose or disturbed materials present at the base of footing excavations must be removed, re-compacted or otherwise repaired. If soft, loose or otherwise unsuitable areas are revealed during evaluation that cannot be compacted to a stable and uniformly firm condition the following options may be considered on a case-by-case basis: (1) unsuitable soils be moisture conditioned and recompacted; or (2) unsuitable soils be overexcavated and replaced with compacted structural fill, as needed.

Prepared bearing surfaces and excavations should be evaluated by GeoEngineers during construction (prior to placement of structural fill or formwork and reinforcement) to confirm bearing surfaces have been prepared in accordance with our recommendations, subsurface conditions are as assumed and to provide recommendations to stabilize or repair bearing surfaces.

4.4.2 Wet Weather Considerations

Foundation bearing surfaces must not be exposed to standing water, as wet surfaces can become easily disturbed. If water pools in the base of the excavation, it should be removed before placing structural fill, formwork or reinforcing steel. Structural fill and concrete should be placed as soon as practical after preparation of the foundation excavations. If foundation excavations are exposed to extended wet weather conditions, a 2- to 3-inch-thick layer of lean mix concrete or controlled density fill (CDF) can be considered for subgrade protection. Alternatively, 6 to 8 inches of compacted crushed rock or select granular fill can also be considered for subgrade protection.

4.4.3 Foundation Design Parameters

4.4.3.1 MINIMUM FOOTING DIMENSIONS

Exterior footings should be established at least 18 inches below the lowest adjacent grade. Interior footings should be founded a minimum of 12 inches below the top of the floor slab. Continuous footings should have a minimum width of 18 inches. Isolated column footings should have a minimum width of 24 inches. Actual footing widths must also consider allowable soil bearing pressure for the design loads.

4.4.3.2 ALLOWABLE SOIL BEARING PRESSURE

For footings prepared as described above, we recommend an allowable soil bearing pressure of 3,500 pounds per square foot (psf) be used for design. This recommended bearing pressure applies to the total of dead and long-term live loads and may be increased by one-third when considering total loads, including earthquake or wind loads. These are net bearing pressures. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

We anticipate improved foundation performance (i.e., higher allowable bearing pressures) can be achieved for foundations bearing on intact glacial till deposits, which were encountered at depths between about 2 and 6 feet bgs at test pit locations. If higher pressures are required or would be advantageous to design, we can provide alternative recommendations for bearing surface preparation and allowable soil pressures once site development approach and footing elevations have been determined.

4.4.3.3 FOUNDATION SETTLEMENT ESTIMATES

We estimate settlement of footings, designed and constructed as recommended, will be on the order of 1 inch or less. Differential settlements could be on the order of about ¼ to ½ inch between comparably loaded isolated column footings or over a distance of about 50 feet along continuous wall footing. Settlement is expected to occur rapidly, essentially as loads are applied. Settlements could be greater than estimated if loose or disturbed soil is present beneath footings.

These estimates are based on footings under a design load of about 3,000 pounds per lineal foot or 20,000 pounds per column and proportioned using the recommended allowable bearing pressures above. We should be notified if foundation loads exceed those presented above so we can review and revise our settlement estimates or footing design recommendations, if necessary.

4.4.3.4 LATERAL RESISTANCE

The ability of the soil to resist lateral loads is a function of frictional resistance (which can develop on the base of footings and slabs) and passive resistance (which can develop on the face of below-grade elements as these elements move into the soil). For cast-in-place footings prepared in accordance with the recommendations presented above, the allowable frictional resistance on the base of the footing may be computed using a coefficient of friction of 0.40 applied to the vertical dead-load forces.

The allowable passive resistance on the face of the footing or other embedded foundation elements may be computed using an equivalent fluid density of 300 pounds per cubic foot (pcf) for proof-compacted non-organic site soils or structural fill extending out from the face of the foundation element a horizontal distance at least equal to 2.5 times the depth of the element.

The above values include a factor of safety of about 1.5. The passive earth pressure and friction components may be combined provided the passive earth pressure component does not exceed two-thirds of the total. The passive earth pressure value is based on the assumptions that the adjacent grade is level and groundwater levels remain below the base of the footing throughout the year. The top foot of soil should be neglected when calculating passive lateral earth pressure unless the area adjacent to the foundation is covered with pavement or slab-on-grade.

4.4.4 Footing and Building Drainage

We recommend the ground surface be sloped away from structures. Downspouts should be tightlined away from the building foundation areas and should also be discharged into a stormwater disposal system. Downspouts should not be connected to footing drains.

Based on our interpretation of the regional groundwater table and groundwater conditions observed in our explorations, it is our opinion footing drains are not necessary to maintain bearing support conditions as provided in this report. However, we have observed some instances where near surface water can migrate below footing elements and into slab areas. This can be more prevalent in glacial till areas. As such, we recommend footing drains be included around the base of the footings to maintain dry conditions, for general building perimeter maintenance and to capture incidental seepage from below grade or outside sources (e.g., irrigation). Perimeter drains may also reduce the risk and potential impact of perched groundwater on moisture sensitive flooring or specific floor types.

We expect the majority of standard civil design sections for foundation drains (i.e., perforated pipe surrounded by clean gravel wrapped in a geotextile fabric, installed at base of footing) will be adequate for these purposes, and some flexibility in this design may be considered. The perforated pipe material should consist of solid wall rigid PVC, or equivalent, and be at least 4 inches in diameter. We can review and/or provide specific recommendations for design of foundation drains, if requested once final structure locations and elevations have been determined.

4.5 SLAB-ON-GRADE FLOORS

We anticipate floor loads for proposed structures could be as high as about 250 psf for heavy duty storage and maintenance areas. If design loads vary from those noted here, our recommendations may need to be revised. In our opinion, satisfactory subgrade support for the considered loading can be obtained, provided building slab subgrades are prepared as recommended in this report, including thorough compaction of the upper exposed soil.

4.5.1 Slab-on-Grade Subgrade Preparation

Exposed slab subgrades should be evaluated after site grading is complete. Based on conditions encountered in the test pits and depending on final building elevations, subgrade soils could be variable and range from relatively loose to dense. Disturbed areas beneath slabs-on-grade should be compacted, if possible, or removed and replaced with compacted structural fill. Forest duff, sod, roots, organic-rich soils, existing pavements, hardscaping or other structural elements must be removed from within building footprint(s). Existing fill (if encountered) may be considered for slab subgrade, provided the subgrade is confirmed or compacted as necessary.

In all cases, the exposed soil should be proof compacted to a dense, firm and unyielding condition. For preliminary planning purposes, we do not anticipate overexcavation and replacement below floor slabs will be required over the entire building footprint. Areas of isolated loose or organic-rich areas may be encountered during excavation and site grading efforts, and as such, require some overexcavation and replacement.

4.5.2 Slab-on-Grade Design Parameters

4.5.2.1 MODULUS OF SUBGRADE

For soil conditions expected, and subgrade conditions constructed as recommended, a modulus of subgrade reaction of 250 pounds per cubic inch (pci) can be used directly for designing large building floor slabs and mat foundations. This recommended value typically represents results for soil types expected and is based on a 1-foot by 1-foot square plate loading condition. Ultimately, the coefficient of subgrade reaction for a foundation will vary based on its minimum width according to the following equation:

$$k_s = k_{s1}[(B+1)/2B]^2$$

Where k_s is the coefficient of subgrade reaction, k_{s1} is the coefficient of subgrade reaction for a 1-foot by 1-foot plate (250 pci), and B is the minimum width or lateral dimension of the mat/footing. For the design of smaller mat foundations and typical structural footings and for use with some common structural programs, we recommend the use of k_s. If foundation size is accounted for and for larger slabs and mat foundations, the above value of 250 pci may be used.

4.5.2.2 SETTLEMENT ESTIMATES

Settlement for floor slabs designed and constructed as recommended is estimated to be less than 1-inch for long-term static loads of 200 psf. We estimate differential settlement of floor slabs will be ½ inch or less over a span of 50 feet.

4.5.2.3 CAPILLARY BREAK

We recommend slab-on-grade floors be underlain by a minimum 6-inch-thick capillary break layer to provide uniform support and reduce the potential for moisture migration into the slab. This minimum section thickness is recommended, in part, to assist in bridging areas where the slab spans may span between soils of varying densities. The capillary break material should conform to recommendations provided in Section 5.10 and be placed as recommended in Section 5.11.

4.5.2.4 OTHER DESIGN CONSIDERATIONS

In our opinion, an underslab drainage system is not necessary for slab subgrades within a few feet of ground surface. If dry slabs are required (e.g., where adhesives are used to anchor carpet or tile to slab), a waterproof liner may be placed as a vapor barrier below the slab. The vapor barrier should be selected by the architect and structural engineer and should be accounted for in the design floor section and mix design selection for the concrete, to accommodate the effect of the vapor barrier on concrete slab curing.

4.6 CONVENTIONAL RETAINING WALLS AND BELOW-GRADE STRUCTURES

Retaining walls and below grade structures such as utility vaults are envisioned as part of the proposed development. The provided Site Plan Alternatives concept includes a retaining wall adjacent to the southern edge of the proposed maintenance facility to retain the upper elevated northern portion, however, detailed grading contours and/or wall heights are not available at the time of this report.

Our recommendations, provided below, are based on maximum wall height of about 8 feet (based on a level front slope from the base of the wall) and assume the ground surface within about 25 feet laterally below the toe of retaining structures will be level or near level. Recommended design pressures are based on drained conditions as discussed in the Wall Drainage section below. If these assumptions are not appropriate, we should be contacted to provide revised recommendations, such as reduced allowable soil bearing pressures, undrained lateral soil pressures, a more detailed global stability analysis.

4.6.1 Recommended Lateral Earth Pressures

We anticipate retaining walls and below grade structures will retain existing site soils as encountered in the test pits and/or be backfilled with structural fill. Overcompaction of fill placed directly behind retaining walls or below-grade structures must be avoided to limit lateral pressures placed on the wall. Additional discussion is provided in Section 5.11.3.

For drained conditions we recommend walls and subsurface structures as described above be designed using the following lateral earth pressures.

- Active soil pressure may be estimated using an equivalent fluid density of 35 pcf for the level backfill condition. For walls with backfill sloping upward behind the wall at 2H:1V (horizontal to vertical), an equivalent fluid density of 50 pcf should be used. If the slope is shallower than 2H:1V, the active lateral earth pressures can be linearly interpolated between the two values above using the slope angle in degrees. The active soil pressure condition assumes the top of the wall is not structurally restrained and is free to rotate and deflect a distance of at least 0.001*H (where H is the wall height).
- At-rest soil pressure may be estimated using an equivalent fluid density of 55 pcf for the level backfill condition. For walls with backfill sloping upward behind the wall at 2H:1V, an equivalent fluid density of 80 pcf should be used. The at-rest condition is applicable where walls are restrained against deflection (not allowed to rotate).
- For seismic considerations, a uniform lateral pressure surcharge of 14*H psf (where H is the height of the retaining structure or the depth of a structure below ground surface) should be added to the lateral earth pressure.

The active soil pressure condition assumes the top of the wall is not structurally restrained and is free to rotate and deflect a distance of at least 0.001*H (where H is the wall height). The at-rest condition is applicable where walls are restrained against deflection (not allowed to rotate). The above recommended lateral soil pressures do not include the effects of sloping backfill surfaces or surcharge loads, except as described.

4.6.2 Surcharge Loads

The above recommended lateral soil pressures do not include surcharge loads. We recommend surcharge effects be considered if surcharge loads are applied closer than one-half of the retaining structure height from the wall face. A typical traffic surcharge of 250 psf should be included if vehicles are allowed to operate within one-half the height of the retaining walls. This traffic surcharge can be represented by assuming walls will contain an additional 2 feet of retained fill.

Other surcharge loads, such as construction equipment, soil stockpiles, structures, etc., should be considered on a case-by-case basis. We can provide additional recommendations for specific surcharge loading conditions upon request.

4.6.3 Wall Foundation Support

Wall foundations may be supported on shallow foundations designed and prepared in accordance with recommendations presented in Section 4.4 above. We estimate settlement of retaining structures will be similar to the values previously presented for structure foundations.

4.6.4 Wall Drainage

The lateral earth pressures provided above are based on drained conditions. A positive drainage system behind walls and below-grade structures must be constructed to collect water and prevent the buildup of hydrostatic pressure against the wall or structure.

We recommend a zone of free-draining material behind the retaining structure with perforated pipes to collect seepage water. The drainage material should consist of coarse sand and gravel containing less than 5 percent fines (material passing the U.S. No. 200 sieve) by weight. We recommend drainage material consist of "Gravel Backfill for Walls" described in Section 9-03.12(2) of the WSDOT Standard Specifications. For this condition, the drainage zone should extend horizontally at least 12 inches from the back of the retaining structure. Other systems and materials may also be considered, where appropriate, and as approved by the project engineer.

Site soils encountered in the explorations contain a significant percentage of fines, which are susceptible to particle migration and could potentially clog the drainage. A filter fabric designed for separation should be placed between the wall backfill and native site soils or common borrow fill to prevent soil migration.

A perforated, smooth-walled, rigid polyvinyl chloride (PVC) pipe with a minimum diameter of 4 inches should be placed at the bottom of the drainage zone along the entire length of the retaining structure with the pipe invert at or below the elevation of the base of the footing. The drainpipes should collect water and direct it to a tightline leading to an appropriate disposal system. Cleanouts should be incorporated into the design of the drains in order to provide access for regular maintenance. Roof downspouts, perimeter drains or other types of drainage systems must not be connected to drain systems for retaining walls or below-grade structures. Weep holes and other through-wall drainage systems may also be considered.

4.7 PAVEMENT DESIGN

4.7.1 General Pavement Design Criteria

Proposed improvements include new at-grade parking and driveway areas for passenger automobile and transit bus traffic. We understand asphalt concrete pavement (AC) and Portland cement concrete (PCC) pavements could be used. Based on draft design criteria prepared by WSP (project industrial design engineer) we understand 125 parking spots will be included for passenger automobile traffic, including employee vehicles and shuttle vans. Transit fleet vehicles are expected to vary from shuttle vans to 40-foot double-decker buses. Specific vehicle loading and/or trip frequency information was not provided, we include assumptions and our estimated traffic loading below, incorporating the above general criteria.

- **Light-Duty:** Light duty pavement areas as described in this report are considered those accessed only by passenger automobile traffic (i.e., general automobile parking and driveway areas only). We have estimated up to 500 passenger vehicles per day, including shuttle vans, may access light duty pavement areas.
- **Heavy-Duty:** Heavy-duty pavement areas consist of parking and driveway areas within the drive path of buses and other transit vehicles. In addition to passenger vehicle traffic noted above, we considered heavy-duty pavement areas will also include transit vehicles up to 30,000-pound dry vehicle weight buses. We did not assume additional large truck loading other than the bus loading.

Both heavy and light duty pavement sections are capable of supporting fire engines and ladder type vehicles during infrequent emergency access situations. Heavy construction traffic (e.g., concrete transit mixers, dump trucks or cranes) has not been considered in our pavement design.

Additional traffic load data and assumptions are summarized in Table 2 below.

TABLE 2. CONSIDERED TRAFFIC LOADING DESIGN CRITERIA

SECTION	DESIGN LIFE (YEAR)	PASSENGER CARS/VANS PER DAY	2-AXLE BUSES PER DAY		GROWTH RATE	TOTAL ALLOWABLE ESALS	
			20,000 LB DRY WEIGHT	30,000 LB DRY WEIGHT		AC	PCC
Light Duty (Automobile Traffic Only)	30	500	0	0	2%	10,300	12,900
Heavy Duty (Bus/Transit Vehicles)	30	500	30	10	2%	283,100	382,606

Notes:

ESAL = Equivalent Single Axle Load. “Standard” or “Equivalent” traffic loading number based on a conversion of wheel loads of various magnitudes to a single number in order to represent an equivalent traffic loading.

4.7.2 Pavement Drainage

Long-term performance of pavements is influenced significantly by drainage conditions beneath the pavement section. Positive drainage can be accomplished by crowning the subgrade and establishing grades to promote drainage. The recommended sections assume final improvements surrounding the pavement areas will be designed and constructed such that stormwater or excess irrigation water from landscape, or other areas does not accumulate below the pavement section or pond on pavement surfaces. If pavements in parking areas slope inward (toward the center of the parking area) full depth curbs or other measures should be used to prevent water from entering and ponding on the subgrade and within the base section.

4.7.3 Pavement Subbase

Based on conditions encountered in the test pits, we anticipate pavement subgrades will likely consist of existing fill and/or glacial till deposits. These soils have a relatively high fines content (silt and clay sized particles). Fine-grained soils are susceptible to particle migration, potentially clogging and reducing drainage performance of the base course layer. As such, we recommend a subbase layer consisting of select granular fill as described in Section 5.10 be included between native subgrade soils and overlying CSBC. The subbase layer is intended to provide separation from fine-grained subgrade soil, provide a uniform grading surface, provide uniform pavement support and maintain drainage.

4.7.4 Pavement Subgrade Preparation

Sod, organic-rich soils, existing pavements, hardscaping and/or other structural elements (if encountered) should be removed prior to placement of new pavement sections. Pavement subgrade should be thoroughly compacted and prepared to a uniformly firm, dense and unyielding condition on completion of stripping and regrading, prior to placing structural fill or pavement base fill. Compaction recommendations are presented in Section 5.11 of this report.

We recommend prepared subgrades be evaluated by a member of our firm to identify areas of yielding, soft/loose or otherwise unsuitable subgrade soils. Probing with a steel probe rod and proof-rolling with a heavy piece of wheeled construction equipment are appropriate methods of evaluation.

Similar to previously discussed general subgrade preparation, if soft or otherwise unsuitable subgrade areas are revealed during evaluation that cannot be compacted to a stable and uniformly firm condition, we recommend: (1) the unsuitable soils be scarified (e.g., with a ripper or farmer's disc), aerated and recompacted, if practical; or (2) the unsuitable soils be removed and replaced with compacted structural fill, as needed. If subgrade soils are excessively loose or soft a geotextile reinforcement layer can also be considered.

4.7.5 Additional Considerations

Structural fill and crushed surfacing materials (CSBC/CSTC) should consist of material as recommended in Section 5.10 and be moisture-conditioned to near optimum moisture content (OMC) and compacted as recommended in Section 5.11. Hot-mix asphalt (HMA) should conform to applicable sections of 5-04, 9-02 and 9-03 of the Washington State Department of Transportation (WSDOT) Standard Specifications. We recommend HMA consist of Class ½-inch (maximum aggregate size), with binder performance grade (PG) of either 58-22 or 64-22 (depending on anticipated climate temperatures the asphalt will be exposed).

Our design PCC sections below assume concrete pavements in vehicular areas will be jointed and that dowel bars will be included at expansion joints to assist in load transfer. Dowels should not be included between exterior pavement slabs and interior pavement slabs to reduce the risk of cracking due to differential settlements. We also assume concrete shoulders or curbs will be installed.

Some areas of pavement may exhibit settlement and subsequent cracking over time, this can be a common part of the pavement lifecycle. Cracks in the pavement will allow water to infiltrate to the underlying base course, which could increase the amount of pavement damage caused by traffic loads. To prolong the effective life of the pavement, cracks should be sealed as soon as possible.

4.7.6 Temporary Construction Surfacing ATB Option

A temporary surfacing of asphalt treated base (ATB) could be used to protect partially constructed pavement sections and subgrades during construction. This can provide a relatively clean working surface, prevent construction traffic from damaging final paving surfaces and reduce subgrade repairs for final paving. A 2-inch-thick section of ATB can typically be substituted for the upper 3 inches of CSBC in the recommended pavement sections, provided ATB is underlain by pavement subbase as described and a minimum 2-inch-thick layer of CSBC is in place below the ATB. Prior to placement of the final pavement surface sections, we recommend ATB be re-evaluated and areas of ATB failure be removed, and the subgrade repaired (if necessary). If ATB is serviceable when final pavements are constructed, the design pavement thickness can be placed directly over the ATB.

4.7.7 Recommended Pavement Sections

Based on the assumptions and estimated traffic loading noted above, our recommended pavement sections are presented in Table 3.

TABLE 3. RECOMMENDED PAVEMENT SECTIONS

SECTION	MINIMUM PCC THICKNESS (INCHES)	MINIMUM AC THICKNESS (INCHES)	MINIMUM AGGREGATE BASE THICKNESS (INCHES)	MINIMUM SUBBASE THICKNESS (INCHES)
Sidewalk (Pedestrian Areas Only, No Vehicle Loading)	-	2	4	-
	4	-	4	-
Light Duty (Automobile Traffic Only)	-	2.5	4	6
	6	-	4	6
Heavy Duty (Access Lanes for Bus/Transit Vehicle Maintenance)	-	4	6	6
	9	-	4	6

4.8 STORMWATER INFILTRATION

On-site infiltration of stormwater is being considered as part of the proposed development. Stormwater facilities could include shallow dispersed systems (e.g., bioswales, rain gardens, etc.) and/or larger detention/infiltration pond(s), depending, in part, on infiltration testing completed as part of this study and discussed below. Infiltration design discussion and recommendations provided below are based on requirements provided in the 2021 Kitsap County Stormwater Design Manual (SDM) for design of infiltration facilities. Based on our experience and requirements presented in the SDM, infiltration rates for glacial till soils present at the site must be determined using a Pilot Infiltration Test (PIT).

4.8.1 Infiltration Testing Results

Small-scale PITs were completed in test pits TP-2 and TP-5 at approximate depths of 4 feet (Elevation 178 feet) and 2.5 feet (Elevation 174.5 feet), respectively. Details regarding PIT methodology and analysis are provided in Appendix A. Recorded water level data from the PITs and plots of the estimated infiltration rates calculated during each cycle of the PITs are presented in Figures 4 and 5. Table 4 below summarizes the calculated rates considering a water depth/head of 12-inches. The table also includes previously measured infiltration rates at monitoring points within nearby stormwater ponds (Stormwater Pond Monitoring Results Memo, GeoEngineers 2017).

TABLE 4. ESTIMATED SOIL INFILTRATION RATE SUMMARY

Location	Depth (feet)	Elevation (feet)	Soil Unit	USCS Soil Type	Calculated Infiltration Rate (in/hr)	
					Measured (Short-Term)	Factored (Long-Term)
TP-2	4	178	Intact Till	SM	0.3	0.1
TP-5	2.5	174.5	Weathered Till	SM	9.3	3.4
Pond 1	-	-	-	-	n/a	-
Pond 2	-	-	-	-	0.04	-
Pond 3	-	-	-	-	0.12 to 0.14	-
Pond 4	-	-	-	-	0.07 to 0.08	-
Pond 5	-	-	-	-	0.12 to 0.25	-

RECOMMENDED DESIGN INFILTRATION RATE

Similar soil types, gradations and layer depths were observed between test pits completed at the site. In our opinion, soil conditions (i.e., glacial till soils) observed at the PIT depths are representative of conditions throughout most of the site. Somewhat increased rates could be possible within upper fill, outwash and/or weathered zones, however ponding and shallow depth to intact glacial till will reduce the overall feasibility.

Based on the PIT results and previous measurements of actual performance of nearby ponds, we recommend a design infiltration rate of 0.1 inches per hour be considered for ponded water depths of about 12 inches. In our opinion this can be considered an average value across the site based on the uniformity of conditions encountered in the explorations. We should be contacted once facilities have been located and sized to provide additional review and confirm the recommended rate is still appropriate as the design progresses. We do not anticipate additional testing will be required to determine infiltration rates for final design.

4.8.2 Design Discussion

The SDM requires a minimum vertical separation of 1-foot from the base of infiltration facility to a hydraulically restrictive layer. Examples provided in the SDM for hydraulically restrictive layers are glacially consolidated soils with greater than 50 percent fines, glacially unconsolidated soils with greater than 70 percent fines and bedrock. Completed PITs and the recommended infiltration rate (0.1 in/hr) provided above are appropriate for glacial till soils. These till soils were observed to be relatively consistent throughout the site. In our opinion, glacial till soils will limit the maximum infiltration rate attainable for this site; we do not recommend relying on rates determined from the fill and/or outwash deposits for stormwater infiltration. Since our recommended rates consider infiltration in glacial till deposits, it is our opinion vertical separation requirements from a hydraulically restrictive layer have been met.

Vertical separation must also be provided from the base of facility to groundwater. As previously discussed, static groundwater levels in nearby wells are on the order of 145 feet deep. Based on excavation depths expected for this project, it is our opinion that groundwater levels are not a limiting factor in stormwater design anticipated (typically less than 8 feet deep below current grade) for this project.

Stormwater treatment could be required to reduce the potential for increased run-off and pollutants. Cation exchange capacity (CEC) and organic content (OC) testing results of selected soil samples from the test pits are summarized in the table below for stormwater treatment design, if applicable and necessary.

TABLE 5. CHEMICAL SOIL ANALYSIS RESULTS

TEST PIT	DEPTH (FEET)	ELEVATION (FEET)	SOIL UNIT	PERCENT FINES	USCS SOIL TYPE	CEC (MEQ/100G)	OC (%)
TP-2	4	178	Intact Till	30	SM	2.1	0.8
TP-5	2.5	174.5	Weathered Till	19	SM	0.9	1.7

meq = milliequivalents; g = grams

Other considerations such as required horizontal setbacks, treatment, effects on nearby properties and similar site review, as presented in the SDM, should also be incorporated as part of the overall stormwater system design. As final locations and sizes of stormwater systems are developed, additional review and/or verification could be requested by the County or another agency.

4.8.3 Construction Considerations

We recommend we be retained during construction to observe soil conditions at the base of the infiltration facilities and verify exposed soil conditions are as anticipated for the proposed design. Equipment should not be permitted in the infiltration areas after they are excavated because of the potential for compaction of the subgrade that could reduce the infiltration rate. To help reduce clogging of infiltration facilities, we recommend they be protected during construction with siltation control facilities such as temporary settling basins, silt fences and hay bales. Suspended solids can clog the soil and reduce the infiltration rate. Periodic sweeping of paved areas, during and following construction, will help extend the life of the infiltration facilities.

5.0 Site Development and Earthwork Recommendations

5.1 EXCAVATION AND DEMOLITION METHODS

We anticipate site development and earthwork activities on site will include: (1) clearing and stripping vegetated areas; (2) demolition of existing structures and pavements; (3) site grading; (4) placing and compacting fill and backfill materials; (5) establishing subgrades for foundations, pavements and hardscaping; and (6) excavating and installing utilities, including stormwater facilities.

Based on the soils encountered in our subsurface explorations, it is our opinion that conventional earthmoving equipment in proper working condition should be capable of completing expected site grading and excavations. The earthwork contractor should be responsible for reviewing this report (including the test pit logs), providing their own assessments and providing equipment and methods needed for earthwork completion, while protecting existing subgrade and bearing surfaces.

5.2 CLEARING AND STRIPPING

Existing surfaces within the proposed building, pavement and hardscaping areas should be cleared and stripped of vegetation and organics prior to site development. Based on our explorations, minimum stripping depths at the site will likely be on the order of 2 to 6 inches to remove forest duff, sod and surface vegetation. Greater stripping depths will be required to remove localized zones of loose or organic-rich soil. Forested areas of the site could also require greater stripping depths to remove stumps and root systems. During clearing and stripping, stumps and primary root systems of shrubs and trees should be completely removed. Voids caused by removal of stumps and/or root systems should be backfilled with compacted structural fill. Stripped material should be transported off site or processed and used as fill in landscaping areas (see additional discussion in Section 5.10 of this report for the use of on-site soils).

Existing structural improvements (e.g., pavements, hardscaping and foundations) within proposed development areas should be demolished and removed. Existing utilities should be removed or abandoned and left in place from within new building footprints and rerouted, if needed. In order to reduce the risk of future settlement, existing utility lines larger than 4 inches in diameter that are located beneath proposed buildings should be completely removed or filled with grout if abandoned and left in place.

Voids and depressions encountered or created during site preparation, clearing, stripping and/or resulting from demolition should be cleaned of loose soil or debris down to firm soil and backfilled with compacted structural fill as recommended in this report. Greater disturbance depths should be expected if site preparation and earthwork are conducted during periods of wet weather.

5.3 DEBRIS AND OBSTACLES

Although not encountered in the test pits, the earthwork contractor should be prepared to encounter obstacles and debris in areas of the site to be regraded and/or excavated. If encountered, obstacles, debris and other deleterious material (e.g., concrete, asphalt or brick rubble) should be removed from within the footprint of the new improvements.

We observed occasional cobbles and boulders in the test pit excavations, as noted in the logs, which is common in glacial till deposits. The contractor should be prepared for the presence of cobbles or boulders in areas to be excavated or regraded. For uniform and adequate compaction, the contractor may be required to separate larger cobbles and boulders from soils generated during excavation, if excavated soils are to be reused as structural fill. Boulders should be removed from the site or used in landscape areas. Voids caused by boulder removal should be backfilled with structural fill.

5.4 SUBGRADE PREPARATION AND EVALUATION

Subgrades should be thoroughly compacted to a uniformly firm and unyielding condition upon completion of stripping and demolition, prior to placing fills, structures or pavements. Compaction recommendations in relation to the maximum dry density (MDD, modified Proctor) are presented in Section 5.11 of this report. We recommend prepared subgrades be evaluated by a member of our firm to identify areas of yielding or soft soil. Probing with a steel probe rod or proof-rolling with a heavy piece of wheeled construction equipment are appropriate methods of evaluation.

If soft or otherwise unsuitable subgrade areas are revealed during evaluation that cannot be compacted to a stable and uniformly firm condition, we recommend: (1) the unsuitable soils be scarified (e.g., with a ripper or farmer's disc), aerated and recompacted, if practical; or (2) the unsuitable soils be removed and replaced with compacted structural fill, as needed.

Subgrade disturbance or deterioration could occur if the subgrade is wet and cannot be dried. If the subgrade deteriorates during compaction, it may become necessary to modify the compaction criteria or methods. It may also be necessary to place crushed rock to protect subgrade areas during construction.

5.5 SUBGRADE PROTECTION AND WET WEATHER CONSIDERATIONS

Site soils encountered in our explorations generally contain a significant percentage of fine-grained particles (i.e., silts and clays). Laboratory testing of selected soil samples indicates fines contents (material passing the U.S. No. 200 sieve) between about 8 and 41 percent. Summary logs, soil descriptions and results of our laboratory testing are included in Appendix A. These materials will be sensitive to small changes in moisture and will be susceptible to disturbance from construction traffic (including vehicle and foot traffic) when wet or if earthwork is performed during wet weather. When the moisture content of these soils is more than a few percent above the OMC, the soil can become muddy and unstable, and it will be challenging or impossible to meet the required compaction criteria. During dry weather, these materials will: (1) be less susceptible to disturbance; (2) provide better support for construction equipment; and (3) be more likely to meet the required compaction criteria.

The wet weather season generally begins in October and continues through May in western Washington; however, periods of wet weather may occur during any month of the year. If earthwork activities (e.g., grading, excavations, etc.) occur during wet weather conditions, additional efforts and methods to secure the site and reduce ground disturbance will be necessary. The contractor should be responsible for wet weather construction practices and protecting the subgrade during construction. For earthwork activities during wet weather, we recommend that the following steps be taken:

- The ground surface in and around the work area should be sloped so that surface water is directed away from the work area. The ground surface should be graded so that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- Earthwork activities should not take place during periods of heavy precipitation.

- The contractor should cover soil stockpiles that will be used as structural fill with plastic sheeting. It may be prudent to segregate the siltier surface materials from underlying cleaner gravels so as to not add fine grained particles, making them potentially unusable.
- Site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with the existing gravel or structural fill materials not susceptible to wet weather disturbance.
- Construction activities should be scheduled so that the length of time that soil is left exposed to moisture is reduced to the extent practical.
- During periods of wet weather, concrete should be placed as soon as practical after preparing excavations and bearing surfaces.
- Foundation bearing surface protection should also be considered as discussed in Section 4.4.
- Protective surfacing such as placing ATB, cement-treated base (CTB), cement-treated subgrades or haul roads made of quarry spalls or a layer of free-draining material such as well-graded pit-run sand and gravel can be used to protect areas from construction traffic. Typically, minimum gravel thicknesses on the order of 12 to 18 inches are necessary to provide adequate subgrade protection for repeated construction traffic. ATB used for temporary construction surfacing is also discussed in Section 4.7.6 of this report.

5.6 TEMPORARY EROSION AND SEDIMENTATION CONTROL

Construction activities, including stripping and grading, will expose soils to the erosional effects of wind and water. The amount and potential impacts of erosion are partly related to the time of year of construction, soil type, slope length and gradient, construction sequencing and weather. Wet weather construction will increase the amount and extent of erosion and potential sedimentation.

Implementing an erosion and sedimentation control plan will reduce the project impact on erosion-prone areas. Measures could include a combination of interceptor swales, straw bale barriers, silt fences and straw mulch for temporary erosion protection of exposed soils. Disturbed areas should be finish graded and seeded as soon as practicable to reduce the risk of erosion. Erosion and sedimentation control measures should be installed and maintained in accordance with the requirements of the city, county or other appropriate jurisdiction.

5.7 GROUNDWATER HANDLING CONSIDERATIONS

5.7.1 Surface Water

Surface water inflow to construction areas and open excavations can be problematic. Proactive handling of surface water can reduce groundwater handling needs. Provisions for surface water control and temporary dewatering during earthwork activities and excavations should be included in the project plans and should be installed prior to commencing earthwork. Surface water should be collected, controlled and directed away from construction areas. Appropriate measures such as sloping ground, berms/curbs, diversion trenches and sumps and pumps should be used, as necessary. Protective measures should be implemented for erosion sensitive areas, such as exposed slopes, excavations behind retaining structures, and open excavations.

5.7.1.1 TEMPORARY GROUNDWATER HANDLING

Static groundwater was not encountered in the completed test pits (as deep as about 9.5 feet bgs and Elevation 170.5 feet). Static groundwater levels were reported at depths of about 145 feet bgs in the nearest reviewed water wells (within about 0.7 miles radius of the site). Based on this information, we do not anticipate excavations for proposed development will extend below static groundwater levels.

Although not encountered in the completed test pits, we anticipate perched groundwater could be present at various times throughout the year. We recommend perched groundwater seepage be considered during design and into construction areas and open excavations. The presence, quantity and location of perched groundwater (if encountered) is expected to be dependent on infiltration of surface water that slows or terminates atop the underlying glacial till. Slow to moderate groundwater seepage should be expected if perched groundwater conditions are encountered. We recommend the contractor performing the work be made responsible for developing a plan to control and collect groundwater encountered.

We anticipate perched groundwater will be discontinuous, intermittent and will vary depending on a variety of conditions, including time of year during which construction is conducted, rainfall, irrigation activities and excavation depths. Groundwater handling needs will typically be lower during the summer and early fall months. Site grading can also affect the quantity and location of perched groundwater. The level of effort required could vary throughout the site and duration of the project. For the anticipated construction, we estimate slow to moderate groundwater seepage (on the order of 1 to 3 gallons per minute) could occur during the wet season or if earthwork occurs during wet weather. We anticipate controlling groundwater with sumps, pumps and/or diversion ditches will be adequate for excavations expected at this site; relatively shallow excavations that are only open for a short amount of time.

5.8 TEMPORARY EXCAVATIONS, CUT SLOPES AND SHORING

Soils encountered in the explorations (as deep as about 9.5 ft bgs) typically consist of glacial till soils; densities generally increased with depth. Observations and notes of caving in the test pits are included in the summary logs. Based on our observations and experience, excavations at the site could experience minor caving, especially at shallower depths within zones of relatively less dense soils (i.e., fill, outwash and weathered deposits). Intact glacial till deposits can maintain near vertical cut slopes for a time; however, near vertical slopes in these soils can also slough quickly and without warning.

All excavations and temporary shoring developed at this site should be designed and/or constructed in accordance with applicable Occupational Safety and Health Administration (OSHA), Washington Industrial Safety and Health Act (WISHA) and other appropriate regulations. Excavations deeper than 4 feet should be shored or laid back to provide safe working conditions and prevent soil from falling into excavations. In our opinion, site soils would be considered an OSHA Soil Type C, provided there is no seepage and excavations occur during periods of dry weather. In general, to maintain site grading and provide safe working conditions, we recommend temporary cut slopes and excavations greater than 4 feet in height be inclined no steeper than about 1.5H:1V (horizontal to vertical). This guideline assumes surface loads are kept at a minimum distance of at least one-half the depth of the cut away from the top of the slope and seepage is not present on the slope face. Steeper cut slopes may be feasible depending on locations and depths and if necessary, these slopes should be reviewed prior to planning for excavation. Flatter cut slopes could be necessary with changes in boundary conditions, such as seepage, surface surcharge loading, vibrations, nearby structures and other factors.

If excavations on site are to exceed the temporary slope recommendations and/or surface surcharge loading, seepage or other factors are anticipated, temporary shoring might be required. Shoring for trenches less than about 8 feet deep and above static groundwater are expected to be possible with typical trench box or similar systems. Moderate sloughing should be expected outside the box. Shoring deeper than 8 feet should be reviewed and/or designed by a registered engineer before installation.

5.9 PERMANENT CUT AND FILL SLOPES

We recommend permanent slopes be constructed at a maximum inclination of 2H:1V. Where 2H:1V permanent slopes are not feasible, protective facings and/or retaining structures should be considered. Where access for landscape maintenance is desired, we recommend a maximum inclination of 3H:1V. These guidelines assume surface loads are kept at a minimum distance of at least one-half the height of the slope away from the top of the slope and seepage is not present on the slope face. Flatter cut slopes or additional drainage measures could be necessary where seepage occurs or if surface surcharge loads are anticipated.

Exposed areas should be re-vegetated as soon as practical to reduce surface erosion and sloughing. Temporary protection should be used until permanent protection is established. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

5.10 FILL MATERIALS

5.10.1 Fill Requirements

Our recommendations for fill materials are presented below. We recommend GeoEngineers review contractor submittals for earthwork materials to be used. Material used for fill must be free of clays, debris, organic contaminants and rock fragments larger than 3 inches. The workability of material for use as fill will depend on the gradation and moisture content of the soil. As the percentage of fines increases, fill materials become increasingly sensitive to changes in moisture. Typically, soil containing more than about 5 percent fines is sensitive to changes in moisture and will become difficult to compact, even when just a few percent above the optimum moisture content (OMC).

Other fill materials such as crushed rock, permeable ballast, and/or quarry spalls, may also be used, as needed, and likely required during wet weather. Budgets should include provisions for import granular fill especially if construction is planned during the wet weather season. We can provide specific recommendations for imported material specific for its intended use and based on the time of year of construction, once site development plans and schedule are finalized. Fill placed to support structural bearing elements, surfaces, pavements and/or hardscapes, should be specified as structural fill.

5.10.2 Reuse of On-Site Soils as Structural Fill

In general, on-site soils can be considered for use as structural fill during extended periods of dry weather, provided material is adequately moisture-conditioned and compacted as recommended in this report. At this time, we do not recommend that on-site materials be considered for use during periods of wet weather. On-site materials will essentially be rendered useless as a structural fill if they become wet. If it is desired to try and use on site materials, we recommend that the earthwork contractor provide a detailed plan of its use including excavation, storage, protection and placement, and that it be reviewed by the project design team and engineers, prior to acceptance for use, and with provisions where applicable.

Soils encountered in the explorations consist generally of glacial till deposits (silty sands with variable gravel and cobble content). Summary logs, soil descriptions and results of our laboratory testing are included in Appendix A. Laboratory testing results of selected soil samples indicate moisture contents up to about 11 percent and fines content up to about 44 percent. In our experience, typical OMCs are on the order of 9 to 15 percent for these soil types. As such, some of the materials could be generated above optimum moisture content and require drying back to obtain proper compaction.

5.10.3 Topsoil Strippings

Topsoil strippings may be placed on site provided they are placed in non-structural areas that can tolerate some long-term total and differential settlements. Settlements of organic-rich soils are highly variable and difficult to quantify. Settlement could continue for several years after construction is completed as the organics break down and decompose. Alternatively, topsoil strippings can be hauled off site.

5.10.4 Imported Structural Fill

For imported material, we recommend crushed rock or select granular fill (described below) be used for structural fill. If prolonged dry weather prevails during the earthwork phase of construction, materials with a somewhat higher fines content (up to about 12 percent passing the U.S. No. 200 sieve) may be acceptable, such as “Select Borrow” or “Gravel Borrow” as described in Section 9-03.14 of the WSDOT Standard Specifications.

5.10.4.1 SELECT GRANULAR FILL

Imported select granular fill should consist of pit run, crushed rock or crushed gravel and sand that is well-graded between coarse and fine sizes. Approximately 25 to 65 percent of material should pass the U.S. No. 4 sieve and less than 5 percent pass the U.S. No. 200 sieve, based on the minus ¾-inch fraction. Although not always necessary, we typically recommend a minimum of two mechanically fractured faces. “Gravel Borrow” as described in Section 9-03.14(1) of the WSDOT Standard Specifications may be considered for use as select granular fill, provided that the fines content (material passing the US No. 200 sieve) does not exceed 5 percent.

5.10.4.2 CRUSHED ROCK

Crushed rock should consist of imported clean, durable, crushed angular rock. We recommend crushed rock used as a structural fill consisting of material of approximately the same quality as CSBC. Alternatively, permeable ballast (WSDOT Section 9-03.9[2]) or quarry spalls (WSDOT Section 9-13.1[1] and 9-13.1[5]) can be considered, depending on use and review and acceptance by the project engineer.

5.10.4.3 CAPILLARY BREAK

Capillary break material should consist of a well-graded sand and gravel or crushed rock with a maximum particle size of ¾ inch and less than 5 percent fines by weight. In our opinion, material with gradation characteristics similar to American Association State Highway and Transportation Officials (AASHTO) “AASHTO Grading No. 7” (¾-inch-minus) as described in WSDOT Standard Specification 9-03.1.4(C) is suitable for capillary break. Alternatively, CSBC (as described above) may also be considered.

5.10.4.4 GRAVEL BACKFILL FOR WALLS

We recommend fill material and placement in wall drainage zones be consistent with recommendations provided in Section 4.6.4 of this report. In our opinion, we recommend “Gravel Backfill for Walls” as described in Section 9-03.12 of the WSDOT Standard Specifications be considered based on the design of the drainage zone.

5.10.4.5 GRAVEL BACKFILL FOR DRAINS

We recommend material used for footing drains be of approximately the same quality as “Gravel Backfill for Walls” or “Gravel Backfill for Drains” as described in Section 9-03.12 of the WSDOT Standard Specifications. Alternative materials, such as washed rock, may also be considered provided the fines content is limited to less than 5 percent passing the No. 200 sieve.

5.10.4.6 PIPE BEDDING

Trench backfill for the bedding and pipe zone should consist of well-graded granular material similar to “Gravel Backfill for Pipe Zone Bedding” described in Section 9-03.12(3) of the WSDOT Standard Specifications. Other materials may be required depending on pipe manufacturer specifications and/or jurisdictional requirements where utilities extend off the property and into the public right-of-way.

5.11 FILL PLACEMENT AND COMPACTION

5.11.1 Lifts

Fill should be placed in uniform horizontal lifts above subgrades prepared as previously recommended. Each lift must be compacted to a firm, non-yielding condition by mechanical means prior to additional fill placement. Loose lift thickness should not exceed 12 inches when using heavy compaction equipment (e.g., vibratory hoe-packs and steel drum rollers). Loose lift thickness should not exceed 6 inches when using hand operated compaction equipment. The actual thickness will be dependent on the fill material and the type and size of compaction equipment used. It is the contractor’s responsibility to select appropriate compaction equipment and place the material in lifts that are thin enough to meet the required compaction criteria.

5.11.2 Compaction Criteria

Each lift should be moisture-conditioned to within 3 percent of the OMC and then compacted to the specified density before placing subsequent lifts. OMC content and maximum dry density (MDD) should be determined in accordance with ASTM Standard Practices Test Method D 1557 (Modified Proctor). The OMC varies with gradation and should be evaluated during construction. Fill material that is not near the OMC should be moisture-conditioned prior to compaction.

During fill and backfill placement, material should be regularly evaluated and tested to verify adequate compaction is being achieved. Compaction should typically be evaluated by means of in-place density testing unless other methods are proposed for oversized materials and are approved by GeoEngineers during construction. These other methods typically involve procedural placement and compaction specifications together with verifying requirements such as proof-rolling. Recommended fill compaction criteria is provided in Table 6.

TABLE 6. RECOMMENDED FILL COMPACTION CRITERIA

Fill Material	LOCATION	RECOMMENDED COMPACTION, % MDD AT ± 3% OF OMC
Structural Fill (including trench backfill)	Foundations & Floor Slabs	95
	Pavements & Hardscapes	95
Retaining Wall Backfill	Within Five Feet Laterally from Walls & Subsurface Structures	92
Nonstructural Zones	Landscaping, etc.	85-90
Pipe Zone Bedding	Utility Trenches	90

5.11.3 Backfill Behind Retaining Walls

Overcompaction of fill placed directly behind retaining walls should be avoided. We recommend use of hand-operated compaction equipment and maximum 6-inch loose lift thickness when compacting fill within about 5 feet behind retaining walls.

5.11.4 Trench Backfill

For utility excavations, we recommend the initial lift of fill over the pipe be thick enough to reduce the potential for damage during compaction but generally should not be greater than about 18 inches. In addition, rock fragments greater than about 1 inch in maximum dimension should be excluded from this initial lift. The pipe manufacturer(s) and jurisdictional requirements should also be reviewed for additional considerations.

5.11.5 Existing Slopes

Fill placement on existing slopes steeper than 5H:1V should be benched into the slope face. The configuration of benches depends on the equipment being used and the inclination of the existing slope. Bench excavations should be level and extend into the existing slope face at least half the width of the compaction equipment used. To achieve uniform compaction, we recommend fill slopes be overbuilt by at least 12 inches and subsequently cut back to expose well-compacted fill.

5.11.6 Fill in Non-Structural Areas and Stripping Material

Areas that will not support buildings or pavements and can tolerate moderate total and differential settlements will not require structural fill. To limit long-term settlements that could affect site drainage, we recommend fill placed in these non-structural areas be compacted to 85 to 90 percent of the MDD and generally contain no more than 10 percent organic matter by weight.

If topsoil strippings or other materials with increased moisture and/or organics are placed in on-site excavations, the sidewalls of the excavation should be sloped at 3H:1V or flatter. This will help provide a transition between the thick layer of compressible soil in the center of the excavation and the on-site soil surrounding the excavation. Similarly, the top of the fill should be crowned to allow for future settlement. Soils should be placed in moderate lifts and tracked or back-bladed into place with a bulldozer or similar equipment. A sheepsfoot type roller will likely be more effective for breaking up and compacting sod than a smooth drum roller.

6.0 Limitations

We have prepared this report for the exclusive use of TCF Architecture, Kitsap Transit, and their authorized agents and/or regulatory agencies for the proposed North Base Maintenance Facility Design project (KT No. 23-820) in Poulsbo, Washington. This report is not intended for use by others, and the information contained herein is not applicable to other sites. No other party may rely on the product of our services unless we agree in advance and in writing to such reliance.

The services and recommendations provided in this report are based on the assumptions and design information provided to us as described in this report; recommendations may not be appropriate to design if plans are modified. We recommend GeoEngineers be retained to review the geotechnical engineering-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in this report.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. The conclusions, recommendations and opinions presented in this report are based on our professional knowledge, judgment and experience. No warranty or other conditions, express or implied, should be understood.

Any electronic form, facsimile or hard copy of the original document (email, text, table and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

Please refer to Appendix C "Report Limitations and Guidelines for Use" for additional information pertaining to use of this report.

7.0 References

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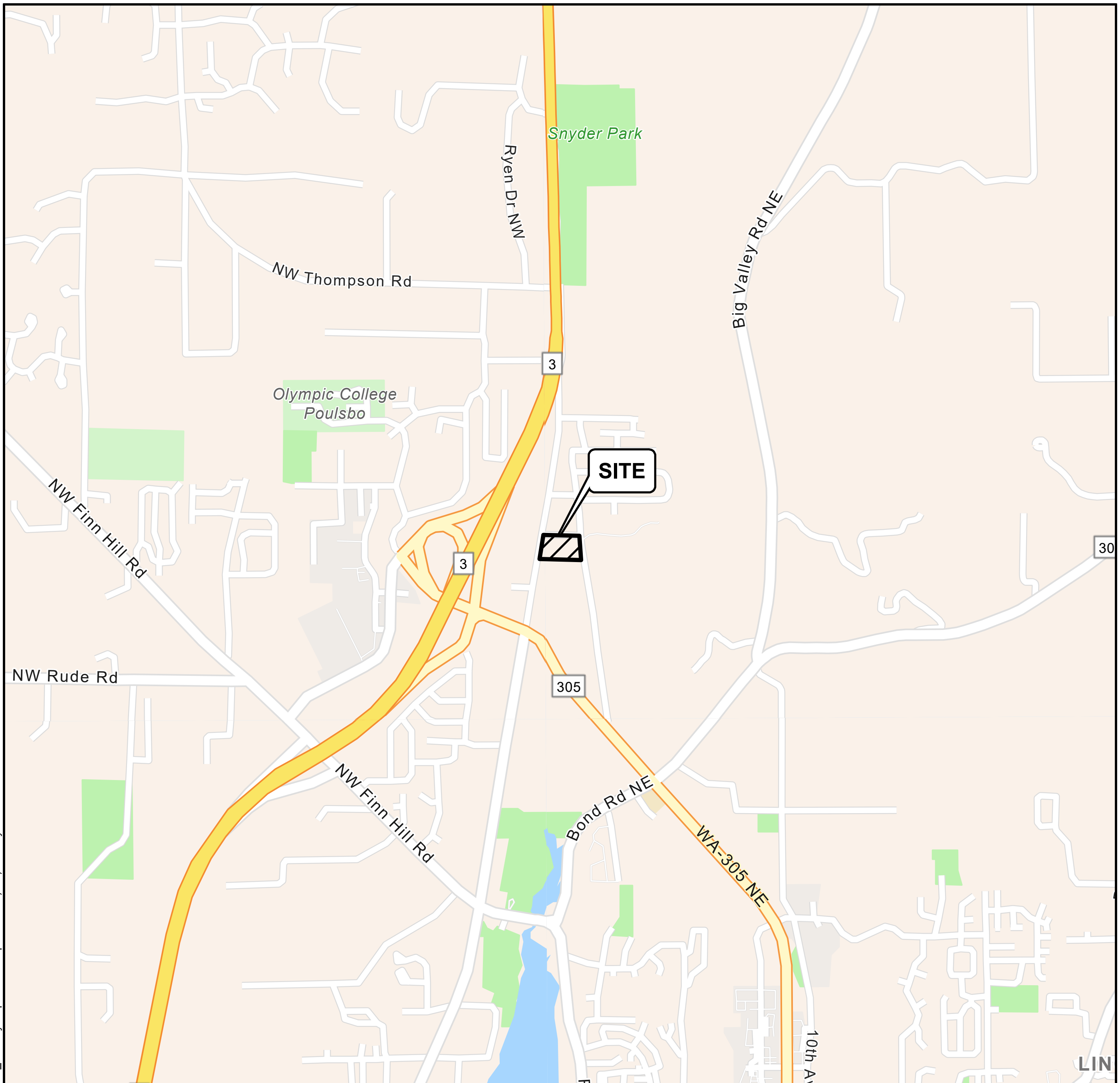
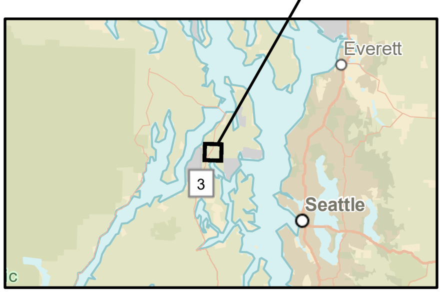
Figures





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Feet

Source(s):
- ESRI

Coordinate System: NAD 1983 UTM Zone 10N

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Vicinity Map

North Base Maintenance Facility
Poulsbo, Washington

GeoEngineers

Figure 1

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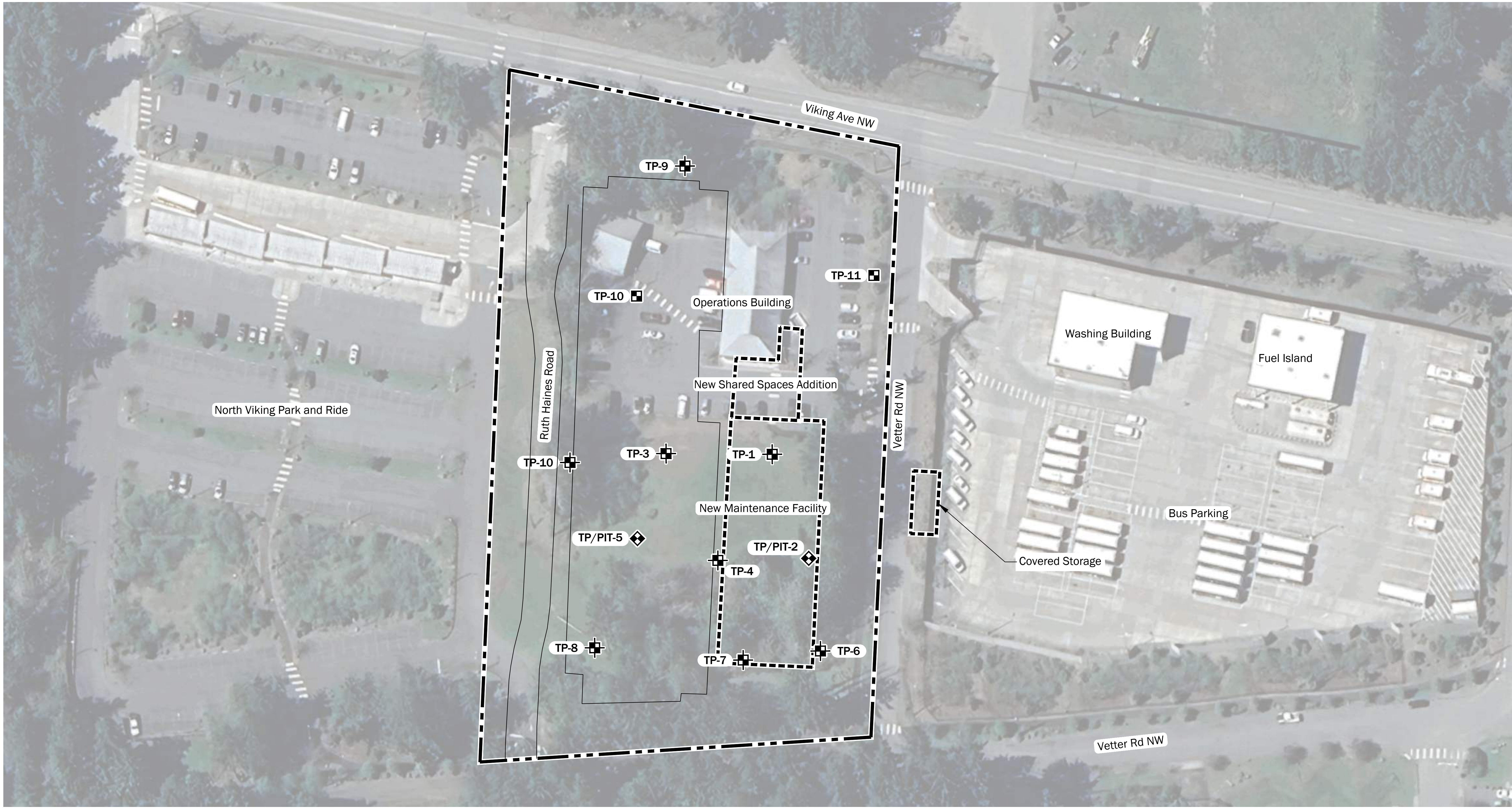

Source(s):

- Aerial from Google Earth Pro, dated 4/27/2023

Projection: WA State Plane, North Zone, NAD83, US Foot

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Legend

- Site Boundary
- Proposed New Structure
- TP-1 — Test Pit by GeoEngineers, Inc., 2023
- TP/PIT-5 — Test Pit with Pilot Infiltration Test by GeoEngineers, Inc., 2023
- TP-10 — Test Pit by GeoEngineers, Inc., 2012

N

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Feet

Site Plan

North Base Maintenance Facility
Poulsbo, Washington

GeoEngineers

Figure 2

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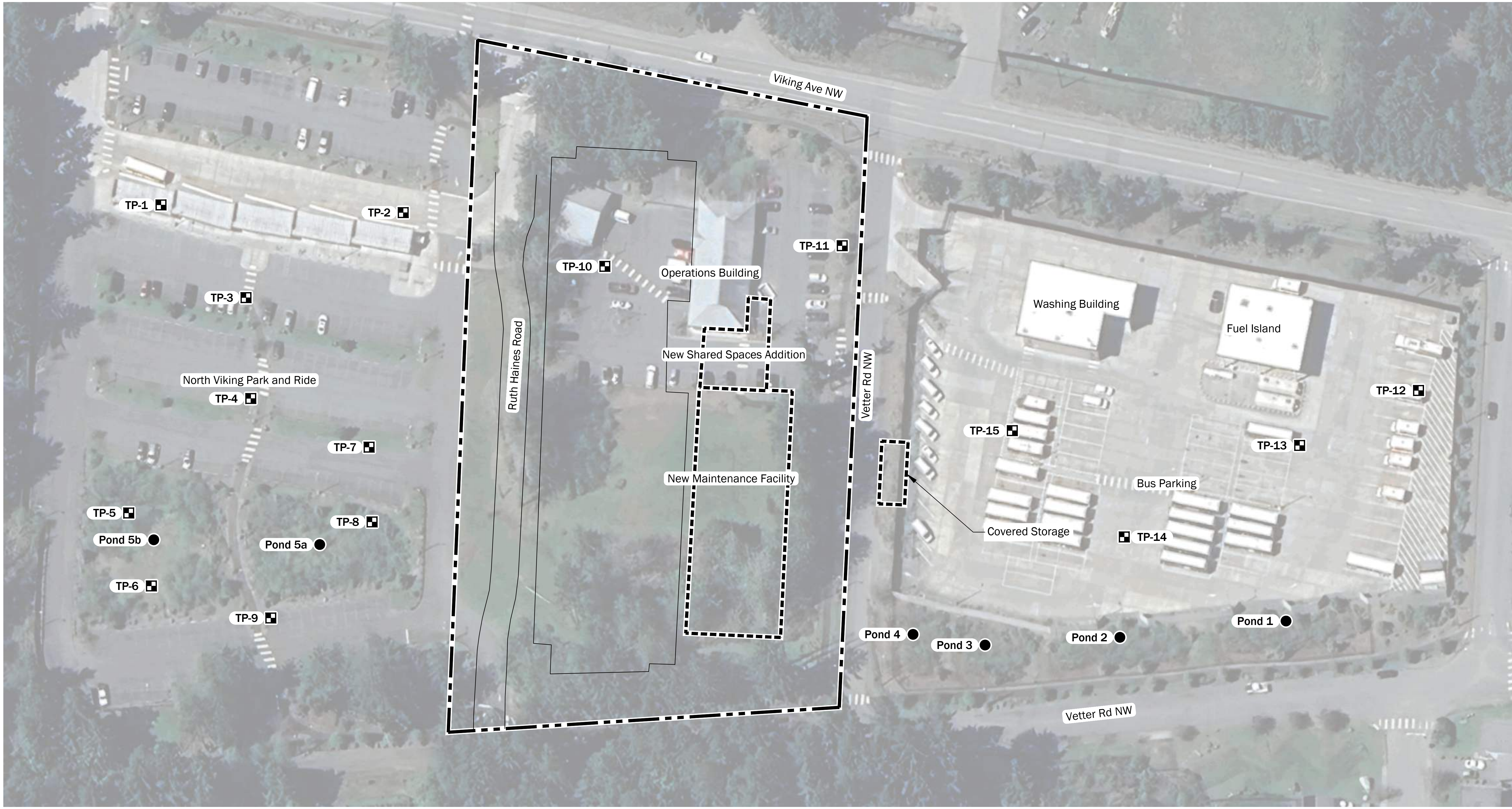

Legend

- Site Boundary
- Proposed New Structure
- Pond-1 ● Stormwater Pond Monitoring Location by GeoEngineers, Inc., 2017 (See 2017 Report for Additional Detail)
- TP-1 ▣ Test Pit by GeoEngineers, Inc., 2012 (See 2012 Report for Additional Detail)

N

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Feet

Source(s):

- Aerial from Google Earth Pro, dated 4/27/2023

Projection: WA State Plane, North Zone, NAD83, US Foot

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Previous Site Work

North Base Maintenance Facility
Poulsbo, Washington

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Figure 3

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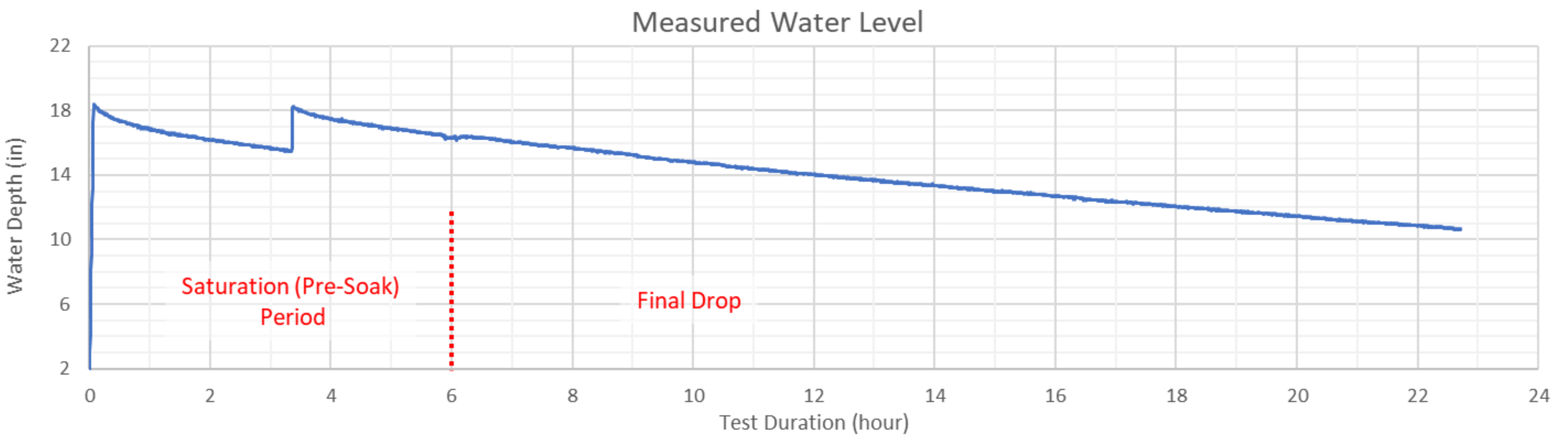
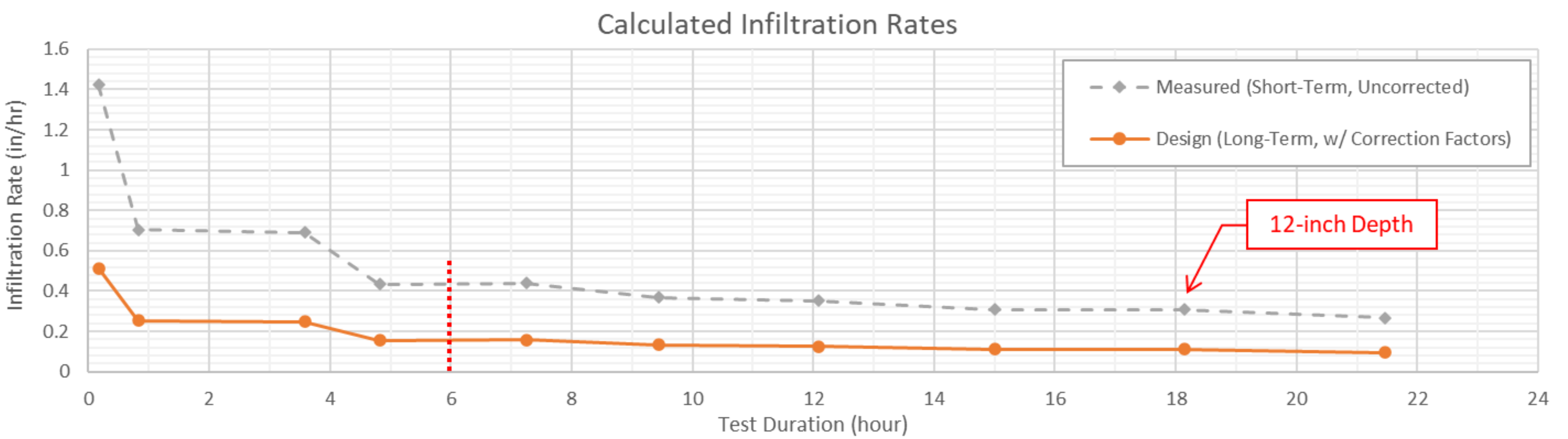
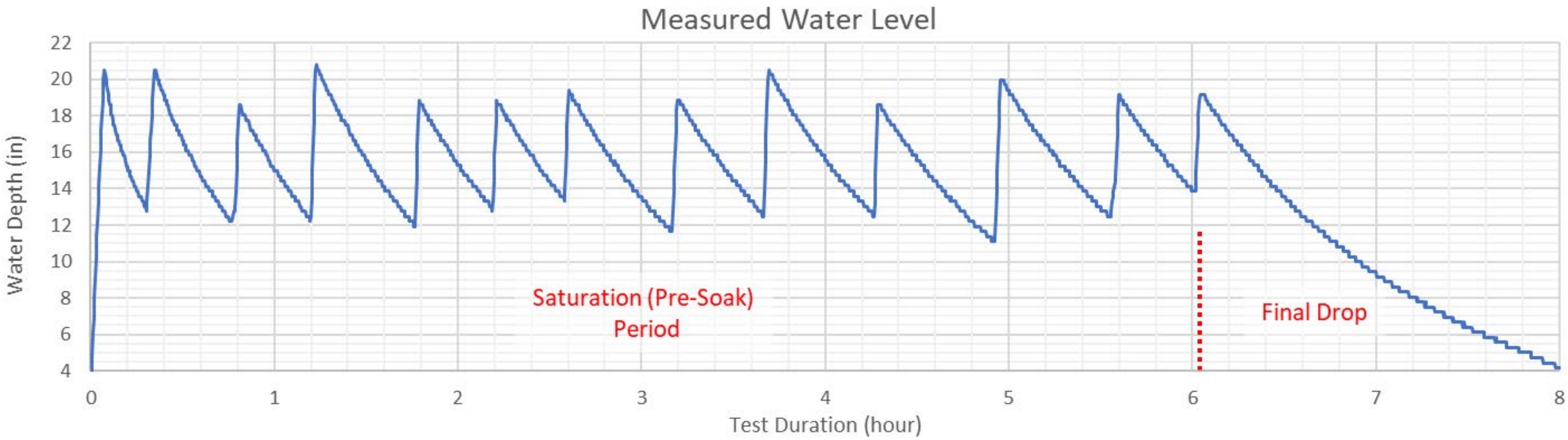


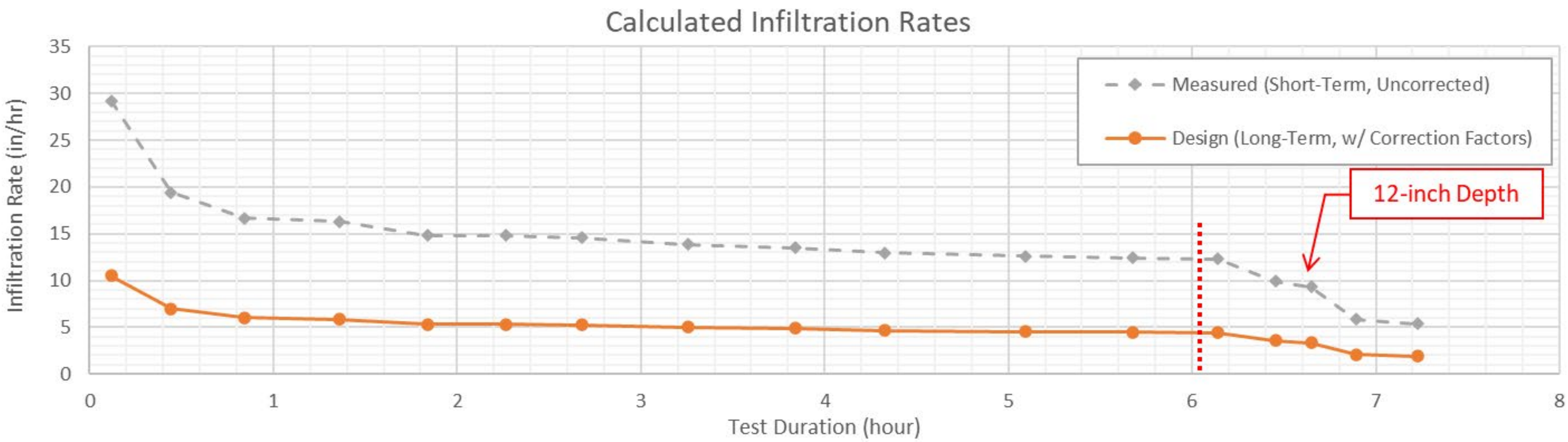


Note(s):

1. TP-2 Pilot Infiltration Test completed at 4 feet below ground surface. Standing water remained in hole overnight

TP-2 Pilot Infiltration Test Results	
North Base Maintenance Facility Poulsbo, Washington	
GeoEngineers	**Figure 4**





Note(s):

1. TP-5 Pilot Infiltration Test completed at 2.5 feet below ground surface. Water drained in hole overnight.

TP-5 Pilot Infiltration Test Results

North Base Maintenance Facility
Poulsbo, Washington

GEOENGINEERS

Figure 5

Appendices

Appendix A
Subsurface Explorations and Laboratory Testing

Appendix A
Subsurface Explorations and Laboratory Testing

SUBSURFACE EXPLORATIONS

Soil and groundwater conditions at the subject property were explored by advancing test pit explorations on July 19 and 20, 2023. Excavations were completed using a Takeuchi TB 138 rubber-tracked mini-excavator provided and operated by Kelly's Excavating, Inc. under subcontract to GeoEngineers. Locations of the explorations were determined in the field using an electronic tablet with global positioning system (GPS) software. Approximate locations of the explorations are shown on the Site Plan, Figure 2. The locations and elevations of the explorations should be considered approximate.

Ten test pits (TP-1 through TP-10) were advanced to depths between about 5.0 and 9.5 feet below ground surface (bgs), at which point digging refusal was encountered. Two small-scale pilot infiltration tests (PIT) were also completed at an intermediate depth in TP-2 and TP-5 to evaluate infiltration characteristics at the site.

Explorations were completed under observation by an engineer from our firm. Our field representative examined and classified the soils encountered, obtained soil samples, observed groundwater conditions and maintained a detailed log of the explorations. Recovered soils were visually classified in the field in general accordance with ASTM International (ASTM) D 2488 and the classification chart listed in Key to Exploration Logs, Figure A-1. Summary logs of the explorations are presented as Figures A-2 through A-11.

The logs are based on interpretation of the field and laboratory data and indicate the depth at which subsurface materials, or their characteristics change, although these changes might be gradual. If the change occurred between samples, it was interpreted. Soil densities noted on the logs are based on our judgment and experience of the conditions encountered, including difficulty of excavation and observations of caving.

Observations of groundwater conditions at the time of the explorations were made and are noted in the logs. Groundwater conditions observed during exploration represent a short-term condition, may or may not be representative of the long-term groundwater conditions at the site and should be considered approximate.

After each test pit was completed, the excavation was backfilled using the generated material and compacted in lifts using the heel of the excavator bucket.

PILOT INFILTRATION TESTING

PIT Methodology

Small-scale pilot infiltration tests (PITs) were completed following GeoEngineers' standard methodology for stormwater facilities in Western Washington. GeoEngineers' PIT procedure has been developed to provide increased confidence that fully saturated conditions have been achieved and the infiltration rate measured at the end of the test is representative of the saturated hydraulic conductivity of the soil. Our methodology is a synthesis of the standard practices and procedures set forth in the Washington State Department of Ecology Stormwater Management Manual of Western Washington (SMMWW). Our methodology is also consistent with procedures outlined in the Kitsap County Stormwater Design Manual (SDM).

Small-scale PITs were completed in test pits TP-2 and TP-5 at approximate depths of 4 feet (Elevation 178 feet) and 2.5 feet (Elevation 174.5 feet), respectively. The approximate dimensions of the base of the PIT excavations were 4 feet by 4 feet. Upon reaching the target depth for the PIT, a graduated yard stick was driven into the floor of the test pit as a visual reference for monitoring water levels during testing. A piezoelectric pressure transducer was secured to the bottom of the yard stick to provide water level measurements at 15-second intervals throughout the duration of the test. GeoEngineers' PIT procedure consists of a saturation period (pre-soak) where the water depth in the PIT is maintained above about 12 inches. During the saturation period, the water depth is raised and lowered in a series of falling-head stages. The approximate depths to which water is filled and allowed to drain are based on the minimum PIT water depth requirements in the SMMWW, anticipated water depths in the proposed facilities and the soil conditions present in the excavations. Water level measurements collected by the pressure transducer during each falling-head stage are used to calculate the apparent infiltration rate for each stage. The falling-head stage methodology is intended to fully saturate the soils below the base of the PIT while allowing for a direct measurement of when saturated or near-saturated conditions have been achieved. This is usually manifested by a progressive decline in the apparent infiltration rate until the rate approximately stabilizes. The stabilized rate corresponds to the saturated infiltration rate of the soil. For the completed PITs, relatively uniform infiltration rates were observed after the required minimum 6-hour saturation period.

Once the saturation time has elapsed and a stabilized infiltration rate is observed during the falling-head stages, the PIT is left undisturbed until the water drains away completely. The final drain down period shows how infiltration changes over a continuous range of declining water depths. According to the Kitsap County SDM, infiltration rates should be determined based on a water depth in the PIT between 6 and 12 inches. After the PITs are complete, the test pits are excavated deeper to obtain samples, observe for indications of lateral water flow and to characterize the underlying soils.

PIT Analysis

Infiltration rates calculated from the transducer data are representative of the measured (uncorrected) infiltration rate of the soils at the test location. Per the SDM, the design infiltration rate is determined by applying correction factors to the measured infiltration rate. The correction factors account for uncertainties in the site variability (CF_v), testing method (CF_t), and long-term siltation and bio-buildup (CF_m). Appendix G.4 of the SDM outlines correction factors appropriate for design and provides recommended values. We selected appropriate correction factors based on the recommended values, our project understanding, observed soil conditions and our experience. The table below summarizes the partial and total correction factor(s) that, in our opinion, are suitable for design. We should be contacted once facilities have been designed to confirm these correction factors are still appropriate.

CONSIDERED PIT CORRECTION FACTORS

ISSUE	CORRECTION FACTOR
Site Variability (CF_v)	0.8
Test Method (CF_t)	0.5
Siltation and Bio-Buildup (CF_m)	0.9
Total Correction Factor = CF_v x CF_t x CF_m	**CF = 0.36**

LABORATORY TESTING

Soil samples obtained from the explorations were retained in sealed plastic bags to prevent moisture loss and transported to the GeoEngineers' laboratory. Representative soil samples were selected for laboratory tests to evaluate the pertinent geotechnical engineering characteristics of the soils and to confirm our field classification. The following paragraphs provide a description of the tests performed.

Particle Size Gradation – Sieve Analysis (SA)

Sieve analyses were performed in general accordance with ASTM Test Method D 6913. This test method covers the quantitative determination of the distribution of particle sizes in soils. Typically, the distribution of particle sizes larger than 75 micrometers (μm) is determined by sieving. Figures A-12 and A-13 present the results of our sieve analyses.

Percent Fines (%F)

Percent fines content represents the percentage by weight of the sample passing (finer than) the U.S. No. 200 sieve. Samples were "washed" through the U.S. No. 200 sieve to estimate the relative percentages of coarse- and fine-grained particles in the soil in general accordance with ASTM D 1140. Test results are presented on the exploration logs at the respective sample depths.

Moisture Content (MC)

Moisture content was determined in general accordance with ASTM Test Method D 2216. The test results are used to aid in soil classification and correlation with other pertinent engineering soil properties. The test results are presented on the exploration logs, as indicated for the sample tested.

Organic Content (OC)

Organic content (OC) is the fraction of the soil that consists of plant or animal tissue in various stages of decomposition; it influences a soil's water infiltration characteristics and water holding capacity and provides a buffer against soil acidification. OC tests were determined in accordance with ASTM D 2974. Test results are indicated on the exploration logs, as appropriate.

Cation Exchange Capacity (CEC)

Cation exchange capacity (CEC) is a measure of the soil's ability to hold positively charged ions. It influences soil structure stability, nutrient availability, soil pH and the soil's reaction to fertilizers and provides a buffer against soil acidification. CEC tests were completed at an off-site laboratory (Analytical Resources, LLC located in Tukwila, Washington) under subcontract to GeoEngineers. Test results are indicated on the exploration logs, as appropriate.

SOIL CLASSIFICATION CHART

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

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

Sampler Symbol Descriptions

- Modified California Sampler (6-inch sleeve) or Dames & Moore
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- Direct-Push
- Bulk or grab
- Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	SOD	Sod/Forest Duff
	TS	Topsoil

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PL	Point load test
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
UU	Unconsolidated undrained triaxial compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

Key to Exploration Logs

GeoEngineers

Figure A-1

Date Excavated 7/19/2023	Total Depth (ft) 9	Logged By LSP Checked By ST	Excavator Kelly's Excavating, Inc. Equipment Hitachi TB 138 Mini Excavator	Groundwater not observed Caving not observed

Surface Elevation (ft) 183 Vertical Datum NAVD88	Easting (X) 1193537 Northing (Y) 282655	Coordinate System WA State Plane South Horizontal Datum NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
183	0				SOD	Approximately 2 to 3 inches of sod			
					SM	Grayish brown silty fine sand with occasional gravel and trace organic matter (fine roots) (loose, moist) (fill)			
182	1				SM	Brown silty fine to medium sand with gravel and occasional cobbles (medium dense, moist) (weathered till)			
181	2		1						
180	3				SM	Gray silty fine to medium sand with gravel and occasional cobbles (very dense, moist) (intact till)			
179	4		2						Very slow digging at 4 feet Cemented soil matrix
178	5								
177	6								
176	7		3						
175	8								
174	9								

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-1

GeoEngineers

Project: North Base Maintenance Facility
Project Location: Poulsbo, Washington
Project Number: 6193-025-00

Date Excavated	7/19/2023	Total Depth (ft)	8	Logged By	LSP	Excavator	Kelly's Excavating, Inc.	Groundwater not observed
				Checked By	ST	Equipment	Hitachi TB 138 Mini Excavator	Caving not observed

Surface Elevation (ft)	182	Easting (X)	1193622	Coordinate System	WA State Plane South
Vertical Datum	NAVD88	Northing (Y)	282685	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
182	0				SOD	Approximately 1 to 2 inches of sod			
					SM	Light brown silty fine sand with gravel, occasional cobbles and trace organic matter (fine roots) (medium dense, moist) (weathered till)			
181	1								
180	2				SM	Gray silty fine to medium sand with gravel and occasional cobbles (very dense, moist) (intact till)			Cemented soil matrix
179	3								
178	4		1 CEC; OC; SA				7	30	PIT completed at 4 feet, standing water remained in hole overnight, moist soiled observed on continued excavation CEC = 2.1 meq/100g OC = 0.8%
177	5								
176	6		2						
175	7								
174	8								

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-2 (w/ PIT)

GEOENGINEERS

Project: North Base Maintenance Facility
Project Location: Poulsbo, Washington
Project Number: 6193-025-00

Date Excavated 7/19/2023	Total Depth (ft) 6	Logged By LSP Checked By ST	Excavator Kelly's Excavating, Inc. Equipment Hitachi TB 138 Mini Excavator	Groundwater not observed Caving not observed

Surface Elevation (ft) 180 Vertical Datum NAVD88	Easting (X) 1193536 Northing (Y) 282568	Coordinate System WA State Plane South Horizontal Datum NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				SOD	Approximately 2 to 3 inches of sod			
					SM	Light brown silty fine sand with occasional gravel and trace organic matter (fine roots) (loose, moist) (fill)			
179	1				SM	Brown silty fine to medium sand with occasional gravel (medium dense, moist) (recessional outwash)			
178	2								
177	3								
176	4		1 / %F				10	16	
			2		SP-SM	Brown and gray fine to medium sand with silt and trace gravel (medium dense, moist)			
175	5				SM	Gray with occasional oxidation staining silty fine to medium sand with gravel and occasional cobbles (very dense, moist) (intact till)			Cemented soil matrix
			3						
174	6								Digging refusal at 6 feet

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-3

GeoEngineers

Project: North Base Maintenance Facility
Project Location: Poulsbo, Washington
Project Number: 6193-025-00

Date Excavated	7/19/2023	Total Depth (ft)	7.5	Logged By	LSP	Excavator	Kelly's Excavating, Inc.	Groundwater not observed
				Checked By	ST	Equipment	Hitachi TB 138 Mini Excavator	Caving not observed

Surface Elevation (ft)	179	Easting (X)	1193623	Coordinate System	WA State Plane South
Vertical Datum	NAVD88	Northing (Y)	282611	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				SOD	Approximately 2 to 3 inches of sod			
					SM	Grayish brown silty fine to medium sand with occasional gravel and trace organic matter (fine roots) (medium dense, moist) (fill)			
178	1		1 SA				8	26	
177	2				SM	Light brown silty fine to medium sand with gravel and trace cobbles (medium dense, moist) (weathered till)			
176	3		2						
175	4					Becomes brown with burnt organic matter			
174	5								
	5.5				SM	Gray and brown with oxidation staining silty fine to medium sand with gravel, cobbles and occasional boulders (very dense, moist) (intact till)			6- to 18-inch cobble and boulder at 5½ feet, cemented soil matrix
173	6								
172	7		3 SA				11	33	

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-4

GEOENGINEERS

Project: North Base Maintenance Facility
Project Location: Poulsbo, Washington
Project Number: 6193-025-00

Date Excavated	7/19/2023	Total Depth (ft)	6	Logged By	LSP	Excavator	Kelly's Excavating, Inc.	Groundwater not observed
				Checked By	ST	Equipment	Hitachi TB 138 Mini Excavator	Caving not observed

Surface Elevation (ft)	177	Easting (X)	1193606	Coordinate System	WA State Plane South
Vertical Datum	NAVD88	Northing (Y)	282545	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
					SOD	Approximately 1 to 2 inches of sod			
					SM	Grayish brown silty fine sand with occasional gravel and trace organic matter (fine roots) (loose, moist) (fill)			
					SM	Light brown silty fine sand with occasional gravel and trace cobbles (medium dense, moist) (weathered till)			
176	1		1						
175	2								
174			2 CEC; OC; SA				6	19	PIT completed at 2-1/2 feet, water completely drained from hole overnight, moist soils observed on continued excavation CEC = 0.9 meq/100g OC = 1.7%
	3								
					SM	Gray with moderate oxidation staining silty fine sand with gravel and occasional cobbles (very dense, moist) (intact till)			
173	4		3						Cemented soil matrix
172	5								
171			5			Becomes without oxidation staining at 5½ feet			
	6		4						Digging refusal at 6 feet

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on Topographic Survey.

Date:6/25/24 Path:\\GEOENGINEERS.COM\WAN\PROJECTS\6\6193025\GINT\619302500.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_TESTPIT_1P_GEOTEC_%F

Log of Test Pit TP-5 (w/ PIT)

GEOENGINEERS

Project: North Base Maintenance Facility
Project Location: Poulsbo, Washington
Project Number: 6193-025-00

Date Excavated	7/20/2023	Total Depth (ft)	5	Logged By	LSP	Excavator	Kelly's Excavating, Inc.	Groundwater not observed
				Checked By	ST	Equipment	Hitachi TB 138 Mini Excavator	Caving not observed

Surface Elevation (ft)	179	Easting (X)	1193697	Coordinate System	WA State Plane South
Vertical Datum	NAVD88	Northing (Y)	282694	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0			SOD	Approximately 2 to 3 inches of sod			
				SM	Grayish brown silty fine sand with occasional gravel and trace organic matter (fine roots) (loose, moist) (fill)			
178	1			SM	Light brown silty fine to medium sand with gravel and cobbles (medium dense, moist) (weathered till)			
177	2							
176	3			SM	Gray with moderate oxidation staining silty fine to medium sand with gravel and occasional cobbles (very dense, moist) (intact till)			Hard digging at 3 feet
175	4	[sample interval 3.5–4]	1					
					Becomes without oxidation staining at 4½ feet			Cemented soil matrix
174	5							

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-6

GEOENGINEERS

Project: North Base Maintenance Facility
Project Location: Poulsbo, Washington
Project Number: 6193-025-00

Date Excavated 7/20/2023	Total Depth (ft) 6	Logged By LSP Checked By ST	Excavator Kelly's Excavating, Inc. Equipment Hitachi TB 138 Mini Excavator	Groundwater not observed Caving not observed	

Surface Elevation (ft) 178 Vertical Datum NAVD88	Easting (X) 1193705 Northing (Y) 282631	Coordinate System WA State Plane South Horizontal Datum NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				SOD	Approximately 2 to 3 inches of sod			
					SM	Grayish brown silty fine sand with gravel and occasional cobbles (medium dense, moist) (fill)			
177	1				SM	Brown silty fine sand with gravel and occasional cobbles (medium dense, moist) (weathered till)			
176	2		1 SA				6	22	
175	3								
174	4				SM	Gray with oxidation staining silty fine to medium sand with gravel and occasional cobbles (very dense, moist) (intact till)			
173	5		2						
						Becomes without oxidation staining at 5½ feet			
172	6								

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-7

GEOENGINEERS

Project: North Base Maintenance Facility
Project Location: Poulsbo, Washington
Project Number: 6193-025-00

Date Excavated	7/20/2023	Total Depth (ft)	9.5	Logged By	LSP	Excavator	Kelly's Excavating, Inc.	Groundwater not observed
				Checked By	ST	Equipment	Hitachi TB 138 Mini Excavator	Caving not observed

Surface Elevation (ft)	180	Easting (X)	1193695	Coordinate System	WA State Plane South
Vertical Datum	NAVD88	Northing (Y)	282510	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
180	0				SOD	Approximately 1 to 2 inches of sod			
					SM	Grayish brown silty fine sand with gravel and trace organic matter (fine roots) (medium dense, moist) (fill)			
179	1								
178	2				SP-SM	Brown fine to medium sand with silt, gravel, occasional cobbles and trace organic matter (wood debris) (medium dense, moist) (recessional outwash)			
177	3		1 / %F				5	10	
176	4								
175	5								
174	6				SM	Gray with oxidation staining silty fine to medium sand with gravel and occasional cobbles (dense, moist) (intact till)			Slow excavation at 6 feet Cemented soil matrix
173	7		2						
172	8								
171	9		3						

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-8

GEOENGINEERS

Project: North Base Maintenance Facility
Project Location: Poulsbo, Washington
Project Number: 6193-025-00

Date Excavated	7/20/2023	Total Depth (ft)	9	Logged By	LSP	Excavator	Kelly's Excavating, Inc.	Groundwater not observed
				Checked By	ST	Equipment	Hitachi TB 138 Mini Excavator	Caving not observed

Surface Elevation (ft)	187	Easting (X)	1193302	Coordinate System	WA State Plane South
Vertical Datum	NAVD88	Northing (Y)	282584	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				DUFF	Approximately 4 inches of forest duff			
					SP-SM	Light brown fine sand with silt and occasional cobbles (medium dense, moist) (recessional outwash)			
186	1								
185	2	▯	1 %F				4	8	
184	3				SM	Brownish gray with oxidation staining silty fine to medium sand with occasional gravel and cobbles (dense, moist) (intact till)			
183	4	▯	2						Lightly cemented soil matrix
182	5								
181	6								
		▯	3			Becomes without oxidation staining			
180	7								
						Becomes very dense			Cemented soil matrix
179	8								
		▯	4						
178	9								

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-9

GEOENGINEERS

Project: North Base Maintenance Facility
Project Location: Poulsbo, Washington
Project Number: 6193-025-00

Date Excavated	7/20/2023	Total Depth (ft)	6	Logged By	LSP	Excavator	Kelly's Excavating, Inc.	Groundwater not observed
				Checked By	ST	Equipment	Hitachi TB 138 Mini Excavator	Caving not observed

Surface Elevation (ft)	177	Easting (X)	1193544	Coordinate System	WA State Plane South
Vertical Datum	NAVD88	Northing (Y)	282490	Horizontal Datum	NAD83 (feet)

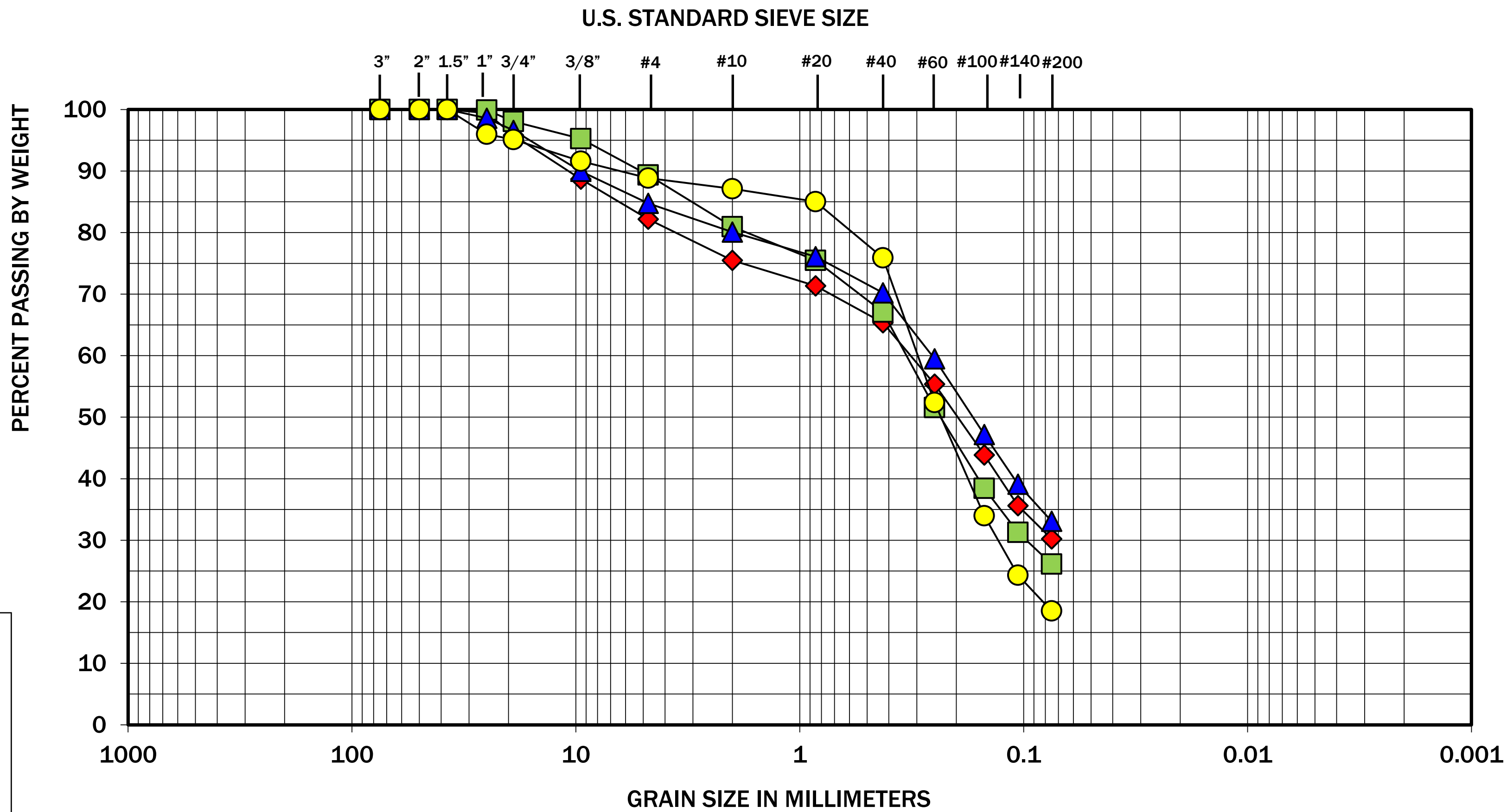
Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				DUFF	Approximately 4 inches of forest duff			
					SM	Grayish brown silty fine sand with occasional organic matter (roots and bark) loose, moist) (fill)			
176	1				SM	Light brown with oxidation staining silty fine sand with occasional gravel and trace cobbles (medium dense, moist) (weathered till)			
175	2								
174	3								Lightly cemented soil matrix
173	4		1 %F				7	41	
172	5		2		SM	Brownish gray with oxidation staining silty fine to medium sand with gravel, cobbles and occasional boulders (very dense, moist) (intact till)			Cemented soil matrix
171	6								Digging refusal at 6 feet

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on Topographic Survey.

Log of Test Pit TP-10

GEOENGINEERS

Project: North Base Maintenance Facility
Project Location: Poulsbo, Washington
Project Number: 6193-025-00



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	TP-2	4	7	Silty sand with gravel (SM)
■	TP-4	1	8	Silty sand (SM)
▲	TP-4	7	11	Silty sand with gravel (SM)
●	TP-5	2.5	6	Silty sand (SM)

AASHTO ACCREDITED

Note: This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc. Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations, or generated by separate operations or processes.

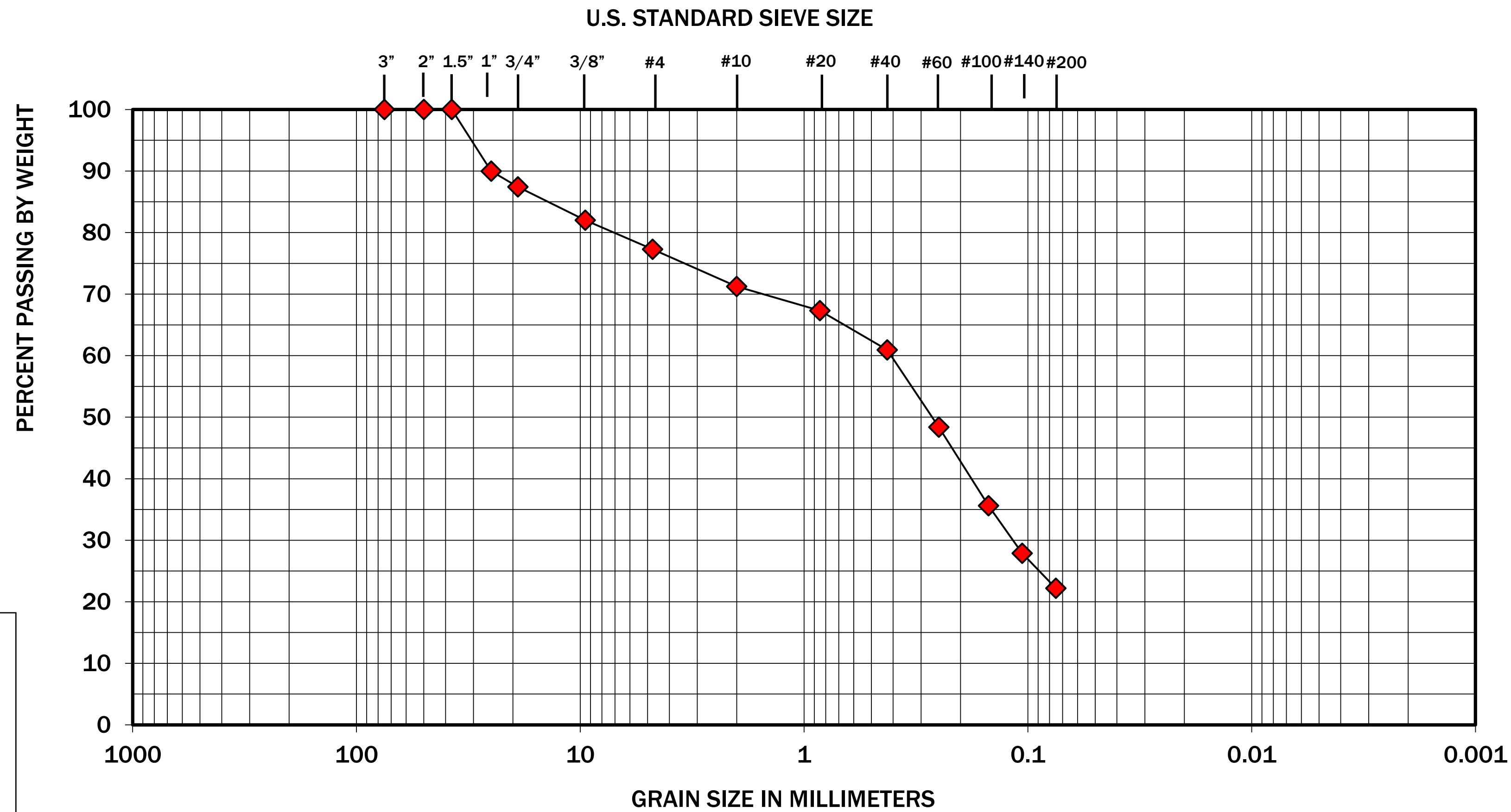
The grain size analysis results were obtained in general accordance with ASTM D6913. GeoEngineers 17425 NE Union Hill Road Ste 250, Redmond, WA 98052

GeoEngineers

North Base Maintenance Facility
Poulsbo, Washington

Sieve Analysis Results

Figure A-12



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	TP-7	2	6	Silty sand with gravel (SM)

The grain size analysis results were obtained in general accordance with ASTM D6913. GeoEngineers 17425 NE Union Hill Road Ste 250, Redmond, WA 98052

GEOENGINEERS

North Base Maintenance Facility
Poulsbo, Washington

Sieve Analysis Results

Figure A-13

Appendix B
Previous Subsurface Exploration Logs

SOIL CLASSIFICATION CHART

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

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	CC	Cement Concrete
	AC	Asphalt Concrete
	CR	Crushed Rock/ Quarry Spalls
	TS	Topsoil/ Forest Duff/Sod

Measured groundwater level in exploration, well, or piezometer

Groundwater observed at time of exploration

Perched water observed at time of exploration

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Material Description Contact

Distinct contact between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

A "P" indicates sampler pushed using the weight of the drill rig.

Laboratory / Field Tests

%F	Percent fines
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
OC	Organic content
PM	Permeability or hydraulic conductivity
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen
NT	Not Tested

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

KEY TO EXPLORATION LOGS

GEOENGINEERS

FIGURE A-1

Date Excavated: 11/1/2011

Equipment: Mini Excavator

Logged By: BEL

Total Depth (ft) 5.5

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				SM		Brown silty fine to medium sand with trace organics (loose, moist) (weathered till)		
	2	X	1 SA					13	%F=18
	3.5	X	2 MC					11	
	4.5						Grades to dense		
	5	X	3		SM		Brown silty fine to coarse sand (very dense, moist) (glacial till)		

Test pit completed at 5.5 feet
No groundwater seepage observed
No caving observed

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-10

GeoEngineers

Project: Proposed North Base Park and Ride
Project Location: Poulsbo, Washington
Project Number: 6193-010-03

Figure A-11
Sheet 1 of 1

Date Excavated: 11/1/2011

Equipment: Mini Excavator

Logged By: BEL

Total Depth (ft) 3.0

Elevation (feet)	Depth (feet)	SAMPLE Testing Sample	SAMPLE Sample Name Testing	Graphic Log	Group Classification	Encountered Water	MATERIAL DESCRIPTION	Moisture Content, %	REMARKS
	0				SM		Gray silty fine to coarse sand with gravel (very dense, moist) (glacial till)		
	1	X	1						
	2								
	3	X	2						Refusal

Test pit completed at 3 feet
No groundwater seepage observed
No caving observed

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to 0.5 foot.

Log of Test Pit TP-11

GEOENGINEERS

Project: Proposed North Base Park and Ride
Project Location: Poulsbo, Washington
Project Number: 6193-010-03

Figure A-12
Sheet 1 of 1

Appendix C
Report Limitations and Guidelines for Use

Appendix C
Report Limitations and Guidelines for Use[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory "limitations" provisions in its reports. Please confer with GeoEngineers if you need to know more how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical Services are Performed for Specific Purposes, Persons and Projects

This report has been prepared for TCF Architects, PLLC for the Project(s) specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the Project, and its schedule and budget, our services have been executed in accordance with the agreement between TCF Architects and GeoEngineers dated June 14, 2023 and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

A Geotechnical Engineering or Geologic Report is based on a Unique Set of Project-Specific Factors

This report has been prepared for the proposed North Base Maintenance Facility project in Poulsbo, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- Not prepared for you,
- Not prepared for your project,
- Not prepared for the specific site explored, or
- Completed before important project changes were made.

[1] Developed based on material provided by GBA, GeoProfessional Business Association; www.geoprofessional.org.

For example, changes that can affect the applicability of this report include those that affect:

- The function of the proposed structure;
- Elevation, configuration, location, orientation or weight of the proposed structure;
- Composition of the design team; or
- Project ownership.

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

Environmental Concerns are Not Covered

Unless environmental services were specifically included in our scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

Information Provided by Others

GeoEngineers has relied upon certain data or information provided or compiled by others in the performance of our services. Although we use sources that we reasonably believe to be trustworthy, GeoEngineers cannot warrant or guarantee the accuracy or completeness of information provided or compiled by others.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Geotechnical and Geologic Findings are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

Geotechnical Engineering Report Recommendations are Not Final

We have developed the following recommendations based on data gathered from subsurface investigation(s). These investigations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions. If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable but separating logs from the report can create a risk of misinterpretation.

Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- Advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- Encourages contractors to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer.

Contractors are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field.