Subsurface Exploration and
Geotechnical Engineering Evaluation

MILTON TELECARE FACILITY

Milton, Washington

Prepared For:
TELECARE CORPORATION

Project No. 170289E001
June 13, 2017

associated earth sciences incorporated

June 13, 2017
Project No. 170289E001

Telecare Corporation
1080 Marina Village Parkway, Suite 100
Alameda, California 94501

Attention: Matt Broz, Manager of Facilities Operations

Subject: Subsurface Exploration and Geotechnical Engineering Evaluation
Milton Telecare Facility
7224 Pacific Highway East
Milton, Washington

Dear Mr. Broz:

Associated Earth Sciences, Inc. (AESI) is pleased to submit this report describing our subsurface exploration and geotechnical engineering evaluation concerning the planned healthcare facility in Milton, Washington. Our services were completed in general accordance with our proposal dated May 5, 2017, and were authorized by your signature on May 8, 2017.

We have enjoyed working with you on this study and are confident that the recommendations presented in this report will aid in the successful completion of your project. If you should have any questions, or if we can be of additional help to you, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Tacoma, Washington

James M. Brisbine, P.E., L.G., L.E.G.
Senior Associate Geotechnical Engineer

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SUBSURFACE EXPLORATION AND GEOTECHNICAL ENGINEERING EVALUATION

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Milton, Washington

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1080 Marina Village Parkway, Suite 100
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1.0 PROJECT AND SITE DESCRIPTION

The project site comprises an undeveloped commercial parcel located in the southwestern part of Milton, as shown on the attached "Vicinity Map" (Figure 1). This parcel is visually delineated by a wooded property on the north, by a commercial property (Linwood Custom Homes) on the south, by Pacific Highway East (SR-99) on the west, and by Interstate-5 on the east. It has a trapezoidal shape that measures approximately 310 feet by 330 feet overall and covers about 2 acres. Our attached "Site and Exploration Plan" (Figure 2) illustrates the site boundaries and adjacent features.

Development plans call for constructing a new healthcare building in the center of the site. We understand that this new building will be a one-story, at-grade structure covering about 12,000 square feet. We assume that the building walls and columns will impose relatively low to moderate foundation loads. A parking lot is planned for the southwestern portion of the site, walkways or patios will surround most of the building, and several retaining walls will be built east and north of the building. We assume that stormwater will be infiltrated in a subsurface vault or gallery if soil conditions are found to be receptive; otherwise, stormwater will likely be detained in a pond or vault and then conveyed off site. It is also possible that permeable pavements will be used onsite if found to be feasible. Figure 2 shows the proposed layout of the new building, parking lot, and other features.

We understand that a Phase I and Limited Phase II Environmental Assessment was recently conducted at the site by Farallon Consulting, LLC (Farallon) to identify possible soil contaminants. Part of their data was obtained from a Phase II Environmental Assessment performed by Environmental Associates, Inc. (EAI) in 2008. Among these two studies, numerous subsurface explorations were performed at the site (using direct-push methods) and a temporary ground water monitoring well was installed. EAI identified up to 20 feet of fill overlying the site. Farallon concluded that on-site contaminants within this fill are below regulatory cleanup levels, but they did recommend a conservative soil-handling strategy for all on-site excavations.

2.0 PURPOSE AND SCOPE

Associated Earth Sciences, Inc. (AESI) performed this study to characterize subsurface conditions below the site, such that we can derive geotechnical conclusions and recommendations concerning design criteria and earthwork considerations. Our scope of work included the following tasks.

- Reviewed topographic maps, geologic maps, site layout drawings, aerial photos, and other available information pertaining to the site vicinity.
- Performed a visual surface reconnaissance of the site and immediate surroundings.

- Advanced five exploration borings (designated EB-1 through EB-5) to a maximum depth of about 41½ feet, at strategic locations across the site.
- Visually classified all soil samples obtained from our explorations.
- Conducted laboratory grain-size (sieve) tests on two samples of near-surface soils.
- Submitted one sample of the near-surface soils to an independent analytical laboratory for cation-exchange capacity and organic content testing.
- Analyzed all research, field, and laboratory data in context with the proposed site development features.
- Prepared this report summarizing our geotechnical findings, conclusions, and recommendations.

Figure 2 shows the locations of all subsurface explorations with respect to existing and proposed site features. Appendix A contains our exploration logs, and Appendix B contains our laboratory testing results.

3.0 FIELD EXPLORATION PROCEDURES

We explored subsurface conditions at the site on June 6, 2017. The number, locations, and depths of our explorations were completed within the constraints of surface access, utility conflicts, and project budgets. Our exploration procedures are described below. The various types of sediments, as well as the depths where characteristics of the sediments changed, are indicated on the exploration logs presented in Appendix A. Soil contact depths shown on the logs should be regarded as only an approximation; the actual changes between sediment types are often gradational and/or undulating.

The conclusions and recommendations presented in this report are based, in part, on conditions encountered by our explorations completed for this study. Due to the nature of subsurface exploratory work, it is necessary to interpolate and extrapolate soil conditions between and beyond the field explorations. Differing subsurface conditions could be present outside the area of the explorations due to the random nature of deposition and the alteration of topography by past grading and/or filling. The nature and extent of any variations between the field explorations might not become fully evident until construction. If variations are observed at that time, it could be necessary to modify specific conclusions or recommendations in this report.

3.1 Exploration Borings

All exploration borings were performed by Holocene Drilling, Inc., working under subcontract to AESI. Each boring was completed by advancing an 8-inch outside-diameter, hollow-stem auger with a track-mounted drill rig. During the drilling process, disturbed but representative soil samples were obtained at 2½- or 5-foot-depth intervals using the Standard Penetration Test (SPT) procedure in accordance with the *American Society for Testing and Materials* (ASTM D-1586). After drilling, each borehole was backfilled with bentonite chips, and the surface was patched with concrete.

The SPT testing and sampling procedure consists of driving a standard, 2-inch outside-diameter, split-barrel sampler a distance of 18 inches into the soil with a 140-pound hammer free-falling a distance of 30 inches. The number of blows for each 6-inch interval is recorded, and the number of blows required to drive the sampler the final 12 inches represents the Standard Penetration Resistance (also known as the "N-value"). If a total of 50 blows is reached within one 6-inch interval, the N-value is recorded as 50 blows for the corresponding number of inches of penetration. The N-value provides a measure of the relative density of granular soils or the relative consistency of cohesive soils. Higher N-values correspond to a denser or stiffer soil. Our measured N-values are plotted on the exploration boring logs presented in Appendix A.

All exploration borings were continuously observed and logged by an AESI geologist. The samples obtained from the split-barrel sampler were classified in the field, and representative portions were placed in watertight containers. The samples were then transported to our laboratory for further visual classification. Soil descriptions shown on our exploration logs are based on N-values, drilling action, field observations, and laboratory classifications.

4.0 SITE CONDITIONS

The following text sections describe current site conditions, including development features, vegetation, regional and local topography, regional geology, local soils, and local ground water. Our sources of information include topographic and geologic maps published by the U.S. Geological Survey (USGS) and aerial photographs published by Google Earth.

4.1 Development Features and Vegetation

Presently, the site is vacant and undeveloped except for a small stormwater pond located on the east-central side of the property at the top of a slope. A drainage ditch parallels the eastern slope in a roughly north– south direction and directs water into the pond. A fence runs along the northern, western, and southern property boundaries. Some mature trees are present along the northern property boundary, but the majority of the property is vegetated only with grasses and sparse shrubs.

4.2 Regional and Local Topography

The project site is situated at the base of the southeastern slope of Fife Heights. Regional surface grades slope moderately to steeply downward from Fife Heights toward Hylebos Creek and the Puyallup River. The western and central portions of the site slope gently eastward, with an elevation ranging from about 30 to 42 feet (NGVD 29). The eastern and northern portions of the site slope steeply down toward Interstate 5 and the undeveloped north adjacent property, respectively, with an elevation of about 15 to 30 feet.

4.3 Regional Geology

The site straddles two geologic maps: the 2004 USGS *Geologic Map of the Poverty Bay 7.5 Minute Quadrangle, King and Pierce Counties, Washington*, and the 2006 USGS draft *Puyallup 7.5 Minute Quadrangle.* Both maps indicate that the site is underlain by Vashon-age recessional outwash and recessional lacustrine deposits (designated Qvr and Qvrl, respectively). Vashon recessional outwash was deposited in glacial outwash channels during the retreat of the Vashon ice sheet about 12,500 to 13,000 years ago, at the end of the Fraser glaciation. These deposits generally consist of stratified sand and gravel with minor amounts of silt. The Vashon recessional lacustrine sediments were deposited in glacial lakes during the retreat of the same ice sheet, and these sediments consist of fine-grained sand, silt, and clay.

According to the maps, the Fife Heights upland is underlain by Vashon-age lodgement till, Vashon-age advance outwash, pre-Olympia-age glacial deposits (designated Qvt, Qva, and Qpog, respectively). These various deposits are expected to extend underneath the on-site recessional outwash and recessional lacustrine deposits.

4.4 Local Soils

Our exploration borings confirmed the presence of Vashon recessional lacustrine deposits, Vashon advance outwash, and pre-Olympia deposits below the site, as indicated on the above-referenced geology maps. We also encountered a surficial layer that appears to be consistent with the fill soil previously described in environmental reports by EAI and Farallon. On-site soil conditions are summarized in the paragraphs below and are detailed on the attached exploration boring logs.

Uncontrolled Fill Soil: All five exploration borings revealed a thick layer of silty sand with minor amounts of gravel and silt. Densities ranged from loose to medium dense, or medium stiff to stiff. Roots and other organic matter was prevalent throughout the layer in some borings. We interpret this layer to be uncontrolled fill soil associated with past grading activities. The observed fill thickness ranged from about 13 to 23 feet at each boring, except EB-5, which did not penetrate the full thickness of the fill soils. For convenience, the observed fill thicknesses are noted on Figure 2.

Lacustrine Soil: Exploration borings EB-1 through EB-4, which were located within the proposed building footprint, encountered about 5 to 17 feet of medium stiff to hard, sandy silts, silty clays, and clays, as well as some medium dense, very fine sands, and silty sands. Organic matter was prevalent throughout the deposit. We interpret this soil to be a recessional glacial lacustrine (lake) deposit.

Advance Outwash and/or Pre-Olympia Glacial Deposits: Underlying the recessional lacustrine deposits, exploration borings EB-1 through EB-4 encountered stratified, medium dense to very dense sand and silty sand, as well as laminated, very stiff to hard silt. We infer this soil to be a Vashon-age advance outwash and/or older pre-Olympia glacial deposit. These deposits were first observed at depths ranging from 28 to 33 feet below ground surface, and they extended beyond our maximum exploration depth of 41½ feet at the site.

4.5 Local Ground Water

Four exploration borings, EB-1 through EB-4, encountered ground water at the time of drilling, at depths ranging from 20 to 32 feet below ground surface. We interpret this ground water to be associated with a regional shallow ground water table. It should be noted, however, that ground water levels can vary significantly with seasonal precipitation, on- and off-site land usage, and other factors. It is also likely that perched ground water zones form atop the subsurface silt lenses and other impermeable soil horizons during the wet season or immediately after periods of heavy precipitation.

5.0 INFILTRATION ASSESSMENT

Based on the 2015 *Pierce County Stormwater and Site Development Manual* (SWSDM), low-impact development (LID) features such as permeable pavements, infiltration systems, and/or bioretention swales are generally required unless certain site conditions render them infeasible. In order to assess the feasibility of infiltrating stormwater on the project site, we analyzed our subsurface data and also performed specific laboratory tests. The following text sections describe our findings and interpretations.

5.1 Stratigraphic and Hydrologic Considerations

As previously mentioned, our exploration borings revealed that the site is underlain by 13 to 23 feet of uncontrolled fill materials. This fill deposit comprises random layers of silts, silty sands, and sandy silts, all of which possess low to very low permeability characteristics. It is also possible that shallow zones of perched water form within the fill layers during the wet season. Overall, these subsurface conditions are not suitable for shallow or intermediate-depth stormwater infiltration systems such as ponds, galleries, and trenches. However, permeable pavements might be feasible at the site if an adequate thickness of base rock is provided for detention purposes. We infer that pavement subgrade infiltration rates would vary greatly with location and depth.

5.2 Water Treatment Considerations

For water treatment purposes, the SWSDM requires that infiltration subgrade soils have a cation exchange capacity (CEC) greater than 5 milliequivalents per 100 grams (meq/100g) and an organic content (OC) of at least 1 percent. We submitted a sample of near-surface soils to Spectra Labs in Tacoma, for determination of the CEC and organic content OC. This sample was obtained from a depth range of 1 to 3 feet below ground surface, in order to represent the receptor subgrades for possible future permeable pavement sections. The laboratory testing results will be presented in the final version of this report.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on our surface reconnaissance, subsurface explorations, and document research, we conclude that the proposed site development is feasible from a geotechnical standpoint, contingent on proper design implementation and construction practices. Most notably, the presence of a relatively thick deposit of uncontrolled fill material will create special design and construction challenges. Our geotechnical conclusions and recommendations concerning general considerations, site preparations, excavations, foundations, slab-on-grade floors, drainage systems, retaining walls, pavement sections, and structural fill are presented in subsequent text sections.

Specification Codes: The following reference documents are cited for specification purposes within this report.

- ASTM: Refers to the latest manual published by the *American Society for Testing and Materials* (ASTM).
- WSDOT: Refers to the 2016 edition of *Standard Specifications for Road, Bridge, and Municipal Construction* published by the Washington State Department of Transportation (WSDOT).

6.1 General Considerations

We offer the following comments, conclusions, and recommendations concerning general geotechnical design and construction issues affecting the overall project.

Geological Hazards: Our evaluation did not reveal any geological hazards associated with steep slopes, erosion zones, landslide zones, or abandoned landfills in the site vicinity. In addition, we infer that the dense and/or fine-grained soils underlying the site represent a negligible hazard with respect to seismically induced liquefaction. Earthquake activity is obviously a widespread hazard throughout Western Washington, but the risk of associated shaking and ground rupture does not appear to be any higher at this site than elsewhere in Pierce County. Consequently, the proposed site development is not constrained by any prevailing geological hazards, in our opinion.

Foundation Considerations: Our subsurface explorations encountered a thick deposit of uncontrolled fill soils and some loose native soils underlying the proposed building footprint. In our opinion, these soils are not suitable for foundation bearing purposes, due to the risk of excessive total and differential building settlements. Consequently, we recommend one of the following foundation options.

1. Support the building directly on augercast concrete piles that extend downward to native bearing soils. This option provides the most reliable settlement performance but entails a significant cost for pile installation. Details are provided in Section 6.3 of this report.
2. Support the building on conventional spread footings that are in turn supported by compacted aggregate piers extending through the uncontrolled fill deposit. This option provides adequate settlement performance at a somewhat lower cost than augercast concrete piles. Details are provided in Section 6.4 of this report.
3. Support the building on conventional spread footings that bear on a preloaded subgrade. This option provides adequate settlement performance but entails a significant cost for embankment construction and requires several weeks or months of preloading time. Details are provided in Section 6.5 of this report.

Retaining Wall Considerations: Retaining walls are currently planned for several locations across the site. In our opinion, backfilled concrete walls can be used for each location if appropriate subgrade remediation is performed to reduce long-term settlements of the existing fill soils. However, a favorable alternative would be reinforced-soil walls, which are inherently more flexible and settlement-tolerant. AESI can submit a scope and fee estimate for design of reinforced-soil walls upon request.

Seismic Site Class: The 2015 *International Building Code* (IBC) assigns a seismic Site Class on the basis of geological conditions prevailing within a depth of 100 feet below the local ground surface. Although our subsurface explorations did not extend to such a depth, we infer from shallower soil observations and from available geologic maps that the subsurface conditions correspond to Site Class "D" as defined by the IBC.

Infiltration Potential: The thick deposit of uncontrolled fill underlying the site has low to very low permeability characteristics, with the potential for shallow perched water to form during the wet season. As a result, we conclude that the site is not suitable for shallow or intermediate-depth stormwater infiltration systems such as ponds, galleries, and trenches. However, permeable pavements might be feasible at the site if an adequate thickness of base rock is provided for detention purposes. In the event that permeable pavements are desired, field infiltration tests should be performed to determine a design infiltration rate. AESI would be happy to submit a scope and fee estimate for such additional work upon request.

Earthwork Scheduling: Our explorations indicate that much of the near-surface on-site soils comprise silty sands and sandy silts. These silty soils are moisture-sensitive and highly susceptible to disturbance when wet. As such, earthwork should be scheduled for the dry season in order to minimize subgrade problems and to maximize the potential for reusing on-site soils. Greater export and import quantities should be expected during the wet season.

6.2 Site Preparation

Preparation of the project site will involve tasks such as temporary drainage, stripping, cutting, erosion control, and subgrade compaction. The paragraphs below present our geotechnical comments and recommendations concerning these various site issues.

Temporary Drainage: Any sources of surface or near-surface water that could potentially enter the construction zones should be intercepted and diverted before stripping and excavating activities begin. We tentatively anticipate that a system of temporary swales or berms placed around the construction zone will adequately intercept most off-site surface water runoff. Because the selection of an appropriate drainage system will depend on the water quantity, season, weather conditions, construction sequence, and contractor's methods, final decisions regarding temporary drainage details are best made in the field at the time of construction.

Clearing and Stripping: After surface and near-surface water sources have been controlled, the construction zones should be cleared and stripped of all existing vegetation, sod, topsoil, pavements, and other surface features. Our exploration borings disclosed about 3 to 6 inches of sod and root mat mantling the site. However, the actual thicknesses could vary considerably from one location to another.

Site Excavations: Based on our exploration borings, on-site excavations will generally encounter low- to moderate-density fill soils consisting of silty sands, sandy silts, and silts. We anticipate that all of these soils can be excavated with conventional earthworking equipment. Although none of our borings revealed boulders or debris within the fill, such obstructions could potentially be encountered at random locations beneath the site.

Temporary Cut Slopes: All temporary cut slopes associated with site grading and utility excavations should be suitably inclined to mitigate the potential for sloughing and collapse. For the various soil deposits that will likely be encountered during on-site earthwork, we tentatively infer that the following maximum inclinations (given as a horizontal to vertical, or "H:V" ratio) could be planned. However, appropriate inclinations will depend on the actual soil and ground water conditions encountered during earthwork. Ultimately, the site contractor must be responsible for maintaining safe excavation slopes that comply with applicable regulations.

Cuts in Loose to Medium Dense Silty Sands:	1.5H:1V
Cuts in Medium Stiff to Very Stiff Sandy Silts:	1.0H:1V

Weather Considerations: It should be realized that if the stripping or grading operations proceed during wet weather, greater stripping depths will likely be necessary to remove moisture-sensitive subgrade soil areas that become saturated and disturbed. For this reason, site earthwork should be avoided during periods of wet weather. During the summer months, sprinkling will likely be needed to moisture-condition soils that have become too dry.

Erosion Control Measures: Because stripped surfaces and soil stockpiles are typically a source of runoff sediments, they should be given particular attention. If earthwork occurs during wet weather, we recommend that all stripped surfaces be covered with straw to reduce runoff erosion. Similarly, soil stockpiles and cut slopes should be covered with plastic sheeting for erosion protection. We also recommend that silt fences, berms, and/or swales be maintained around stripped areas and stockpiles in order to capture runoff water and thereby reduce the downslope sediment transport. Stripped areas should be revegetated as soon as possible, also reducing the potential for erosion.

6.3 Augercast Concrete Piles

In our opinion, augercast concrete piles would provide favorable foundation support for the new building. Compared to other pile types, augercast concrete piles provide a moderate capacity at a moderate cost and offer great versatility. This foundation option would likely be more expensive than the alternatives described herein but would provide better long-term settlement performance. Our pile comments and recommendations are presented below.

Pile Installation: An augercast concrete pile is constructed by drilling a hole with a continuous-flight, hollow-stem auger, and then pumping concrete down the stem while withdrawing the auger. All piles should be installed with grout head of at least 5 feet, to help maintain shaft continuity. We also recommend that a minimum center-to-center spacing of five shaft diameters be maintained between piles installed within a 24-hour period.

Pile Sizes and Properties: Typical shaft diameters of augercast concrete piles can range from 12 to 24 inches in 2-inch or 4-inch increments. Considering that the new building will impose relatively low to moderate foundation loads, we anticipate that a diameter of 12, 14, or 16 inches would be appropriate. We assume that the following shaft stiffnesses (EI) would apply to these diameters.

12-inch diameter:	$EI = 3.5 \times 10^9$ pound-inch^2
14-inch diameter:	$EI = 6.1 \times 10^9$ pound-inch^2
16-inch diameter:	$EI = 10.5 \times 10^9$ pound-inch^2

Tip Depths: Augercast concrete piles will need to extend entirely through the deposit of uncontrolled fill soils and into medium dense or stiff native soils in order to gain adequate load resistance. Based on our exploration borings, we recommend that the piles for each wing of the new building extend to the following minimum tip depths. All tip depths are referenced to existing ground surface. The project structural engineer should be responsible for determining the spacing, diameters, and other details needed to accommodate building loads.

North Wing of Building:	30 feet
South Wing of Building:	35 feet
West Wing of Building:	25 feet
East Wing of Building:	33 feet

Static Compressive Capacities: We infer that all piles will mobilize compressive resistance through a combination of skin friction and end bearing. Our calculated allowable static compressive capacities for single piles are shown below. These values include a safety factor of 3.0 or more, but they do not include any reductions for group effects, which are discussed subsequently. Because existing grades will not be changed significantly, static downdrag on the piles is not anticipated.

12-inch diameter:	45 kips
14-inch diameter:	55 kips
16-inch diameter:	65 kips

Transient Compressive Capacities: Transient compressive loads may be imposed on the foundation piles by earthquake loads and wind loads. Our calculated allowable transient compressive capacities for single piles are shown below. These values include a safety factor of 2.0 or more but do not include any reductions for group effects, which are discussed subsequently. Because liquefaction of the subsurface soils appears unlikely, seismic downdrag on the piles is not anticipated.

12-inch diameter:	30 kips
14-inch diameter:	37 kips
16-inch diameter:	46 kips

Transient Uplift Capacities: Uplift forces, which may be imposed on the foundation piles by earthquake loads and wind loads, are resisted by skin friction acting along the pile shafts, as well as by the structure's dead load. Our calculated allowable transient uplift capacities for single piles are shown below. These values include a safety factor of 2.0 or more but do not include any reductions for group effects, which are discussed subsequently.

12-inch diameter:	31 kips
14-inch diameter:	37 kips
16-inch diameter:	43 kips

Lateral Capacities: Lateral forces may be imposed on the piles by earthquake loads and wind loads. The lateral capacity of a pile depends on its length, stiffness in the direction of loading, proximity to other piles, and degree of fixity at the head, as well as on the engineering properties of the upper 10 to 15 feet of in-situ soils. Our calculated lateral groundline capacities for free-head and fixed-head piles are shown below, based on an allowable groundline deflection of ½ inch. These values include a safety factor of 2.0 but do not include any reductions for group effects.

Free-Head Condition	
12-inch diameter:	5 kips
14-inch diameter:	9 kips
16-inch diameter:	14 kips
Fixed-Head Condition	
12-inch diameter:	12 kips
14-inch diameter:	16 kips
16-inch diameter:	20 kips

Additional Lateral Resistance: Piles gain additional lateral resistance from passive earth pressure acting against the pile caps, grade beams, and other embedded structural elements. We recommend using an allowable passive pressure (equivalent fluid weight) of 300 pcf for these embedded elements. If even more lateral resistance is needed, some of the piles could be battered (inclined) by as much as 20 degrees. It should be mentioned, however, that battered piles are more prone to construction problems.

Group Effects: Our calculated values for compressive, uplift, and lateral capacities refer to single piles unaffected by group interactions. To reduce or eliminate group effects, we recommend that the pile spacing never be less than three diameters (center to center). If piles are spaced at least three diameters apart, group effects can be neglected for both vertical loads and perpendicularly applied lateral loads. For in-line lateral loads, however, group effects reduce the lateral load capacity at a pile spacing less than eight diameters. The following capacity reduction factors should be applied to in-line laterally loaded piles with a center-to-center spacing between three and eight diameters.

Pile spacing of 3 diameters:	Reduction Factor = 0.4
Pile spacing of 4 diameters:	Reduction Factor = 0.5
Pile spacing of 6 diameters:	Reduction Factor = 0.8
Pile spacing of 8 diameters:	Reduction Factor = 1.0

Foundation Settlements: We estimate that total post-construction static settlements of structural elements supported on properly installed piles will not exceed ½ inch. We also estimate that differential settlements over the building's width and length will not exceed half of the total settlement.

Installation Monitoring: We recommend that an AESI geotechnical representative be retained to continuously monitor the installation of all augercast concrete piles, in order to verify that suitable tip depths are reached and proper procedures are followed. This monitoring program should include observation and documentation of construction equipment, pile materials, installation procedures, drilling conditions, and installation sequencing.

6.4 Compacted Aggregate Piers

Compacted aggregate piers can be used as a subgrade improvement technique to allow the use of conventional spread footings for new building support. Once installed, the piers provide direct bearing support of footings, and they also densify the subgrade areas surrounding each pier. This foundation option would likely be less expensive than augercast concrete piles but would probably not provide the same degree of long-term settlement performance. Our aggregate pier comments and recommendations are provided in the paragraphs below.

Pier installation: An aggregate pier is typically constructed by drilling or driving a circular hole and then backfilling it with compacted angular gravel. "Geopier" is a tradename for the most common type of aggregate pier, but other types are locally available. In all cases, the piers are installed in rows below continuous (wall) footings or in clusters below isolated (column) footings.

Pier Depths: Based on our exploration borings, we recommend that the aggregate piers for each wing of the new building extend to the following minimum tip depths. All tip depths are referenced to existing ground surface. The pier installation contractor may determine that greater depths are needed to achieve the minimum allowable static bearing capacity stated in the Spread Footings section of this report.

North Wing of Building:	23 feet
South Wing of Building:	17 feet
West Wing of Building:	13 feet
East Wing of Building:	20 feet

Other Pier Details: Due to the proprietary nature of aggregate piers, the installation contractor should be responsible for determining the spacing, diameters, materials, and other details needed to achieve the minimum allowable static bearing capacity stated in the Spread Footings section of this report. Design capacities should then be verified by means of modulus tests on a representative number of completed piers.

Installation Monitoring: We recommend that an AESI geotechnical representative be retained to continuously monitor the installation of all compacted aggregate piers, in order to verify that suitable tip depths are reached and proper procedures are followed. This monitoring program should include observation and documentation of construction equipment, pier materials, installation procedures, driving/drilling conditions, installation sequencing, and modulus testing.

6.5 Subgrade Preloading

The term *preloading* refers to the technique of placing a temporary fill embankment over a compressible subgrade and then waiting for a certain period of time before starting construction, thereby allowing the majority of settlement to occur in advance. If the new building footprint is properly preloaded, we infer that post-construction settlements of spread footings can be reduced to tolerable magnitudes. This foundation option might be less expensive than the alternatives described herein, but it would require more time to execute. Our preloading comments and recommendations are presented below.

Embankment Materials: When preloading a building footprint, it is usually most efficient to select an embankment material that can be reused elsewhere on the site, such as below floor slabs and pavements. This eliminates the time and cost associated with importing and exporting a single-use material. We therefore recommend using a well-graded sand and gravel, such as "Ballast" per WSDOT 9-03.9(1) or "Gravel Borrow" per WSDOT 9-03.14, for the preload embankment.

Embankment Construction: The preload embankment material should be placed in horizontal lifts approximately 12 inches thick, and each lift should be track-walked to achieve a uniformly firm condition. We recommend that the top of the embankment extend at least 5 feet beyond the entire building footprint, with side slopes no steeper than 1H:1V. In our opinion, an embankment height of 4 to 5 feet would likely be appropriate for the subject site and proposed building. Although a higher embankment would increase the rate of subgrade consolidation, it would also increase the risk of inducing a shear failure within the underlying soft soils.

Preloading Duration: When preloading a compressible subgrade, settlement rates steadily diminish over time, such that the greatest benefits are realized during the initial several weeks or months of a program. Depending on the height and density of the surcharge embankment, we tentatively anticipate that a preloading duration in the range of 6 to 12 weeks would be needed. The actual duration will be defined by the point at which settlement rates become negligible.

Preload Monitoring: Because all settlement estimates inherently involve numerous uncertainties, actual settlements should be monitored in the field throughout the preloading period. We recommend that vertical survey measurements be taken at settlement plates installed near each corner and the center of the embankment. Measurements should be made on a weekly basis, with an accuracy of 0.01 foot, and transmitted to AESI for review.

6.6 Spread Footings

The uncontrolled fill soils underlying the site are not presently suitable for foundation bearing purposes, in our opinion. However, conventional spread footings can be used for supporting the new building if all footings bear on a properly installed array of compacted aggregate piers (as described in Section 6.4) or on an adequately preloaded subgrade (as described in Section 6.5). We offer the following comments and recommendations concerning the design and construction of spread footings.

Footing Depths and Widths: For frost and erosion protection, the bottoms of all exterior footings should bear at least 18 inches below adjacent outside grades, whereas the bottoms of interior footings need bear only 12 inches below the surrounding slab or crawl-space level. To reduce post-construction settlements, continuous (wall) and isolated (column) footings should be at least 18 and 24 inches wide, respectively. It should be noted, however, that greater depths or widths might be needed for other reasons, as determined by the project structural engineer.

Bearing Capacities: Assuming proper installation of compacted aggregate piers or proper preloading of the subgrade, as previously described, we recommend that all footings be designed for the following maximum allowable bearing capacities. These capacities are stated in pounds per square foot (psf), and they incorporate static and seismic safety factors of at least 2.0 and 1.5, respectively.

Static Allowable Bearing Capacity:	2,500 psf
Seismic Allowable Bearing Capacity:	3,300 psf

Footing Setbacks: For stability purposes, footings should not be placed near steep slopes or steps in the bearing soils. We specifically recommend keeping all footings at least 3 feet behind any slopes, and also behind a 0.75H:1V line extending upward from the toe of any retaining walls. Furthermore, utility trenches, footing trenches, and other excavations should not encroach on a 1.0H:1V influence line extending downward from any existing footing. If the new building location requires localized excavations very close to any existing footings, proper underpinning or shoring should be provided. Upon request, we can supply specific underpinning or shoring recommendations for specific situations at the site.

Footing Settlements: We estimate that total post-construction settlements of properly designed footings bearing on properly prepared subgrades will not exceed 1 inch. Differential settlements between new foundation elements over horizontal spans on the order of 50 feet could approach ¾ inch. In all cases, these settlements would be reduced if the actual design bearing pressures are lower than our recommended maximum allowable pressures.

Footing and Stemwall Backfill: To provide erosion protection and lateral load resistance, we recommend that backfill be placed on both sides of the footings and stemwalls after the concrete has cured. Either on-site or imported granular soils can be used for this purpose. All footing and stemwall backfill soil should be compacted to a uniform density of at least 90 percent (based on ASTM D-1557).

Lateral Resistance: Footings and stemwalls that have been properly backfilled as described above will resist lateral loads by means of both passive earth pressure and base friction. We recommend using the following allowable values. These earth pressures are stated in pounds per cubic foot (pcf), and they incorporate static and transient (wind or seismic) safety factors of at least 1.5 and 1.1, respectively. Allowable base friction, which includes a safety factor of 1.5, can be combined with the respective passive pressure to resist static and transient loads.

Static Allowable Passive Pressure:	300 pcf
Transient Allowable Passive Pressure:	400 pcf
Allowable Base Friction Coefficient:	0.35

Subgrade Verification and Construction Monitoring: Footings should never be cast directly atop loose, soft, organic, or frozen soil, slough, debris, uncontrolled fill, or surfaces covered by standing water. We recommend that the condition of all subgrades be verified by an AESI representative before any structural fill is placed and before any concrete is poured. Furthermore, if any aggregate piers are installed below footings, an AESI representative should be allowed to continuously observe and document the installation procedures.

6.7 Slab-On-Grade Floors

Because floor slabs typically carry a light load in comparison to building foundations, they allow more latitude concerning support options. This is especially important for the new building because the site is underlain by a thick deposit of uncontrolled fill soils. We offer the following comments and recommendations for slab-on-grade floors.

Floor Sections: A conventional slab-on-grade floor section typically comprises a reinforced concrete slab over a plastic vapor retarder over an aggregate base and, in many cases, a granular subbase. Assuming that the slab has a conventional thickness on the order of 4 or 5 inches and is subjected to typical loads, we recommend the following underslab layers (top to bottom) and minimum thicknesses.

Vapor Retarder:	10 mils
Base Course:	4 inches
Subbase Course:	12 inches

Subgrade Preparation: All floor areas should be excavated as needed to accommodate the slab and underlying layers, as described above. We recommend that the entire exposed subgrade then be compacted to a firm and unyielding condition using a heavy vibratory roller. Any localized zones of soft, organic, or wet soils observed over the subgrades should be overexcavated and replaced with granular structural fill. .

Subbase Course: A subbase course helps to provide more-uniform slab support over variable subgrade conditions, thereby reducing long-term differential settlements. For this purpose, we recommend using a well-graded sand and gravel, such as "Ballast" per WSDOT 9-03.9(1) or "Gravel Borrow" per WSDOT 9-03.14. Other acceptable options include crushed rock and crushed recycled concrete. In all cases, the subbase should be compacted with a vibratory roller to achieve a uniform density of at least 90 percent (based on ASTM D-1557).

Base Course: A base course serves as both a capillary break layer and a leveling layer for the floor slabs. Ideally, the base course would consist of clean, uniform, well-rounded gravel, such as $^5/_8$-inch or $^7/_8$-inch washed rock. It would also be acceptable to use a washed, angular gravel or crushed rock for this purpose. In all cases, the base course should be lightly compacted to create a firm, smooth surface.

Vapor Retarder: A vapor retarder consists of heavy-duty plastic sheeting that is placed between the base course and floor slab. In our opinion, a vapor retarder provides a significant benefit by reducing the amount of ground moisture that penetrates the floor slab. We recommend that a vapor retarder be installed beneath all floor areas that will be covered by carpet, wood, tile, or any other moisture-sensitive materials. The vapor retarder should be selected on the basis of allowable vapor transmission rates for the planned floor finish materials, and be installed in strict accordance with the manufacturer's guidelines.

Floor Settlements: If the subgrade and underslab layers are properly constructed, we estimate that total post-construction static settlements of a slab-on-grade floor bearing on native soil will not exceed ¾ inch under conventional loading conditions. Differential settlements across the length or width of the floor could approach one-half of the actual total settlement.

Subgrade Verification and Construction Monitoring: Floor slab sections should never be placed atop loose, soft, organic, or frozen soil, slough, debris, or surfaces covered by standing water. We recommend that an AESI representative be allowed to monitor all floor slab construction to verify suitable conditions. Our monitoring services would include probings of subgrade soils, observation of underslab fill layers, and a check of layer thicknesses.

6.8 Drainage Systems

In order to reduce the risk of future moisture problems, the new building should be provided with permanent drainage systems. We offer the following recommendations and comments regarding various drainage elements and related features.

Foundation Drains: We recommend that the new building be encircled with perimeter foundation drains to collect exterior seepage water. Each drain should consist of a

4-inch-diameter, rigid, perforated pipe within an envelope of pea gravel or washed rock, extending at least 6 inches on all sides of the pipe. The gravel envelope should be wrapped with filter fabric (such as Mirafi 140N) to reduce the migration of fines from the surrounding soils. Ideally, the drain invert would be installed no more than 8 inches above or below the base of the perimeter footings or grade beams.

Subfloor Drains: Based on the soil and ground water conditions observed in our site explorations, we currently do not infer a need for drains beneath the floor slabs if the foundation drains are properly installed. However, the final decision regarding the need for subfloor drains should be made at the time of construction, after the floor subgrade has been exposed and the foundation walls have been cast.

Runoff Water: Roof downspouts, parking lot drains, and drains from any other runoff surfaces should not be tied into the perforated piping system of the foundation drains. Instead, the runoff water collected from such sources should be routed through a separate tightline piping system. Also, final site grades should be sloped so that surface water flows away from the building rather than ponding near the foundation walls.

6.9 Backfilled Retaining Walls

We anticipate that backfilled concrete retaining walls might be used to accommodate grade changes in certain exterior site locations. Furthermore, any subsurface vault walls should also be designed as backfilled retaining walls. Our design and construction recommendations for new backfilled retaining walls are presented below.

Wall Foundations: In our opinion, new retaining walls should not be constructed on existing uncontrolled fill soils, due to the risk of excessive settlements. Instead, we recommend that all retaining wall subgrades be overexcavated downward and outward by a minimum distance of 2 feet from the footing. Next, the overexcavated soil should be replaced with imported, well-graded sand and gravel, such as "Ballast" per WSDOT 9-03.9(1) or "Gravel Borrow" per WSDOT 9-03.14, and then be vibratory-compacted to achieve a uniform density of at least 95 percent (based on ASTM D-1557). The allowable static and transient bearing capacities presented in the "Spread Footings" section of this report would apply to retaining wall footings.

Static Lateral Earth Pressures: Yielding (cantilever) walls that are allowed to deflect more than 0.005 times the wall height should be designed to withstand an appropriate static *active* lateral earth pressure. Non-yielding (restrained) walls that are allowed to deflect less than 0.005 times the wall height should be designed to withstand an appropriate static *at-rest* lateral earth pressure. These pressures act over the entire back of the wall and vary with the backslope inclination. For retaining walls with a level or 2H:1V backslope and well-drained conditions, we recommend using the following values, which are given in pcf of equivalent fluid pressure.

Static Active Earth Pressure with Level Backslope:	35 pcf
Static Active Earth Pressure with 2H:1V Backslope:	50 pcf
Static At-Rest Earth Pressure with Level Backslope:	55 pcf
Static At-Rest Earth Pressure with 2H:1V Backslope:	80 pcf

Static Lateral Surcharge Pressures: Any backslope load located within a 0.75H:1V line projected upward from the wall base will apply a lateral surcharge on the wall. Possible sources of surcharge loading include parking lots, traffic lanes, and structure footings. These surcharge pressures act over the portion of wall adjacent to the load source. For distributed vertical loads, active and at-rest static lateral surcharge pressures can be approximated by multiplying the vertical pressure "Q" (in psf) by the appropriate coefficient shown below. We recommend using a vertical pressure of 250 psf to model traffic and parking loads behind the wall.

Static Active Surcharge Pressure:	0.30(Q) psf
Static At-Rest Surcharge Pressure:	0.45(Q) psf

Seismic Lateral Surcharge Pressures: The total static pressures acting on a wall should be increased to account for seismic surcharge loadings resulting from lateral earthquake motions. These surcharge pressures act over the entire back of the wall and vary with the backslope inclination, the seismic acceleration, and the wall height. For retaining walls with a level backslope, active and at-rest seismic lateral surcharge pressures can be approximated by multiplying the wall height "H" (in feet) by the appropriate coefficient shown below.

Seismic Active Surcharge Pressure:	8(H) psf
Seismic At-Rest Surcharge Pressure:	12(H) psf

Curtain Drains: A curtain drain is a vertical layer of drainage material placed against the back of a wall to dissipate hydrostatic pressures. We recommend that a curtain of washed gravel be used behind all walls. This curtain drain should extend outward at least 12 inches from the wall and should extend upward nearly to the ground surface. The backslope directly above this drain should be capped with asphalt or concrete or a layer of low-permeability soil.

Heel Drains: A heel drain is a horizontal drainage element placed behind the rearward projection (heel) of a wall foundation to collect water from the curtain drain. We recommend that a heel drain be included behind the subject wall. The heel drain should comprise a 4-inch-diameter perforated pipe surrounded by at least 6 inches of washed gravel, all wrapped with filter fabric (such as Mirafi 140N). The drainpipe should then be connected to a tightline discharge pipe that routes water to an appropriate location.

Backfill Soil: We recommend that all backfill placed behind the curtain drain consist of granular structural fill. Suitable materials include imported, well-graded sand and gravel, such as "Ballast" per WSDOT 9-03.9(1) or "Gravel Borrow" per WSDOT 9-03.14. Non-organic, sandy portions of the on-site soils would also be suitable for this purpose. If the backfill soil contains more than 10 percent fines, a layer of filter fabric (such as Mirafi 140N) should be placed between the curtain drain and backfill.

Backfill Compaction: Because soil compactors place significant lateral pressures on walls, we recommend that only small, hand-operated compaction equipment be used within 3 feet of a wall. Also, the soil within 3 feet should be compacted to a density as close as possible to 90 percent of the maximum dry density (based on ASTM D-1557). A greater degree of compaction closely behind the wall would increase the lateral earth pressure, whereas a lesser degree of compaction might lead to excessive post-construction settlements. Structural backfill placed more than 3 feet behind the wall should be compacted to a density of at least 95 percent.

Construction Monitoring: We recommend that an AESI representative be allowed to monitor all retaining wall construction. Our monitoring services would include verification of foundation systems, observation of drainage components, and testing of backfill compaction.

6.10 Conventional Pavement Sections

We understand that conventional flexible (asphalt concrete) pavements might be used in the new car parking areas and driveways, whereas rigid (cement concrete) pavement might be used for certain other locations. The following comments and recommendations are given for pavement design and construction purposes.

Soil Design Values: Soil conditions can be defined by a California Bearing Ratio (CBR), which quantitatively predicts the effects of wheel loads imposed on a saturated subgrade. Although our scope of work did not include a CBR test on the surficial site soils, we infer from our observations and limited textural testing that a CBR value on the order of 4 to 6 would likely be appropriate for pavement design purposes.

Traffic Design Values: Traffic conditions can be defined by a Traffic Index (TI), which quantifies the combined effects of projected car and truck traffic. Although no specific traffic data was available at the time of our analysis, we estimate that a TI of 3.0 to 4.0 would likely be appropriate for the car parking areas. A higher TI of about 5.0 to 6.0 appears appropriate for driveways and other areas that are occasionally or periodically subjected to delivery trucks and other heavy vehicles.

Flexible Pavement Sections: A flexible pavement section typically comprises an asphalt concrete pavement (ACP) over a crushed aggregate base (CAB) over a granular subbase (GSB). Our recommended minimum thicknesses for flexible pavement sections, which are based on the aforementioned design values and a 20-year lifespan, are shown below.

Car Parking Lots

Asphalt Concrete Pavement (ACP):	2½ inches
Crushed Aggregate Base Course (CAB):	3 inches
Granular Subbase Course (GSB):	9 inches

Access Driveways

Asphalt Concrete Pavement (ACP):	4 inches
Crushed Aggregate Base Course (CAB):	4 inches
Granular Subbase Course (GSB):	15 inches

Rigid Pavement Sections: A rigid pavement section typically comprises a cement concrete pavement (CCP) over a CAB over a GSB. We recommend the following minimum thicknesses for a rigid pavement section that is subjected to occasional delivery trucks. Pavements and slabs that are subjected to frequent truck traffic or to other heavy structural loads would require a special design.

Access Driveways

Cement Concrete Pavement (CCP):	7 inches
Crushed Aggregate Base Course (CAB):	3 inches
Granular Subbase Course (GSB):	9 inches

Subgrade Preparation: All pavement subgrades should be compacted to a firm and unyielding condition before any pavement layers are placed. We recommend using a heavy vibratory-drum roller for granular (sand and gravel) subgrades, and a heavy static-drum roller for cohesive (silt and clay) subgrades. The resulting subgrade condition should then be verified by proof-rolling with a loaded dump truck or other heavy construction vehicle, in the presence of an AESI representative. Any localized zones of soft, organic-rich, or debris-laden soils disclosed during the proof-rolling test should be overexcavated and replaced with compacted structural fill.

Granular Subbase: A subbase course helps to provide more-uniform structural support for a pavement section bearing on uncontrolled fill. For the subbase course, we recommend using an imported, well-graded sand and gravel, such as "Ballast" per WSDOT 9-03.9(1) or "Gravel Borrow" per WSDOT 9-03.14. It would also be acceptable to use a crushed recycled concrete, provided that it meets the same general textural criteria as the aforementioned WSDOT materials. In all cases, the subbase should be vibratory-compacted to achieve a uniform density of at least 95 percent (based on ASTM D-1557).

Crushed Aggregate Base: We recommend that all CAB material conform to the criteria for "Crushed Surfacing Base Course" per WSDOT 9-03.9(3). In the interest of using recycled materials from on-site or off-site sources, it would be acceptable to substitute up to 20 percent of the CAB with crushed cement concrete, provided that the final mixture meets the same grain-size criteria as the aforementioned WSDOT material. Regardless of composition, all CAB material should be compacted to a minimum density of 95 percent based on the modified Proctor maximum dry density (per ASTM D-1557).

Asphalt Concrete Pavement: We recommend that the ACP aggregate gradation conform to the control points for a ½-inch mix (per WSDOT 9-03.8(6)) and that the binder conform to Performance Grade 58-22 criteria (per WSDOT 9-02.1(4)). We also recommend that the ACP be compacted to a target average density of 92 percent, with no individual locations compacted to less than 90 percent nor more than 96 percent, based on the Rice theoretical maximum density for that material (per ASTM D-2041).

Cement Concrete Pavement: We recommend that the CCP consist of Portland cement concrete with a minimum compressive strength of 4,000 pounds per square inch (psi) and a minimum rupture modulus of 500. We also recommend that the concrete be reinforced with a welded wire mesh, such as W2-6x6, positioned at a one-third depth within the CCP layer.

Pavement Life and Maintenance: It should be realized that conventional asphaltic pavements are not maintenance-free. The foregoing pavement sections represent our minimum recommendations for an average level of performance during a 20-year design life; therefore, an average level of maintenance will likely be required. Furthermore, a 20-year pavement life typically assumes that an overlay will be placed after about 10 years. Thicker asphalt, base, and subbase courses would offer better long-term performance, but would cost more initially; thinner courses would be more susceptible to "alligator" cracking and other failure modes. As such, pavement design can be considered a compromise between a high initial cost and low maintenance costs versus a low initial cost and higher maintenance costs.

6.11 Permeable Pavement Sections

We anticipate that permeable pavements might be used in certain new parking areas and/or driveways. These permeable pavements could include some combination of flexible (asphalt concrete) and/or rigid (cement concrete) surfacing. Our geotechnical comments and recommendations concerning permeable pavements are presented in the following paragraphs.

Design Values: For design of permeable flexible and rigid pavement sections, we have assumed the same soil and traffic design values discussed in the “Conventional Pavement Sections” portion of this report. It should be noted that driveways are assumed to be subjected to delivery trucks and similar vehicles.

Permeable Pavement Layers: A permeable pavement section typically comprises a porous asphalt concrete pavement (PACP) or pervious cement concrete pavement (PCCP) over an aggregate drainage base (ADB). In some situations, an aggregate choker base (ACC) is placed between the pavement and base courses, but we regard this as an optional item to be used at the discretion of the civil engineer or paving contractor. Our recommended minimum layer thicknesses for various on-site uses are shown below.

Car Parking Lots - Flexible Section	
Porous Asphalt Concrete Pavement (PACP):	3 inches
Aggregate Drainage Base (ADB):	12 inches
Access Driveways - Flexible Section	
Porous Asphalt Concrete Pavement (PACP):	5 inches
Aggregate Drainage Base (ADB):	16 inches
Access Driveways - Rigid Section	
Pervious Cement Concrete Pavement (PCCP):	7 inches
Aggregate Drainage Base (ADB):	12 inches

Subgrade Preparation: All pervious pavement subgrades should be lightly compacted to achieve a firm condition. Excessive compaction should be avoided because it can reduce the infiltration characteristics of the subgrade soils. After compaction, the surface should be hand-raked or gently scarified to eliminate any "soil skin" that has formed.

Filter Fabric: If the subgrade consists of fine-grained (silt or clay) soils, we recommend that a layer of non-woven filter fabric (such as Mirafi 140N) be placed between the prepared subgrade and the ADB layer. This fabric will help prevent migration of native soils into the ADB gravel.

Aggregate Drainage Base: The ADB serves as both a reservoir and discharge layer for stormwater. As such, the actual thickness might need to be increased beyond our above-stated minimum value if greater storage capacity is required. Regardless of thickness, we recommend using an imported, uniform, coarse, angular gravel meeting the specifications of "Permeable Ballast" per WSDOT 9-03.9(2) or "No. 3 Stone" per ASTM C-33. The ADB material should be lightly compacted to achieve a reasonably firm and stable condition, but excessive compaction should be avoided.

Aggregate Choker Course: Because the ADB consists of a moderately large-grained material, some contractors prefer to cover it with a choker course to serve as a leveling layer. Where a choker course is desired, we recommend using 2 inches of imported, uniform, medium-grained, angular gravel meeting the specifications of "No. 57 Stone" per ASTM C-33. The choker course should be lightly compacted to achieve a reasonably firm, smooth, and stable condition, but excessive compaction should be avoided.

Porous Asphalt Concrete Pavement: We recommend that PACP use an aggregate consisting of uniform, small- to medium-grained, crushed gravel meeting the specifications of "No. 8 Stone" per ASTM C-33. The binder should conform to PG 70-22 criteria and should be placed at a ratio of 5.75 to 6.00 percent by weight. We also recommend that the PACP be compacted to a firm condition by means of approximately two passes with a heavy vibratory roller. Excessive compaction should be avoided. Ultimately, the finished PACP should provide a minimum infiltration rate of 200 inches per hour (in/hr).

Pervious Cement Concrete Pavement: We recommend that PCCP use an aggregate consisting of uniform, small- to medium-grained, crushed gravel meeting the specifications of "No. 8 Stone" per ASTM C-33. Typically, the concrete paste is a six-sack mix with a water/cement ratio in the range of 0.27 to 0.35. Ultimately, the finished PCCP should provide a minimum compressive strength of 2,000 psi, and a minimum infiltration rate of 200 in/hr.

Pavement Life and Maintenance: It should be realized that all permeable pavements require routine maintenance to maintain their permeability. The entire surface should be vacuum-swept at least once per year under normal conditions; if the pavement is exposed to dirt, excessive traffic, or turbid water, then vacuum-sweeping should be performed more frequently. In addition, routine structural maintenance, such as patching, will likely be required over the 20-year pavement life.

6.12 Structural Fill

The term *structural fill* refers to any materials placed under foundations, retaining walls, slab-on-grade floors, sidewalks, pavements, and other such features. Our comments, conclusions, and recommendations concerning structural fill are presented in the following paragraphs.

Soil Moisture Considerations: The suitability of soils used for structural fill depends primarily on their grain-size distribution and moisture content when they are placed. As the fines content (that soil fraction passing the U.S. No. 200 Sieve) increases, soils become more sensitive to small changes in moisture content. Soils containing more than about 5 percent fines (by weight) cannot be consistently compacted to a firm, unyielding condition when the moisture content is more than 2 percentage points above or below optimum.

Structural Fill Materials: For general use, a well-graded mixture of sand and gravel with a low fines content (commonly called "gravel borrow" or "pit-run") provides an economical structural fill material. For specialized applications, it may be necessary to use a highly processed material such as crushed rock, quarry spalls, clean sand, granulithic gravel, pea gravel, drain rock, CDF, or LMC. Recycled asphalt or concrete, which are derived from pulverizing the parent materials, are also potentially useful as structural fill in certain applications. Soils used for structural fill should not contain any organic matter, debris, environmental contaminants, or individual particles greater than about 6 inches in diameter.

On-Site Soils: Because only minor grading appears necessary at the site, it is expected that relatively small quantities of on-site native soils will be generated during earthwork activities. These on-site soils will likely consist of silty sands, sandy silts, or silts (uncontrolled fill). We anticipate that some of these non-organic soils can be reused as structural fill during the summer months. However, they will be difficult or impossible to reuse during the wet season or during isolated periods of rainy weather. Furthermore, any organic-rich native or fill soils would not be suitable for reuse as structural fill at any time of year.

Fill Placement and Compaction: Structural fill materials should be placed in horizontal lifts not exceeding about 12 inches in loose thickness. Unless stated otherwise in this report, we recommend that each lift then be thoroughly compacted with a mechanical compactor to a uniform density of at least 95 percent, based on the modified Proctor test (per ASTM D-1557). Compaction is not necessary for certain structural fill materials, such as pea gravel, drain rock, quarry spalls, CDF, and LMC.

Subgrade Verification and Compaction Testing: Regardless of material or location, all structural fill should be placed over firm, unyielding subgrades prepared in accordance with our various recommendations for site preparation. The condition of all subgrades should be verified by an AESI representative before soil or concrete placement begins. Also, fill soil compaction should be verified by means of in-place density testing, hand-probing, proof-rolling, or other appropriate methods performed during fill placement so that the adequacy of soil compaction efforts may be evaluated as earthwork progresses.

7.0 CLOSURE

AESI has prepared this report for the exclusive use of our client and their agents, for specific application to this project. Within the limitations of scope and schedule, our services have been performed in accordance with generally accepted local geotechnical engineering practices in effect at the time our report was prepared. No other warranty, express or implied, is made.

We appreciate the opportunity to be of continued service to you on this project. Should you have any questions regarding this report or other geotechnical aspects of the project, please call us at your earliest convenience.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Tacoma, Washington

Matthew A. Miller, P.E.
Principal Geotechnical Engineer

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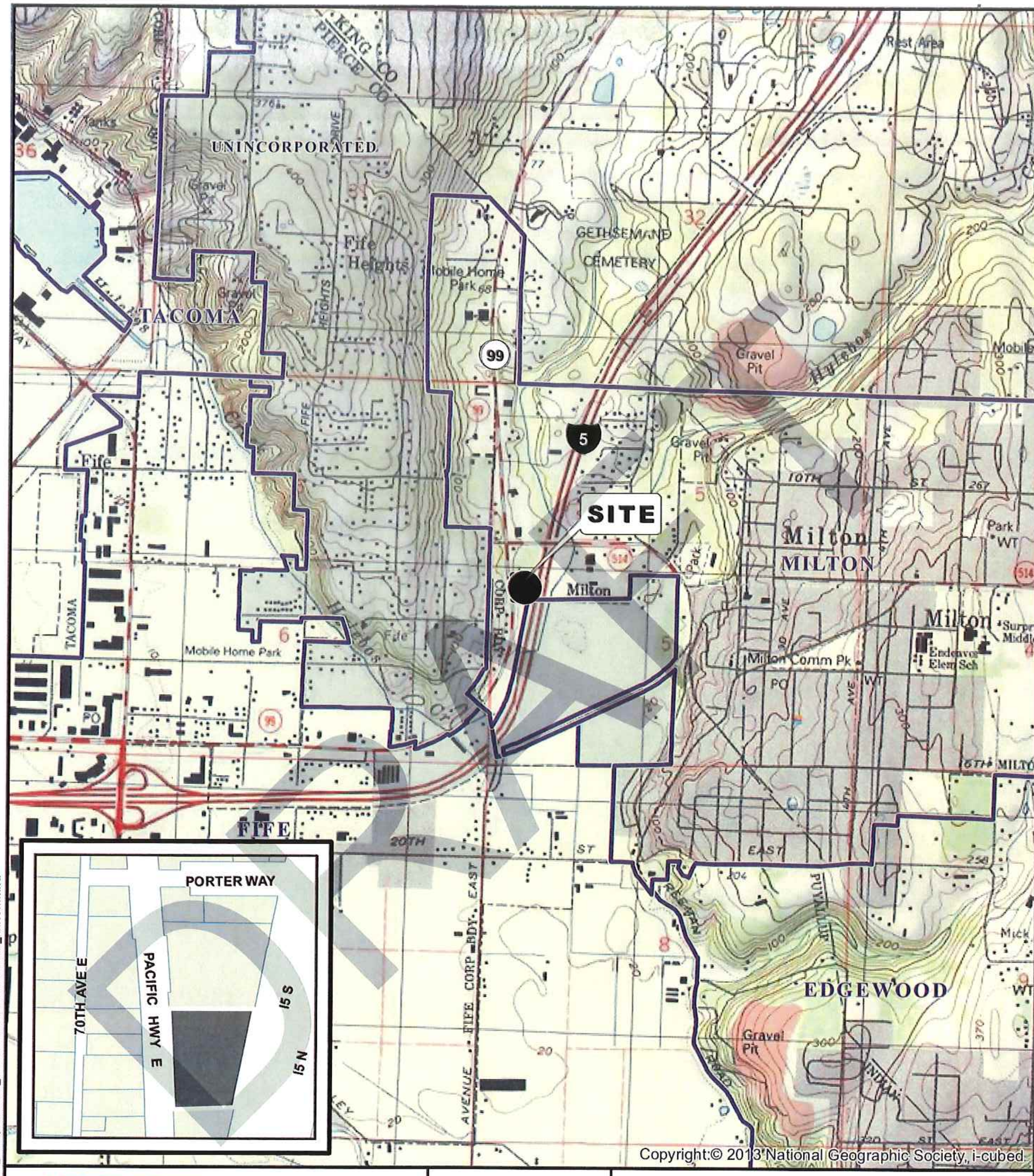
Senior Associate Geotechnical Engineer



DATA SOURCES / REFERENCES:
USGS: 24K SERIES TOPOGRAPHIC MAPS
PIERCE CO: STREETS, CITY LIMITS, PARCELS 02/17

LOCATIONS AND DISTANCES SHOWN ARE APPROXIMATE

N

0 1000 2000
FEET

NOTE: BLACK AND WHITE REPRODUCTION OF THIS COLOR ORIGINAL MAY REDUCE ITS EFFECTIVENESS AND LEAD TO INCORRECT INTERPRETATION

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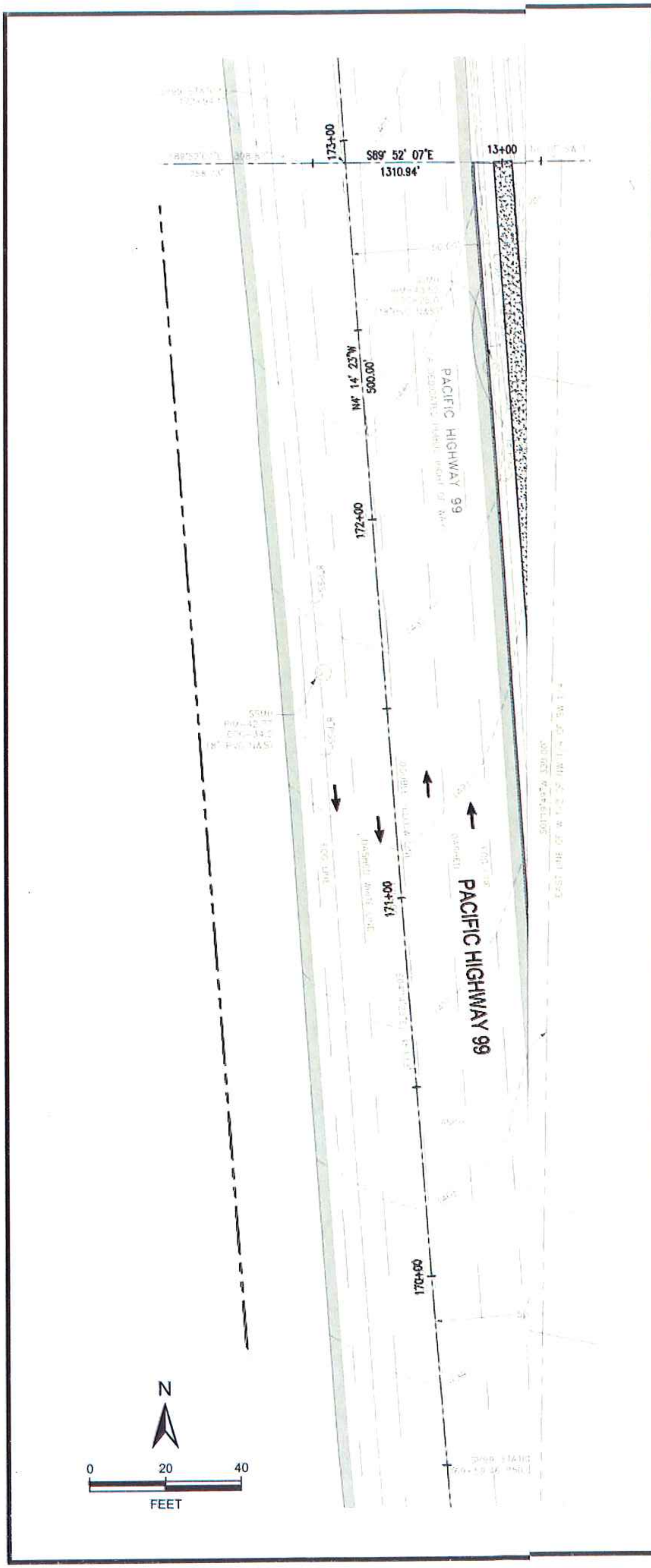
VICINITY MAP

MILTON TELECARE FACILITY
MILTON, WASHINGTON

PROJ NO.	DATE:	FIGURE:
170289E001	6/17	1

Document Path: G:\GIS_Projects\aTemplates\aVM_Template\VM_MXD\170289E001 F1 VM_Milton.mxd



LEGEND:

EB — EXPLORATION BORING - WITH DEPTH OF OBSERVED THICKNESS OF UNCONTROLLED FILL SOIL IN FEET

SITE BOUNDARY

PROPOSED RETAINING WALL

CONTOUR INTERVAL = 1'

NOTE: LOCATION AND DISTANCES SHOWN ARE APPROXIMATE.

NOTES:
1. BASE MAP REFERENCE: BCRA PIERCE COUNTY SITE AND SURFACING, C0.01

BLACK AND WHITE REPRODUCTION OF THIS COLOR ORIGINAL MAY REDUCE ITS EFFECTIVENESS AND LEAD TO INCORRECT INTERPRETATION.

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SITE AND EXPLORATION PLAN

MILTON TELECARE FACILITY
MILTON, WASHINGTON

PROJ NO.	DATE:	FIGURE:
170289E001	6/17	2

170289 Telecare \ 170289 F2 S-E Plan.cdr

APPENDIX A

Exploration Logs

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Exploration Log

Project Number	Exploration Number	Sheet
170289E001	EB-1	1 of 2

Project Name	Milton Telecare Facility	Ground Surface Elevation (ft)	
Location	Milton, WA	Datum	N/A
Driller/Equipment	Holocene Drilling Inc. / D-50 HSA	Date Start/Finish	6/6/17,6/6/17
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	8 inches

Depth (ft)	ST	Samples	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot	Other Tests
			Grass / Uncontrolled Fill					
	Grab	S-1	Slightly moist, brown, very silty, fine SAND, some medium to coarse sand, some gravel; unsorted (SM).					
		S-2	Medium dense, dry to slightly moist, gray-brown, very silty, fine SAND, some medium to coarse sand, trace gravel; unsorted (SM).			8 16 9	25	
5		S-3	No recovery; pounding on coarse gravel stuck in sampler tip. Blow counts possibly overstated. Cuttings generally as above.			9 13 18	31	
10		S-4	Stiff, slightly moist to moist, bluish gray, SILT, some sand, some gravel, trace organics; unsorted (ML).			3 5 6	11	
15		S-5	Loose to medium dense, moist, dark gray-brown, silty, fine to coarse SAND, some fine gravel, trace coarse gravel, trace organics; unsorted (SM).			5 6 5	11	
			Vashon Recessional Lacustrine Deposits					
			Driller notes smoother drilling action and little to no gravels.					
20		S-6	Medium stiff to stiff, slightly moist, bluish gray, CLAY; abundant organics and rootlets (CL/CH).			3 3 4	7	
25		S-7	Stiff, slightly moist, gray with brown mottling, SILT / CLAY; scattered thin seams (<1/16 inch thick) very fine to fine sand; fine laminae (ML/CL).			4 6 7	13	

Sampler Type (ST):

- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- No Recovery
- Ring Sample
- Shelby Tube Sample
- M - Moisture
- ▽ Water Level ()
- ▼ Water Level at time of drilling (ATD)

Logged by: LBK
Approved by: JDD

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Exploration Log

Project Number	Exploration Number	Sheet
170289E001	EB-1	2 of 2

Project Name Milton Telecare Facility
Location Milton, WA
Driller/Equipment Holocene Drilling Inc. / D-50 HSA
Hammer Weight/Drop 140# / 30"

Ground Surface Elevation (ft)
Datum N/A
Date Start/Finish 6/6/17,6/6/17
Hole Diameter (in) 8 inches

Depth (ft)	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
		S-8		Medium stiff, moist, gray with brown and orange mottling (iron-oxide), SILT / CLAY; thin (1/16 to 1/8 inch thick) very fine to fine sand seams; fine laminae (ML/CL).			2 3 3	▲6	
				Vashon Recessional Outwash Driller notes density increases and notes ground water.		▼			
35		S-9		Dense, wet, grayish brown, fine to coarse SAND, some fine gravel, trace silt; massive; poor recovery (SW).			16 17 29	▲46	
40		S-10		Medium dense, wet, grayish brown, fine to coarse SAND, trace fine gravel; weakly stratified (SW).			3 11 8	▲19	
				Very stiff, slightly moist, gray, SILT; massive (ML).					
				Bottom of exploration boring at 41.5 feet Ground water encountered at ~32 feet. Backfilled with bentonite chips.					
45									
50									
55									

Sampler Type (ST):

- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- No Recovery
- Ring Sample
- Shelby Tube Sample
- M - Moisture
- ▽ Water Level ()
- ▼ Water Level at time of drilling (ATD)

Logged by: LBK
Approved by: JDD

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Exploration Log

Project Number	Exploration Number	Sheet
170289E001	EB-2	1 of 2

Project Name	Milton Telecare Facility	Ground Surface Elevation (ft)	
Location	Milton, WA	Datum	N/A
Driller/Equipment	Holocene Drilling Inc. / D-50 HSA	Date Start/Finish	6/6/17,6/6/17
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	8 inches

Depth (ft)	ST	Samples	DESCRIPTION	Water Level	Blows/6"	Blows/Foot
			Grass / Uncontrolled Fill			
	Grab	S-1	Moist, brown, fine sandy SILT, trace gravel; unsorted (ML).			
		S-2	Loose to medium dense, moist, gray, silty, fine SAND, some gravel; unsorted (SM). Loose to medium dense, moist, brown, very fine to fine SAND, some silt; massive (SP). Farallon (Project Enviromental Consultant) collected evironmental sample.		3 6 5	11
5		S-3	Loose, moist, gray, very silty, fine SAND, some fine to coarse gravel; unsorted (SM).		4 3 4	7
10		S-4	No recovery. Cuttings generally as above. Becomes loose to medium dense.		0 1 10	11
			Vashon Recessional Lacustrine Deposits			
15		S-5	Stiff, very moist, gray with some brown mottling (iron-oxide), very fine to fine SAND, some silt, interbeds (1/4 to 1 inch thick) very fine sandy silt (SP/ML).		4 5 4	9
20		S-6	Medium dense, wet, brown with orange mottling (iron-oxide), silty, very fine SAND, interbeds (1/4 to 1/2 inch thick) SILT; faint fine laminae (SM/ML). Driller notes ground water at ~20 feet.	ATD	4 7 9	16
25		S-7	Dense, wet, brown, very fine SAND, some silt; faint fine laminae (SP). Hard, moist, brown with dark brown-orange mottling (organics and iron-oxide), very fine sandy SILT; laminated (ML).		9 16 27	43

Other column headings: Graphic Symbol, Well Completion, Other Tests. Blows/Foot scale: 10 20 30 40

DRAFT

Sampler Type (ST):
- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- No Recovery
- Ring Sample
- Shelby Tube Sample
- M - Moisture
- Water Level ()
- Water Level at time of drilling (ATD)

Logged by: LBK
Approved by: JDD

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Exploration Log

Project Number	Exploration Number	Sheet
170289E001	EB-2	2 of 2

Project Name	**Milton Telecare Facility**	Ground Surface Elevation (ft)	
Location	**Milton, WA**	Datum	N/A
Driller/Equipment	Holocene Drilling Inc. / D-50 HSA	Date Start/Finish	6/6/17,6/6/17
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	8 inches

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
		S-8		Very stiff, moist, brown with dark brown and orange mottling (organics and iron-oxide), very fine sandy SILT; laminated (ML).			4 11 14	▲25	
				Pre-Fraser Deposits Very stiff, slightly moist, bluish gray, SILT, with very thin (<1/16 inch thick) very fine sand laminae (ML).					
35		S-9		Very stiff, slightly moist, bluish gray with faint brown and orange mottling (iron-oxide), SILT, some very fine sand; fine laminae (ML). Becomes very silty, very fine SAND; micaceous.			6 9 13	▲22	
				Bottom of exploration boring at 36.5 feet Ground water encountered at ~20 feet. Backfilled with bentonite chips.					
40									
45									
50									
55									

Sampler Type (ST):

- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- No Recovery
- Ring Sample
- Shelby Tube Sample
- M - Moisture
- Water Level ()
- Water Level at time of drilling (ATD)

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Approved by: JDD

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Exploration Log

Project Number	Exploration Number	Sheet
170289E001	EB-3	1 of 2

Project Name	Milton Telecare Facility	Ground Surface Elevation (ft)	
Location	Milton, WA	Datum	N/A
Driller/Equipment	Holocene Drilling Inc. / D-50 HSA	Date Start/Finish	6/6/17,6/6/17
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	8 inches

Depth (ft)	ST	Samples	DESCRIPTION	Water Level	Blows/6"	Blows/Foot
			Grass / Uncontrolled Fill			
	Grab	S-1	Slightly moist, brown, very silty, fine to medium SAND, some fine to coarse gravel; unsorted (SM).			
	SPT	S-2	Loose to medium dense, slightly moist, brown, very silty, fine to medium SAND, some fine to coarse gravel, some organics; unsorted (SM).		2 4 6	10
5	SPT	S-3	Stiff, slightly moist, gray with brown mottling, SILT, some sand, some gravel, some organics and rootlets; unsorted; poor recovery (ML).		3 6 7	13
10	SPT	S-4	Loose, moist, dark brown to dark gray, very silty, fine to medium SAND, some fine gravel; unsorted (SM). Farallon (Project Enviromental Consultant) collected evironmental sample.		2 3 5	8
15	SPT	S-5	Loose, very moist, dark brown, very silty, fine to medium SAND, trace gravel; some wood fragments; unsorted (SM). Poor recovery.		2 2 2	4
20		S-6	No recovery.		6 8 7	15
			Vashon Recessional Lacustrine Deposits			
25	SPT	S-7	Stiff, moist, brown with orange mottling (iron-oxide), SILT; faint laminae (ML). Medium dense, wet, grayish brown with orange mottling (iron-oxide), very fine SAND; laminated (SP). Water on rods at ~27 feet.	▼ (~27 ft)	3 5 8	13
			Vashon Advance Outwash			

Sampler Type (ST):

- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- No Recovery
- Ring Sample
- Shelby Tube Sample
- M - Moisture
- ▽ Water Level ()
- ▼ Water Level at time of drilling (ATD)

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Exploration Log

Project Number	Exploration Number	Sheet
170289E001	EB-3	2 of 2

Project Name: Milton Telecare Facility
Location: Milton, WA
Driller/Equipment: Holocene Drilling Inc. / D-50 HSA
Hammer Weight/Drop: 140# / 30"

Ground Surface Elevation (ft):
Datum: N/A
Date Start/Finish: 6/6/17,6/6/17
Hole Diameter (in): 8 inches

Depth (ft)	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot	Other Tests
		S-8		Medium dense to dense, wet, silty, very fine SAND with thin (1/16 to 1/8 inch thick) SILT seams; fine laminae (SM/ML).			26 16 13	29	
35		S-9		Very dense, wet, brown, very fine SAND, trace silt; massive (SP).			21 27 38	65	
				Bottom of exploration boring at 36.5 feet Ground water encountered at ~27 feet. Backfilled with bentonite chips.					
40									
45									
50									
55									

Sampler Type (ST):
- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- No Recovery
- Ring Sample
- Shelby Tube Sample
- M - Moisture
- Water Level ()
- Water Level at time of drilling (ATD)

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Exploration Log

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Project Number	Exploration Number	Sheet
170289E001	EB-4	1 of 2

Project Name Milton Telecare Facility
Location Milton, WA
Driller/Equipment Holocene Drilling Inc. / D-50 HSA
Hammer Weight/Drop 140# / 30"

Ground Surface Elevation (ft)
Datum N/A
Date Start/Finish 6/6/17,6/6/17
Hole Diameter (in) 8 inches

Depth (ft)	ST	Samples	DESCRIPTION	Blows/6"	Blows/Foot
			Grass / Uncontrolled Fill		
	Grab	S-1	Dry, brown, silty, fine to medium SAND, some gravel; unsorted (SM).		
		S-2	Medium dense, dry to slightly moist, brown with faint orange mottling (iron-oxide), very silty, fine to medium SAND, some gravel; unsorted (SM).	4 9 7	16
5		S-3	Loose, moist, brown, very silty, fine SAND, some medium to coarse sand, trace gravel; unsorted (SM). Poor recovery.	3 4 3	7
10		S-4	No recovery.	2 3 3	6
15		S-5	Medium dense, moist, dark gray-brown, silty, fine to medium SAND, some gravel; unsorted (SM).	2 5 13	18
20			**Vashon Recessional Lacustrine Deposits**		
		S-6	Stiff, slightly moist, bluish gray with orange mottling (iron-oxide), SILT; massive (ML).	3 5 7	12
25		S-7	Medium stiff to stiff, slightly moist, brown with gray mottling, CLAY; massive (CL).	3 4 4	8

Sampler Type (ST):

- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- No Recovery
- Ring Sample
- Shelby Tube Sample
- M - Moisture
- ▽ Water Level ()
- ▼ Water Level at time of drilling (ATD)

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Exploration Log

Project Number	Exploration Number	Sheet
170289E001	EB-4	2 of 2

Project Name Milton Telecare Facility
Location Milton, WA
Driller/Equipment Holocene Drilling Inc. / D-50 HSA
Hammer Weight/Drop 140# / 30"

Ground Surface Elevation (ft)
Datum N/A
Date Start/Finish 6/6/17,6/6/17
Hole Diameter (in) 8 inches

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
		S-8		Stiff to very stiff, slightly moist, brown with gray mottling, SILT / CLAY with scattered very thin (<1/16 inch thick) very fine sand seams; micaceous (ML/CL).			3 5 10	▲15	
				Medium dense, very moist, dark brown, very fine SAND, trace organics; fine laminae (SP).					
				Vashon Advance Outwash / Pre-Fraser Deposits					
35		S-9		Very dense, wet, gray-brown, fine to coarse SAND, some fine to coarse gravel, some silt, grading to gravelly, medium to coarse SAND, some silt; stratified (SP/SW).			12 25 35	▲60	
				Very dense, wet, gray-brown with orange mottling (iron-oxide), silty, gravelly, fine to coarse SAND; weakly stratified (SM).					
40		S-10		Dense to very dense, wet, brown, fine SAND, trace silt grading to gravelly, fine to coarse SAND, trace silt; well sorted (SP).			16 20 28	▲48	
				Hard / dense, moist, gray, SILT grading to very fine SAND; massive (ML/SP).					
				Bottom of exploration boring at 41.5 feet Ground water encountered at ~30 feet. Backfilled with bentonite chips.					
45									
50									
55									

Sampler Type (ST):
- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- No Recovery
- Ring Sample
- Shelby Tube Sample

M - Moisture
∇ Water Level ()
▼ Water Level at time of drilling (ATD)

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Exploration Log

Project Number	Exploration Number	Sheet
170289E001	EB-5	1 of 1

Project Name	Milton Telecare Facility	Ground Surface Elevation (ft)	
Location	Milton, WA	Datum	N/A
Driller/Equipment	Holocene Drilling Inc. / D-50 HSA	Date Start/Finish	6/6/17,6/6/17
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	8 inches

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
				Grass / Uncontrolled Fill					
	Grab	S-1		Slightly moist, brown, very silty, fine SAND, some fine to coarse gravel; unsorted (SM).					
		S-2		Medium dense, slightly moist, gray to brown, very silty, fine SAND, some fine to coarse gravel; unsorted (SM).			4 9 12	▲21	
5		S-3		Loose, dry to slightly moist, brown, very silty, fine to medium SAND, some fine to coarse gravel; unsorted (SM). Poor recovery due to coarse gravel in sampler tip.			2 3 2	▲5	
10		S-4		Very poor recovery; only in sampler tip. Cuttings generally as above. Blow counts possibly overstated due to gravel in sampler tip.			16 26 19	▲45	
				Bottom of exploration boring at 11.5 feet No ground water encountered. Backfilled with bentonite chips.					
15									
20									
25									

DRAFT

Sampler Type (ST):

- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- No Recovery
- Ring Sample
- Shelby Tube Sample
- M - Moisture
- ▽ Water Level ()
- ▼ Water Level at time of drilling (ATD)

Logged by: LBK
Approved by: JDD