Geotechnical Engineering Services Report

Tukwila School District – Thorndyke Elementary
School Improvements
Tukwila, Washington

for
Tukwila School District, No. 406

August 20, 2018

GeoEngineers
Earth Science + Technology

Geotechnical Engineering Services Report

Tukwila School District – Thorndyke Elementary
School Improvements
Tukwila, Washington

for
Tukwila School District, No. 406

August 20, 2018

GeoEngineers

1101 South Fawcett Avenue, Suite 200
Tacoma, Washington 98402
253.383.4940

Geotechnical Engineering Services Report

Tukwila School District – Thorndyke Elementary School Improvements Tukwila, Washington

File No. 23537-001-00

August 20, 2018

Prepared for:

Tukwila School District, No. 406
c/o KMB Architects
906 Columbia Street SW, Suite 400
Olympia, Washington 98501

Attention: Jeffrey Feeney, PE

Prepared by:

GeoEngineers, Inc.
1101 South Fawcett Avenue, Suite 200
Tacoma, Washington 98402
253.383.4940

CHRISTOPHER ROBERT NEWTON
STATE OF WASHINGTON
54300
REGISTERED
PROFESSIONAL ENGINEER

8/20/18

Christopher R. Newton, PE
Staff Geotechnical Engineer

MORGAN MCARTHUR
STATE OF WASHINGTON
46390
REGISTERED
PROFESSIONAL ENGINEER

8/20/18

Morgan McArthur, PE
Associate Geological Engineer

CRN:MM:tt

Table of Contents

LIST OF FIGURES

APPENDICES

INTRODUCTION AND PROJECT UNDERSTANDING

This report presents the results of our geotechnical engineering services for the proposed Thorndyke Elementary School Improvements project. The project site is located at 4415 South 150th Street in Tukwila, Washington as shown on the Vicinity Map, Figure 1. Our services have been completed in general accordance with our signed agreement dated July 24, 2018.

Our project understanding is based on a meeting with KMB Architects (project manager) and Rolluda Architects (project architect) on July 12, 2018 and a preliminary site plan provided during the meeting.

We understand that two new modular classrooms are proposed. Multiple locations are currently under consideration, including one near the northeast corner of site on the blacktop and one adjacent to the east side of the existing school building. We assume that foundations for the modular building(s) will consist of slab-on-grade with thickened edges or shallow spread footings with stem walls.

New parking lot areas and driveways are also planned. The overflow parking lot on the west side of campus is proposed to extend westward into the grass area toward the west perimeter of the property. The bus loop and campus entrance driveways will also be improved and/or reconfigured.

Other improvements include an upgraded playground structure and soccer field addition located near the southeast corner of the site. We understand that drainage improvements are planned for the area of the proposed soccer field.

We anticipate that stormwater infiltration or detention facilities will be included in site improvements. If planned, we assume stormwater infiltration and/or detention facilities will be designed and constructed in accordance with the 2016 King County Surface Water Design Manual (SWDM). We have currently assumed that potential infiltration and/or detention facilities may be located within the overflow parking lot addition on the west side of the site or in the grass area located in the southwest corner of the campus.

SCOPE OF SERVICES

The purpose of our services is to explore subsurface conditions to form a basis for developing geotechnical design and construction recommendations for the proposed improvements. Our specific scope of services included the following tasks:

1. Reviewing readily available published geologic data and our relevant in-house files for existing information on subsurface conditions in the project vicinity.
2. Visiting the project site to mark out exploration locations and contact the “One-Call” Utility Notification Center, as required by Washington State law. We also subcontracted a private utility locator.
3. Exploring subsurface conditions within the project area by advancing five test pits using subcontracted rubber-tire backhoe equipment and operator. The test pits were excavated to depths between about 8 and 11 feet below ground surface (bgs).
4. Conducting two small-scale pilot infiltration tests (PIT) near or within areas of proposed improvements.
5. Conducting geotechnical laboratory testing on selected soil samples.

6. Providing geotechnical seismic design information in accordance with 2015 International Building Code (IBC) criteria and discuss our opinion on the potential for surface rupture, liquefaction and lateral spreading at the site. We did not complete a quantitative liquefaction and lateral spreading analysis for this study.
7. Providing recommendations for site preparation and earthwork. We discuss temporary erosion and sedimentation controls, temporary and permanent cut slopes, fill placement and compaction requirements, wet weather considerations, groundwater handling and site drainage.
8. Providing recommendations for shallow spread footing design, including foundation bearing surface preparation, allowable soil bearing pressure, lateral resistance values and estimates of settlement.
9. Providing design considerations for slab-on-grade design, including subgrade preparation, modulus of subgrade reaction and capillary break thickness and materials.
10. Providing recommended active, passive and at-rest lateral earth pressures for retaining walls. We also provide recommendations for seismic surcharge pressures and drainage criteria.
11. Summarizing the results of our PITs and provide recommended long-term design infiltration rates for the tested locations. We also include a summary of the testing procedure and data collected. We also discuss our opinion for the need of a groundwater mounding analysis based on our observations of subsurface conditions.
12. Providing layer thickness recommendations for asphalt concrete pavement (ACP) and pervious pavement design sections, including subgrade preparation. We include typical pavement sections for heavy and light traffic areas based on our experience.

SITE CONDITIONS

Surface Conditions

The site is bounded by South 150th Street to the north and to the south by a slope with undeveloped, forested land that grades downward to the south. Residential properties bound the campus to the west and the east side is bounded by residential properties and undeveloped, forested land.

The existing school building is located in the central part of the campus. Other existing development features include asphalt paved driveways, parking lots and blacktop areas, sidewalks, landscaping, playground areas and grass fields.

Site topography is generally flat across the site with elevation differences up to about 3 to 4 feet. An asphalt paved access driveway running along the south side of the school building gently slopes downward from the southwest corner of the school building toward the southeast corner. The site also gently grades downward toward the southeast corner of the campus where the grass soccer field is located.

During our explorations, we observed a series of cracks along the southern edge of the asphalt paved access driveway running along south side of the school building next to the existing chain-link fence. The cracks were located just west of the playground and grass soccer field located in the southeast corner of the campus. The cracks were generally oriented parallel with the direction of the driveway and the cracks were less than about ½ to 1 inch in width. We observed a steep slope just on the other side of the chain-link fence in this area. The cracks could be a sign of minor slope movement or slope instability. A slope reconnaissance was outside the scope of this project, and we understand improvements are not planned in this area. However, we are available to provide assistance if the school district is interested in an evaluation of this steep slope area.

Literature Review

The geologic information we reviewed in the project vicinity includes the *Geologic Map of the Des Moines 7.5' Quadrangle, King County, Washington* (Booth and Waldron 2004). Glacial soil deposits underlie the site and surrounding areas. These deposits are the result of glaciations that occurred during the Vashon Stade of the Fraser Glaciation, approximately 10,000 to 15,000 years ago. Surface soils at the site are primarily mapped as glacial recessional lacustrine deposits (Q_{vrl}). During ice recession, the recessional lacustrine deposits were deposited in small glacial lakes and are described to consist of fine sand, silt and clay. Recessional glacial deposits have not been glacially overridden and are, therefore, typically less dense than other glacial deposits, such as glacial till and advance outwash. Also mapped within the project vicinity is glacial till (Q_{vt}). Glacial till is described as a dense, compact mixture of sand, silt and gravel deposited by a glacier.

Subsurface Conditions

Subsurface Explorations and Laboratory Testing

We explored subsurface conditions at the site by excavating five test pits (TP-1 [PIT-1] through TP-5) at the approximate locations shown on the attached Site Plan, Figure 2. A description of our subsurface exploration program and summary exploration logs are provided in Appendix A. Two small-scale PITs were completed in test pits TP-1 (PIT-1) and TP-3 (PIT-2). The test results and methodology for the PITs are discussed in further detail in the “Stormwater Infiltration” section of this report.

Selected samples collected from our test pits were tested in our laboratory to confirm field classifications and to evaluate pertinent engineering properties. Our laboratory testing program included grain-size analyses and moisture content determinations. A summary of our laboratory testing program and the test results are provided in Appendix A.

Soil and Groundwater Conditions

In our explorations, we typically observed about 2 inches of grass sod. Beneath the sod, we generally observed sand with silt and variable gravel content to silty sand with variable gravel and cobbles content in a medium dense to very dense condition. We also observed silt with sand, occasional gravel and stratified sandy silt and clay with occasional gravel in a medium stiff to stiff condition. These materials extended to a depth of about ¾ to 10½ feet bgs in explorations TP-1 (PIT-1), TP-3 (PIT-2) and TP-5. We interpret these materials to be fill. Fill was observed to the full depths explored in TP-2 and TP-4. We observed an approximate 3-inch thick layer of hot-mix asphalt within the fill at about 1½ feet bgs in TP-1 (PIT-1). We also observed an approximate 1-foot layer of silt with organics and occasional sand and gravel in a medium stiff condition at about 5 feet bgs in TP-2.

Underlying the fill in TP-1 (PIT-1) and TP-5, we observed laminated silt and clay with occasional gravel and silt with variable sand content in a stiff to very stiff condition, which we interpret to be recessional lacustrine deposits, extending to the full depths explored.

Underlying the fill in TP-3 (PIT-2), we observed silty sand with gravel and occasional cobbles in a very dense condition, which we interpret to be glacial till, extending to the full depths explored.

We did not observe the regional groundwater table in our explorations. We did, however, observe slow groundwater seepage (less than 1 gallon per minute) in exploration TP-2 at about 9 feet bgs. We also observed wet soil conditions at about 7 feet bgs to the termination depth in TP-4 and from about 5 to 6 feet bgs in TP-5. We interpret the seepage and/or wet soil conditions to be perched groundwater. Though not observed in explorations TP-1 (PIT-1) and TP-3 (PIT-2), we anticipate that perched groundwater could be present depending on rainfall amounts, irrigation activities and other factors. We anticipate that perched groundwater levels will generally be highest during the wet season, typically October through May.

CONCLUSIONS AND RECOMMENDATIONS

Primary Geotechnical Considerations

Based on our understanding of the project, the explorations performed for this study and our experience, it is our opinion that the proposed improvements can be designed and constructed generally as envisioned with regard to geotechnical considerations. A summary of the primary geotechnical considerations for the project is provided below and is followed by our detailed recommendations.

- We did not identify soils that we interpret to be prone to significant liquefaction in our explorations, and in our opinion the risk of liquefaction occurring at this site is low.
- Proposed structures at the site can be supported using shallow foundations and slabs-on-grade, provided that the foundation bearing surfaces are prepared as recommended. We do not anticipate that significant overexcavation will be required, unless isolated areas of loose, or otherwise unsuitable areas are encountered near foundation grade.
- Based on our field testing and observations, the infiltration capacity of the observed site soils is low.
- Soils observed at the site contain a significant quantity of fines, and, therefore, could be difficult or impossible to work with when wet or become easily disturbed if exposed to wet weather. Depending on the intended use of the material and the moisture/weather conditions, it may be difficult to re-use on-site soils as structural fill.

Seismic Design Considerations

Based on subsurface conditions encountered in our explorations and our understanding of the geologic conditions in the site vicinity, the site may be characterized as Class D in accordance with the 2015 International Building Code (IBC) Design Manual. Seismic design parameters are provided in Table 1, below.

TABLE 1. 2015 IBC SEISMIC DESIGN CRITERIA

Site Coefficient	Site Factor	MCE[1] Spectral Response	Design Spectral Response[2]
S_s = 1.477g	F_a = 1.0	S_{MS} = 1.477g	S_{DS} = 0.985g
S_1 = 0.552g	F_v = 1.5	S_{M1} = 0.828g	S_{D1} = 0.552g

Notes:

[1] MCE = Maximum Considered Earthquake

[2] Design spectral response = 2/3 * MCE response

Peak Ground Acceleration

The peak ground acceleration (PGA) is used in seismic analyses such as liquefaction, lateral spreading, and seismic slope stability as well as assessing seismic surcharge loads for retaining walls. Based on our understanding of site conditions, we recommend using a PGA equal to 0.611g for the project site as determined in accordance with Section 11.8.3 of American Society of Civil Engineers (ASCE) Standard 7-10.

Liquefaction

Liquefaction refers to a condition where vibration or shaking of the ground, usually from earthquake forces, results in development of excess pore pressures in loose, saturated soils and subsequent loss of strength in the deposit of soil so affected. In general, soils that are susceptible to liquefaction include loose to medium dense sands to silty sands that are below the water table. The *Liquefaction Susceptibility Map of King County, Washington* (Palmer, et al. 2004) indicates the site soils have a "very low" liquefaction potential. Based on observations and experience, we concur that the potential for liquefaction at the site is very low.

Lateral Spreading Potential

Lateral spreading related to seismic activity typically involves lateral displacement of large, surficial blocks of non-liquefied soil when a layer of underlying soil loses strength during seismic shaking. Lateral spreading usually develops in areas where sloping ground or large grade changes (including retaining walls) are present. Based on our understanding of the liquefaction risk at the site and the proposed improvements it is our opinion that the risk of lateral spreading is low.

Surface Rupture Potential

According to the Washington State Department of Natural Resources Interactive Natural Hazards Map (accessed August 3, 2018), there are no mapped faults within about 1 mile of the site. Based on the proximity of the site to the nearest mapped fault, it is our opinion the risk for surface rupture at this site is low.

Site Development and Earthwork

General

We anticipate that site development and earthwork will include the removal of asphalt pavement in areas of proposed improvements, excavating for shallow foundations, utilities and other improvements, establishing subgrades for foundations and roadways and placing and compacting fill and backfill materials. We expect that site grading and earthwork can be accomplished with conventional earthmoving equipment. The following sections provide specific recommendations for site development and earthwork.

Clearing and Stripping

We anticipate that clearing and stripping depths at the site will typically be on the order of about 6 to 10 inches to remove sod and associated root network at the surface. However, it is likely that greater stripping depths will be required in areas of heavier vegetation, lower lying areas or in areas containing trees.

During stripping operations excessive disturbance of surficial soils may occur, especially if left exposed to wet conditions. Disturbed soils may require additional remediation during construction and grading.

We encountered cobbles in our explorations, and while not observed, boulders can also be present in glacial deposits in the area. The contractor should be prepared to remove boulders and cobbles, if encountered during grading or excavation. Boulders may be removed from the site or used in landscape areas. Voids caused by boulder removal should be backfilled with structural fill.

Erosion and Sedimentation Control

Erosion and sedimentation rates and quantities can be influenced by construction methods, slope length and gradient, amount of soil exposed and/or disturbed, soil type, construction sequencing and weather. Implementing an Erosion and Sedimentation Control Plan will reduce impacts to the project where erosion-prone areas are present. The plan should be designed in accordance with applicable county and/or state standards. The plan should incorporate basic planning principles, including:

- Scheduling grading and construction to reduce soil exposure;
- Re-vegetating or mulching denuded areas;
- Directing runoff away from exposed soils;
- Reducing the length and steepness of slopes with exposed soils;
- Decreasing runoff velocities;
- Preparing drainage ways and outlets to handle concentrated or increased runoff;
- Confining sediment to the project site;
- Inspecting and maintaining control measures frequently.

Temporary erosion protection should be used and maintained in areas with exposed or disturbed soils to help reduce erosion and reduce transport of sediment to adjacent areas and receiving waters. Permanent erosion protection should be provided by paving, structure construction or landscape planting.

Until the permanent erosion protection is established, and the site is stabilized, site monitoring may be required by qualified personnel to evaluate the effectiveness of the erosion control measures and to repair and/or modify them as appropriate. Provisions for modifications to the erosion control system based on monitoring observations should be included in the erosion and sedimentation control plan. Where sloped areas are present, some sloughing and raveling of exposed or disturbed soil on slopes should be expected. We recommend that disturbed soil be restored promptly so that surface runoff does not become channeled.

Temporary Excavations

Excavations deeper than 4 feet must be shored or laid back at a stable slope if workers are required to enter. Shoring and temporary slope inclinations must conform to the provisions of Title 296 Washington Administrative Code (WAC), Part N, "Excavation, Trenching and Shoring." Regardless of the soil type encountered in the excavation, shoring, trench boxes or sloped sidewalls will be required under Washington Industrial Safety and Health Act (WISHA). The contract documents should specify that the contractor is responsible for selecting excavation and dewatering methods, monitoring the excavations for safety and providing shoring, as required, to protect personnel and structures.

In general, temporary cut slopes at this site should be inclined no steeper than about 1½H to 1V (horizontal to vertical). This guideline assumes that all surface loads are kept at a minimum distance of at least onehalf the depth of the cut away from the top of the slope and that seepage is not present on the slope face. Flatter cut slopes will be necessary where seepage occurs or if surcharge loads are anticipated. Temporary covering with heavy plastic sheeting should be used to protect slopes during periods of wet weather.

Groundwater Handling Considerations

Based on our understanding of the proposed site improvements we do not anticipate that the regional groundwater table will be encountered during excavations at the site.

Perched groundwater was observed in explorations TP-2, TP-4 and TP-5 and also is likely to be present in other areas at the site. The interface between the fill and recessional lacustrine deposits and contacts between more permeable and less permeable zones within the glacial soils are likely locations for accumulation of perched groundwater. Groundwater handling needs will typically be lower during the summer and early fall months. We anticipate that shallow perched groundwater can be handled adequately with sumps, pumps, and/or diversion ditches, as necessary. Ultimately, we recommend that the contractor performing the work be made responsible for controlling and collecting groundwater encountered.

Surface Drainage

Surface water from roof downspouts, driveways and landscape areas should be collected and controlled. Curbs or other appropriate measures such as sloping pavements, sidewalks and landscape areas should be used to direct surface flow away from buildings, erosion sensitive areas and from behind retaining structures. Roof and catchment drains should not be connected to wall or foundation drains.

Subgrade Preparation

Subgrades that will support structures and roadways should be thoroughly compacted to a uniformly firm and unyielding condition on completion of stripping and before placing structural fill. We recommend that subgrades for structures and roadways be evaluated, as appropriate, 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 that: (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 Protection and Wet Weather Considerations

Most of the near-surface soils observed in our explorations contain a significant quantity of fines and will be susceptible to disturbance during periods of wet weather. The wet weather season generally begins in October and continues through May in western Washington; however, periods of wet weather can occur during any month of the year. It may be possible to conduct earthwork at the site during wet weather months provided appropriate measures are implemented to protect exposed soil. If earthwork is scheduled during the wet weather months we offer the following recommendations:

- Measures should be implemented to remove or eliminate the accumulation of surface water from work areas. The ground surface in and around the work area should be sloped so that surface water is directed away and 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.

- Earthwork activities should not take place during periods of heavy precipitation.
- Slopes with exposed soils should be covered with plastic sheeting.
- The contractor should take necessary measures to prevent on-site soils and other soils to be used as fill from becoming wet or unstable. These measures may include the use of plastic sheeting, sumps with pumps and grading. The site soils should not be left uncompacted and exposed to moisture. Sealing exposed soils by rolling with a smooth-drum roller prior to periods of precipitation will help 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 working pad materials not susceptible to wet weather disturbance.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practical.
- Protective surfacing such as placing asphalt-treated base (ATB) or haul roads made of quarry spalls or a layer of free-draining material such as well-graded pit-run sand and gravel may be necessary to limit disturbance to completed areas. Minimum quarry spall thicknesses should be on the order of 12 to 18 inches. Typically, minimum gravel thicknesses on the order of 24 inches are necessary to provide adequate subgrade protection.

Fill Materials

Structural Fill

The workability of material for use as structural fill will depend on the gradation and moisture content of the soil. We recommend that washed crushed rock or select granular fill, as described below, be used for structural fill during wet weather. If prolonged dry weather prevails during the earthwork phase of construction, materials with a somewhat higher fines content may be acceptable. Weather and site conditions should be considered when determining the type of import fill materials purchased and brought to the site for use as structural fill.

Material used for structural fill should be free of debris, organic contaminants and rock fragments larger than 6 inches. For most applications, we recommend that structural fill consist of material similar to “Select Borrow” or “Gravel Borrow” as described in Section 9-03.14 of the Washington State Department of Transportation (WSDOT) Standard Specifications.

Select Granular Fill

Select granular fill should consist of well-graded sand and gravel or crushed rock with a maximum particle size of 6 inches and less than 5 percent fines by weight based on the minus ¾-inch fraction. Organic matter, debris or other deleterious material should not be present. In our opinion, material with gradation characteristics similar to WSDOT Specification 9-03.9 (Aggregates for Ballast and Crushed Surfacing), or 9-03.14 (Borrow) is suitable for use as select granular fill, provided that the fines content is less than 5 percent (based on the minus ¾-inch fraction) and the maximum particle size is 6 inches.

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. The material must be free of roots, debris, organic matter and other deleterious material. Other materials may be appropriate depending on manufacturer specifications and/or local jurisdiction requirements.

Trench Backfill

Trench backfill must be free of debris, organic material and rock fragments larger than 6 inches. We recommend that trench backfill material consist of material similar to "Select Borrow" or "Gravel Borrow" as described in Section 9-03.14 of the WSDOT Standard Specifications. Where excavations occur in the wet, alternative materials such as select granular fill should be considered.

On-Site Soil

Based on our subsurface explorations and experience, it is our opinion that existing site soils may be considered for use as structural fill and trench backfill, provided that they can be adequately moisture conditioned, placed and compacted as recommended and does not contain organic or other deleterious material. Based on our experience, the silty sands, silts and clays at the site are extremely moisture sensitive and will be very difficult or impossible to properly compact when wet.

In addition, it is likely that existing soils will be above optimum moisture content (OMC) when excavated, unless earthwork activities take place in the middle of summer. Even then, the soil could still be above OMC when excavated. Soils placed and compacted above OMC are typically difficult to work with and may have trouble achieving adequate compaction. If earthwork occurs during a typical wet season, or if the soils are persistently wet and cannot be dried back due to prevailing wet weather conditions or lack of drying space/time, we recommend the use of imported structural fill or select granular fill, as described above.

Fill Placement and Compaction

General

To obtain proper compaction, fill soil should be compacted near OMC and in uniform horizontal lifts. Lift thickness and compaction procedures will depend on the moisture content and gradation characteristics of the soil and the type of equipment used. The maximum allowable moisture content varies with the soil gradation and should be evaluated during construction. Generally, 8- to 12-inch loose lifts are appropriate for steel-drum vibratory roller compaction equipment. Compaction should be achieved by mechanical means. During fill and backfill placement, sufficient testing of in-place density should be conducted to check that adequate compaction is being achieved.

Area Fills and Pavement Bases

Fill placed to raise site grades and materials under pavements and structural areas should be placed on subgrades prepared as previously recommended. Fill material placed below structures and footings should be compacted to at least 95 percent of the theoretical maximum dry density (MDD) per ASTM International (ASTM) D 1557. Fill material placed shallower than 2 feet below pavement sections should be compacted to at least 95 percent of the MDD. Fill placed deeper than 2 feet below pavement sections should be compacted to at least 90 percent of the MDD. Fill material placed in landscaping areas should be compacted to a firm condition that will support construction equipment, as necessary, typically around 85 to 90 percent of the MDD.

Backfill Behind Walls

Backfill behind retaining walls or below-grade structure walls should be compacted to between 90 and 92 percent of the MDD. Overcompaction of fill placed directly behind 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 walls.

Trench Backfill

For utility excavations, we recommend that 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 above the pipe. In addition, rock fragments greater than about 1 inch in maximum dimension should be excluded from this lift.

Trench backfill material placed below structures and footings should be compacted to at least 95 percent of the MDD. In paved areas, trench backfill should be uniformly compacted in horizontal lifts to at least 95 percent of the MDD in the upper 2 feet below subgrade. Fill placed below a depth of 2 feet from subgrade in paved areas must be compacted to at least 90 percent of the MDD. In non-structural areas, trench backfill should be compacted to a firm condition that will support construction equipment as necessary.

Foundation Support

General

The proposed structures at the site can be satisfactorily supported on continuous wall and isolated column footings. Exterior footings should be established at least 18 inches below the lowest adjacent grade. Interior footings can be founded a minimum of 12 inches below the top of the floor slab. Isolated column and continuous wall footings should have minimum widths of 24 and 18 inches, respectively.

Based on the groundwater conditions in our explorations and our understanding of the proposed footing elevations (bottom of footings established at or within a few feet of existing site grade) it is our opinion footing drains are not necessary to maintain bearing support as provided in this report. However, because of the potential for near-surface seepage during wetter times of the year and from irrigation and potential landscaping, footing drains should be considered to maintain drier conditions around the structure and to reduce groundwater seepage that could migrate below the building slab.

The sections below provide our recommendations for foundation bearing surface preparation and foundation design parameters.

Foundation Bearing Surface Preparation

Shallow footing excavations should be performed using a smooth-edged bucket to limit bearing disturbance. Foundations should bear on existing proof-compacted mineral (non-organic) fill, native glacial soils or on structural fill extending to these soils. The bearing surface should be compacted as necessary to a firm, unyielding condition. Loose or disturbed materials present at the base of footing excavations should be removed or compacted.

If structural fill is placed below footings 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 fill (measured from the base of the footing where necessary), or 3 feet, whichever is less.

Foundation bearing surfaces should not be exposed to standing water. If water is present in the excavation, it must be removed before placing formwork and reinforcing steel. Protection of exposed soil, such as placing a 6-inch thick layer of crushed rock or a 3- to 4-inch layer of lean-mix concrete, could be used to limit disturbance to bearing surfaces.

Prepared foundation bearing surfaces should be evaluated by a member of our firm prior to placement of formwork or reinforcing steel to verify that bearing surface has been prepared in accordance with our recommendations or to provide recommendations for remediating unsuitable bearing soils.

Allowable Soil Bearing Pressure

Shallow foundations bearing on subgrades prepared as recommended may be designed using an allowable soil bearing pressure of 3,000 pounds per square foot (psf). This 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. These bearing pressures are appropriate for shallow foundations constructed within about 2 feet of existing site grade. We should be consulted if foundations will be constructed at elevations lower than about 2 feet of existing site grade.

Foundation Settlement

Disturbed soil must be removed from the base of footing excavations and the bearing surface should be prepared as recommended. Provided these measures are taken, we estimate the total static settlement of shallow foundations will be on the order of 1 inch or less for the bearing pressures presented above. Differential settlements could be on the order of ¼ to ½ inch between similarly loaded foundations or over a distance of 50 feet of continuous footings. The settlements should occur rapidly, essentially as loads are applied. Settlements could be greater than estimated if disturbed or saturated soil conditions are present below footings.

Lateral Resistance

The ability of the soil to resist lateral loads is a function of the base friction, which develops on the base of foundations and slabs, and the passive resistance, which develops on the face of below-grade elements of the structure as these elements move into the soil. For cast-in-place foundations supported in accordance with the recommendations presented above, the allowable frictional resistance on the base of the foundation may be computed using a coefficient of friction of 0.40 applied to the vertical dead-load forces. If precast foundations are included as part of project plans, we can provide specific recommendations for base friction resistance for precast foundations. The allowable passive resistance on the face of the foundation or other embedded foundation elements may be computed using an equivalent fluid density of 300 pounds per cubic foot (pcf).

These values include a factor of safety of about 1.5. The passive earth pressure and friction components may be combined provided that the passive component does not exceed two-thirds of the total. 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 a slab-on-grade.

Slab-on-Grade Floors

Slab-on-grade floors should bear on existing mineral fill, native glacial soils or on structural fill extending to these soils and should be prepared as recommended in the "Subgrade Preparation" section of this report. We recommend the slab subgrades be observed by a member of our firm during construction. Disturbed areas should be compacted, if possible, or removed and replaced with compacted structural fill. In all cases, the exposed soil should be compacted to a firm and unyielding condition.

We recommend the slab-on-grade floors be underlain by a minimum 6-inch-thick capillary break layer consisting of clean sand and gravel, crushed rock, or washed rock. The capillary break material should contain less than 3 percent fine material based on the percent passing the ¾-inch sieve size. Provided that loose soil is removed and the subgrade is prepared as recommended, we recommend slabs-on-grade be designed using a modulus of subgrade reaction of 200 pounds per cubic inch (pci). We estimate that settlement for slabs-on-grade constructed as recommended will be less than ¾ inch for a floor load of up to 500 psf.

Based on our understanding of subsurface conditions at the site it is our opinion that an underslab drain system is not necessary. 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.

Retaining Walls and Below-Grade Structures

Design Parameters

We recommend the following lateral earth pressures be used for design of conventional retaining walls and below-grade structures. Our design pressures assume that the ground surface around the retaining structures will be level or near level. If drained design parameters are used, drainage systems must be included in the design in accordance with the recommendations presented in the "Drainage" section below.

- Active soil pressure may be estimated using an equivalent fluid density of 35 pcf for the drained condition.
- Active soil pressure may be estimated using an equivalent fluid density of 80 pcf for the undrained condition; this value includes hydrostatic pressures.
- At-rest soil pressure may be estimated using an equivalent fluid density of 55 pcf for the drained condition.
- At-rest soil pressure may be estimated using an equivalent fluid density of 90 pcf for the undrained condition; this value includes hydrostatic pressures.
- For seismic considerations, a uniform lateral pressure 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.
- An additional 2 feet of fill representing a typical traffic surcharge of 250 psf should be included if vehicles are allowed to operate within a zone equal to the height of the retaining walls. Other surcharge loads should be considered on a case-by-case basis.

The active soil pressure condition assumes the wall is free to move laterally 0.001 H, where H is the wall height. The at-rest condition is applicable where walls are restrained from movement. The above recommended lateral soil pressures do not include the effects of sloping backfill surfaces or surcharge loads, except as described. Overcompaction of fill placed directly behind retaining walls or below-grade structures must be avoided to limit lateral pressures placed on the wall. We recommend use of hand-operated compaction equipment and maximum 6-inch loose lift thickness when compacting fill within about 5 feet of retaining walls and below-grade structures.

Retaining wall foundation bearing surfaces should be prepared following the "Foundation Bearing Surface Preparation" section of this report. Provided bearing surfaces are prepared as recommended, retaining wall foundations may be designed using the allowable soil bearing value and lateral resistance values presented above for building foundation design. We estimate settlement of retaining structures will be similar to the values previously presented for structure foundations.

Drainage

If retaining walls or below-grade structures are designed using drained parameters, a drainage system behind the structure must be included to collect water and prevent the buildup of hydrostatic pressure against the structure. We recommend the drainage system include a zone of free-draining backfill a minimum of 18 inches in width against the back of the wall. The drainage material should consist of coarse sand and gravel containing less that 5 percent fines based on the fraction of material passing the ¾-inch sieve.

A perforated, rigid, smooth-walled drain pipe with a minimum diameter of 4 inches should be placed along the base of the structure within the free-draining backfill and extend for the entire wall length. The drain pipe should be metal or rigid PVC pipe and be sloped to drain by gravity. Discharge should be routed to appropriate discharge areas and to reduce erosion potential. Cleanouts should be provided to allow routine maintenance. We recommend roof downspouts or other types of drainage systems not be connected to retaining wall drain systems.

Stormwater Infiltration

General

We evaluated stormwater infiltration rates at the site following methodology presented in the 2016 SWDM. We completed two PITs. TP-1 (PIT-1) was located in the grass area within the proposed overflow parking addition on the west side of the campus. TP-3 (PIT-2) was within the grass field located on the southwest corner of the campus. The sections below further describe our methodology and provide recommended infiltration rates for design.

Pilot Infiltration Tests

Methodology

The PITs were conducted following GeoEngineers' standard methodology for stormwater facilities in Western Washington. The GeoEngineers' procedure is a synthesis of best practices and, in our opinion, meets the intended procedures set forth in the 2016 SWDM.

Upon reaching the target excavation depth, 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 accurate water-level measurements at 15-second intervals throughout the duration of the test.

GeoEngineers' PIT procedure consists of a 6-hour (minimum) saturation period where the water depth in the PIT is raised and lowered between about 12 and 16 inches in a series of falling-head stages. Water level measurements collected by the pressure transducer during each water drop is 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.

Once a stabilized infiltration rate is observed and a minimum of 6 hours of saturation time has elapsed, the infiltration rate was estimated from the last stage for each PIT. The total test duration for TP-1 (PIT-1) and TP-3 (PIT-2) was about 7½ hours and 7¼ hours, respectively. After the PITs were complete, the test pits were excavated deeper. Groundwater seepage was not observed at either PIT location. However, we observed some lateral infiltration influence at TP-1 (PIT-1). The water used in the test was observed to infiltrate into the more permeable silty sand with gravel and quarry spalls and sand with silt layers underlying the 3-inch layer of hot-mix asphalt at about 1¾ feet bgs. At the conclusion of PIT-1, while advancing the test pit deeper we observed the water to migrate back into the excavation from these layers. In our opinion, the initial infiltration rate measured in this PIT is overstated, due to the higher permeability of the silty sand and quarry spalls and sand with silt. Accordingly, our recommendations below account for this effect.

Test Results

Figure 3 and Figure 4 show the measured water levels and infiltration rates at each stage of the PIT. Results indicate that saturated conditions and a stable infiltration rate was observed starting around hour 5 in PIT-1 and PIT-2.

The rates calculated in our PITs are representative of the measured (unfactored) infiltration rate of the soils at the test location. The SWDM recommends that correction factors be applied to the measured infiltration rates to estimate the long-term design infiltration rate. Different correction factors are applied depending on the facility type. The correction factors account for the number of infiltration tests in relation to the size of the infiltration facility area, site variability, test method and other factors.

Table 2 summarizes the partial and total correction factor(s) that, in our opinion, are suitable for design. Correction factors were selected based on our project understanding, observed soils conditions and our experience assisting in the design of stormwater infiltration facilities. The total correction factor (CF) is equal to the product of the partial correction factors.

TABLE 2. PIT CORRECTION FACTOR SUMMARY

Issue	Partial Correction Factor
Test Method ($F_{testing}$)	0.5
Geometry/Depth to Groundwater ($F_{geometry}$)	1.0
Long-Term Plugging ($F_{plugging}$)	0.7
Total Correction Factor = $F_{testing}$ x $F_{geometry}$ x $F_{plugging}$	CF = 0.35

Table 3 summarizes the measured and long-term infiltration rates determined in the PITs considering a CF = 0.35.

TABLE 3. INFILTRATION RATE SUMMARY

Pilot Infiltration Test Number	Measured Infiltration Rate (in/hr)	Long-Term Design Infiltration Rate (in/hr)
TP-1 (PIT-1)	0.35	0.1
TP-3 (PIT-2)	0.37	0.1

A discussion and further recommendations based on the testing results are provided below.

Discussion and Additional Considerations

General

Glacial soil deposits were observed at shallow depths at each PIT location. Fine-grained recessional lacustrine deposits were observed at about 3½ feet bgs at TP-1 (PIT-1) and dense glacial till was observed at less than 1-foot bgs at TP-3 (PIT-2). The regional groundwater table was not observed at either PIT location.

We have assumed that, if infiltration facilities are planned, they will serve less than about 1 acre of tributary area. The SWDM states that groundwater mounding analysis is not required for infiltration facilities serving less than 1 acre of tributary area provided that a minimum 5-foot separation is maintained between the bottom of the facility and seasonal high groundwater level or low permeability stratum (i.e., recessional lacustrine deposits and glacial till). At the locations tested where low permeability stratum was observed at shallow depths, this minimum separation would not be maintained. However, it is our opinion that a groundwater mounding analysis is not required provided that the long-term design infiltration rates listed above are used for design, because they represent the lower infiltration rate of the low permeability stratum.

Based on the PIT results, observed subsurface conditions and our experience, it is our opinion the soils at the locations tested have limited stormwater infiltration potential. The long-term design infiltration rates provided above may not be appropriate for large-scale infiltration facilities, such as infiltration ponds, but are suitable for permeable pavement and small footprint or low volume facilities. Other requirements outlined in the SWDM should be evaluated as required.

We request that if infiltration facilities are incorporated into site improvements, that we review the planned facility types, sizes and locations in-order to provide additional recommendations, as necessary.

Additional considerations are provided below for the areas we completed our PITs.

TP-1 (PIT-1) (Overflow Parking Addition)

We have assumed that permeable pavement may be considered by the design team for the proposed overflow parking addition on the west side of the campus. Accordingly, the approximate 3-inch thick layer of hot-mix asphalt observed between about 1½ and 1¾ feet bgs should be removed and the permeable pavement facility should include an adequately thick stormwater storage layer section. We recommend that additional explorations be completed to confirm the extents and/or presence of the hot-mix asphalt layer within the proposed footprint of the overflow parking addition. We can assist with additional explorations if requested.

Recommendations for permeable pavement design is discussed in further detail in the "Pervious Pavement" section of this report.

TP-3 (PIT-2) (Grass Field-Southwest Corner of Campus)

We have assumed that an infiltration pond may be considered by the design team to be located within the current grass field in the southwest corner of campus. We discussed above that based on our observations and the test results that infiltration ponds may not be appropriate. If an infiltration pond or other infiltration facility types are proposed in this area, we recommend that additional testing and explorations be completed within the footprint of each proposed facility. We can assist with additional testing and explorations if requested.

Pavement Recommendations

Conventional Asphalt Concrete Pavements

General

We provide recommended conventional ACP sections below, which are based on our experience because estimated traffic loading is not available. We also provide alternate sections wherein ATB is substituted for the crushed surfacing base course layer. These pavement sections may not be adequate for heavy construction traffic loads such as those imposed by concrete transit mixers, dump trucks or cranes. The contractor should consider planned construction loading and determine whether the design sections are sufficient to support construction loading without damage. The recommended sections assume that final improvements surrounding the conventional ACP will be designed and constructed such that stormwater or excess irrigation water from landscape areas does not accumulate below the pavement section or pond on pavement surfaces.

Pavement subgrade should be prepared, placed and observed as previously described. Crushed surfacing base course and subbase should be moisture conditioned to near optimum moisture content and compacted to at least 95 percent of MDD (ASTM D 1557).

Crushed surfacing base course should conform to applicable sections of 4-04 and 9-03.9(3) of the WSDOT Standard Specifications. Hot mix asphalt should conform to applicable sections of 5-04, 9-02 and 9-03 of the WSDOT Standard Specifications.

Standard-Duty ACP – Automobile Driveways and Parking Areas

- 2 inches of hot mix asphalt, class ½ inch, PG 64-22.
- 4 inches of crushed surfacing base course.
- 6 inches of subbase consisting of select granular fill to provide a uniform grading surface and pavement support, to maintain drainage, and to provide separation from subgrade soils.
- Existing site soils or structural fill prepared in accordance with the "Subgrade Preparation" section.

Heavy-Duty ACP – Areas Subject to Heavy Truck Traffic

- 3 inches of hot mix asphalt, class ½ inch, PG 64-22.
- 6 inches of crushed surfacing base course.

- 6 inches of subbase consisting of select granular fill to provide a uniform grading surface and pavement support, to maintain drainage, and to provide separation from subgrade soils.
- Existing site soils or structural fill prepared in accordance with the "Subgrade Preparation" section.

Pervious Pavement

General

Our recommendations for pervious pavement design sections are based on information provided in the technical guidance manual for LID (Puget Sound LID manual), completed by the Puget Sound Partnership (December 2012) and our experience designing permeable pavements in the region. The pavement sections presented below are suitable for use in driveway and parking areas and may not be suitable for use on surface streets or in areas with heavy traffic loads such as the bus loop area or entrances to the site. The design of pervious pavements for stormwater management should consider storage capacity of the pervious pavement system and infiltration rate of the subgrade soils. Our general recommendations are provided in the following sections; however, we recommend that final pervious pavement design should be in accordance with the complete recommendations provided in the Puget Sound LID manual.

Sections for pervious cement concrete pavement and porous asphalt pavement are presented below followed by specific recommendations for each section.

Pervious Cement Concrete Section

- 6 inches of pervious cement concrete.
- 6 inches (minimum) of permeable ballast, more permeable ballast may be required to provide adequate storage capacity for the section.
- Geotextile separation liner.
- Treatment layer (if necessary).
- Subgrade prepared as recommended below.

Porous Asphalt Concrete Section

- 4 inches of porous hot mix asphalt concrete.
- 6 inches (minimum) of permeable ballast, more permeable ballast may be required to provide adequate storage capacity for the section.
- Geotextile separation liner.
- Treatment layer (if necessary).
- Subgrade prepared as recommended below.

Pavement

Permeable pavements should be open graded and should have a minimum infiltration rate of at least 100 inches per hour when newly installed. Field infiltration tests should be considered on newly placed permeable pavements to verify the infiltration rate.

Permeable Ballast

We recommend a minimum 6-inch thick permeable ballast layer that meets the specification for American Public Works Association (APWA) General Special Provision (GSP) 9-03.9(2) Option 1 (shown in Table 4 below). A thicker permeable ballast layer may be necessary to provide sufficient storage capacity for the design infiltration rate. In general, the permeable ballast can be considered to have a porosity of 30 percent.

TABLE 4. GRADATION SPECIFICATION FOR PERMEABLE BALLAST

Sieve Size	Percent Passing
2½ inch	99-100
2 inches	65-100
¾ inch	40-80
No. 4	0-5
No. 100	0-2
% Fracture	95

Permeable ballast layers between 6 and 12 inches thick should be placed as a single lift. The ballast should be lightly compacted to a firm unyielding condition. Overcompaction of the ballast can result in reduced permeability. The prepared ballast layer should be observed by the geotechnical engineer to ensure that the ballast has been adequately compacted prior to placement of the permeable pavement. If the permeable ballast layer is thicker than 12 inches, it should be placed and compacted in multiple lifts not exceeding 12 inches in thickness.

Treatment Layer

Stormwater must be treated prior to infiltration. Stormwater can be captured and pretreated prior to infiltration, treatment layers can be built into the infiltration systems, or the existing site soils must meet treatment criteria outlined in the SWDM. In order to be suitable for stormwater treatment existing site soils must have a cation exchange capacity (CEC) greater than 5 milliequivalents/100 grams and an organic content of at least 1 percent. Completing CEC and organic content tests on the site soils was beyond our scope. Site soils should be tested to determine if they are suitable for stormwater treatment.

A geotextile separation fabric should be included between the bottom of the treatment layer and the prepared subgrade to prevent the treatment media from migrating into the subgrade soils. The separation geotextile should be non-woven and meet the requirement of WSDOT Standard Specification 9-33.1 for separation.

Subgrade Preparation

Subgrades below permeable pavement sections should be lightly compacted to a firm and unyielding condition before constructing the permeable pavement section; however, overcompaction of the subgrade should be avoided. Prepared subgrades should be protected from construction traffic, standing water or other disturbance. If portions of the subgrade become disturbed or are overcompacted, the subgrade should be scarified to a minimum depth of 8 inches and recompacted. The subgrade should be recompacted to between 90 and 92 percent of the MDD.

Protection, Maintenance and Icing

It is imperative that soils are not tracked onto pervious pavement surfaced areas during construction. Periodic visual inspections should be performed throughout the pavement life to determine if pervious pavement surfaces are clogged with fine soil or vegetation. Surfaces should be swept with a high-efficiency or vacuum sweeper regularly (typically at least two to four times per year) and washed with a high-pressure hose at least once per year.

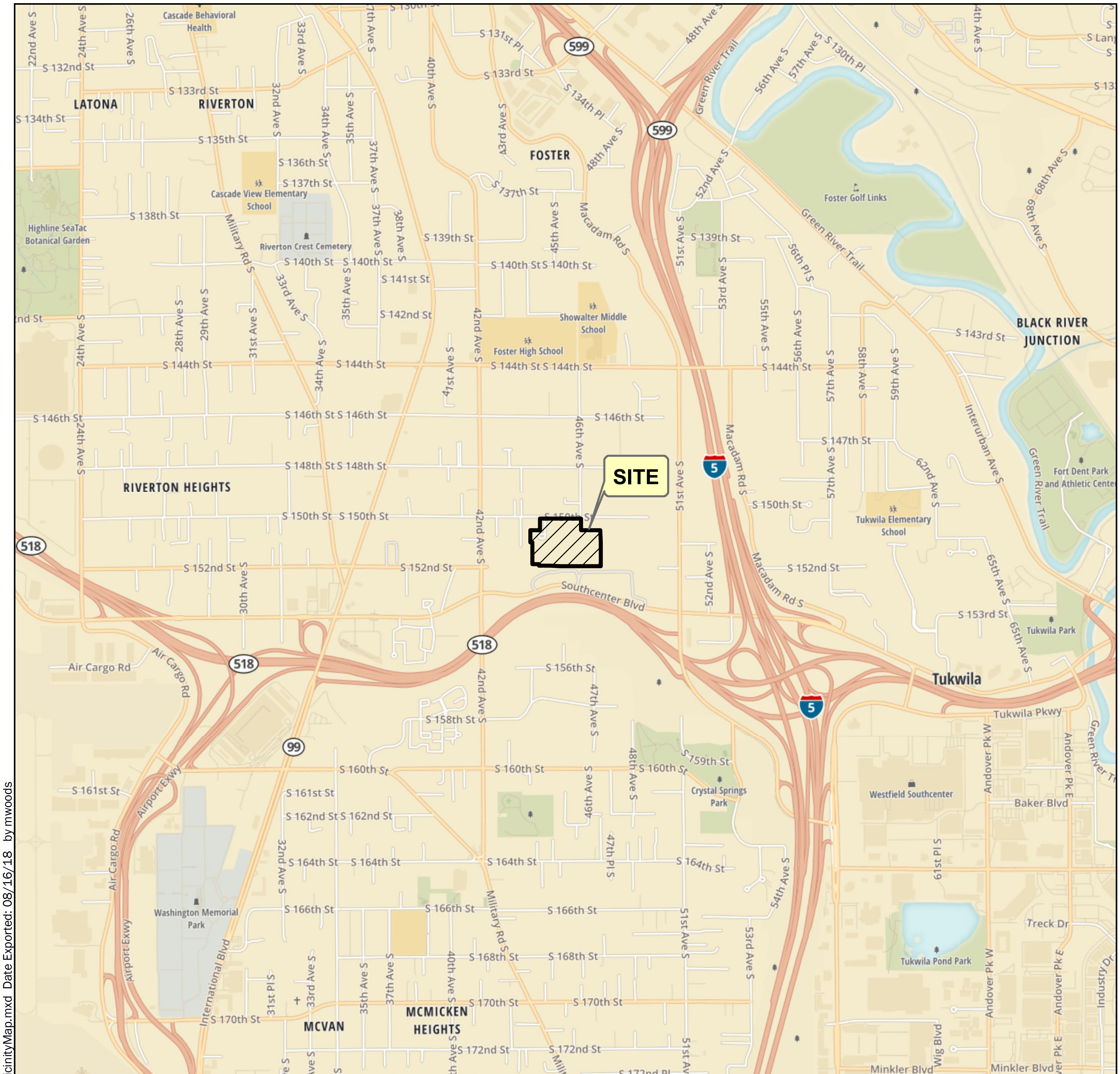
Because the relatively porous base and subbase layers allow some air movement below the pavement, pervious pavement surfaces may become icy more easily than conventional pavement surfaces. This problem is similar to differential icing of bridges and elevated road structures. Users should be made aware of the possibility of differential icing if pervious pavements are used.

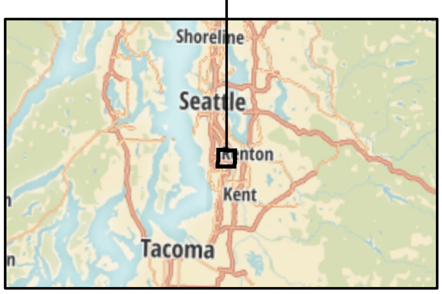
LIMITATIONS

We have prepared this report for Tukwila School District, No. 406 for the Thorndyke Elementary School Improvements project in Tukwila, Washington. Tukwila School District may distribute copies of this report to owner's authorized agents and regulatory agencies as may be required for the Project.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices for 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, express or implied, applies to the services or this report.

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





N
W E
S

2,000 0 2,000
Feet

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

Vicinity Map

Thorndyke Elementary School Improvements
Tukwila, Washington

GEOENGINEERS

Figure 1

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Aerial from Google Earth dated 5/13/2018.

Projection: NAD83 Washington State Planes, North Zone, US Foot

Legend

Site Boundary

TP-1 (PIT-1) Test Pit by GeoEngineers, 2018

N W E S

100 0 100

Feet

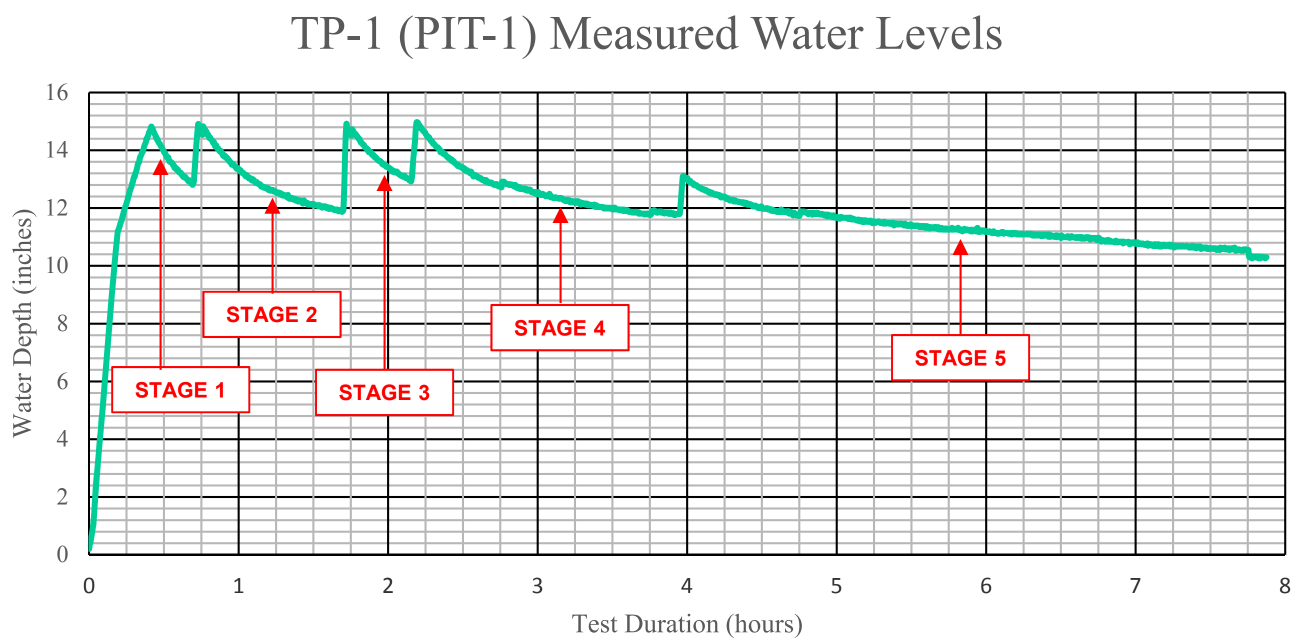
Site Plan

Thorndyke Elementary School Improvements
Tukwila, Washington

GeoEngineers

Figure 2

\\geoengineers.com\WAN\Projects\23\23537001\CAD\00\GeoTech\2353700100_F02_Site Plan.dwg TAB:F02 Date Exported: 08/16/18 - 14:05 by mwoods



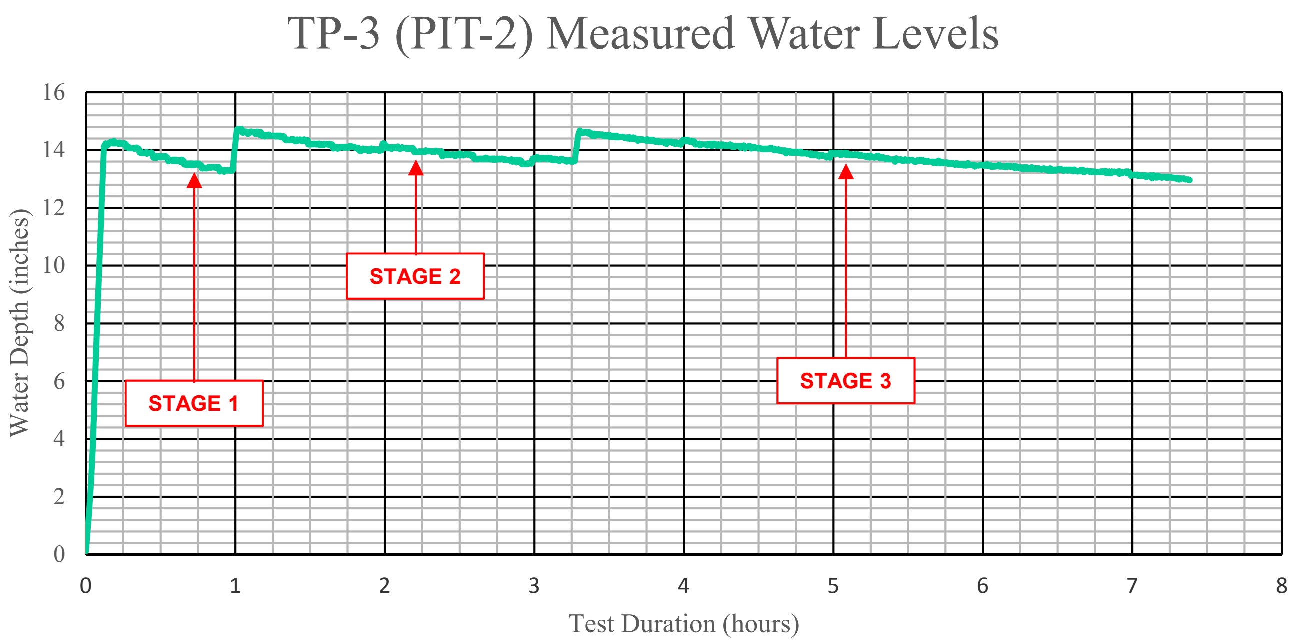
TP-1 (PIT-1) Measured (Short-Term) Infiltration Rates	
Stage	**Measured Infiltration Rate (in/hr)**
1	7.02
2	2.81
3	4.32
4	0.93
5	0.35

TP-1 (PIT-1) Results

Thorndyke Elementary School Improvements
Tukwila, Washington

GEOENGINEERS

Figure 3



TP-3 (PIT-2) Measured (Short-Term) Infiltration Rates	
Stage	**Measured Infiltration Rate (in/hr)**
1	1.27
2	0.46
3	0.37

23537-001-00 Date Exported: 081718

TP-3 (PIT-2) Results

Thorndyke Elementary School Improvements
Tukwila, Washington

GEOENGINEERS

Figure 4

APPENDIX A
Subsurface Explorations and Laboratory Testing

APPENDIX A
SUBSURFACE EXPLORATIONS AND LABORATORY TESTING

Subsurface Explorations

Test Pits and Pilot Infiltration Tests

Subsurface conditions for the proposed Thorndyke Elementary School Improvements project were explored by excavating five test pits on July 30, 2018 at the approximate locations shown on Figure 2. Pilot infiltration tests (PIT) were completed at about 3¼ feet and 4 feet below ground surface (bgs) at TP-1 (PIT-1) and TP-3 (PIT-2), respectively. The test pits were excavated to depths between about 8 and 11 feet bgs using a subcontracted backhoe and operator to GeoEngineers. After each test pit was completed, the excavation was backfilled using the generated material. The backfill was compacted using the bucket of the backhoe.

Our field representative obtained samples, classified the soils encountered, and maintained a detailed log of each exploration. The relative densities noted on the test pit logs are based on the difficulty of excavation and our experience and judgment. The samples were collected and retained in sealed plastic bags and then transported back to our office. The soils were classified visually in general accordance with the system described in Figure A-1, which includes a key to the exploration logs. Summary logs of the explorations are included as Figures A-2 through A-6.

The locations of the test pits were determined via an electronic tablet with global positioning system (GPS) software. The locations of the explorations should be considered approximate.

Laboratory Testing

Soil samples obtained from the borings were transported to GeoEngineers laboratory. Representative soil samples were selected for laboratory tests to evaluate the pertinent geotechnical engineering characteristics of the site soils and to confirm our field classification.

Our testing program consisted of the following:

- Five grain-size distribution analyses (four sieve analyses [SA] and one hydrometer analysis [HA])
- Four moisture content determinations (MC)

Tests were performed in general accordance with test methods of ASTM International (ASTM) or other applicable procedures. The following sections provide a general description of the tests performed.

Sieve Analysis (SA)

Grain-size distribution analyses were completed on selected samples 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. The results of the tests were used to verify field soil classifications and determine pertinent engineering characteristics. Figures A-7 and A-8 present the results of our sieve analyses.

Hydrometer Analysis (HA)

A grain-size distribution analysis was performed on a selected sample in general accordance with ASTM Test Method D 422. This test method covers the quantitative determination of the distribution of particle sizes in soils. Typically, the distribution of particle sizes larger than 75 µm is determined by sieving, and the distribution of particle sizes smaller than 75 µm is determined by a sedimentation process using a hydrometer. The hydrometer analysis alone determines the distribution of particle sizes smaller than 2 millimeters (mm). The hydrometer test sample included particle sizes smaller than 2 mm but did not include a corresponding sieve analysis. The results of the test were used to verify field soil classifications and determine pertinent engineering characteristics. Figure A-7 presents the results of our hydrometer analysis.

Moisture Content (MC)

The moisture content of selected samples 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 results are presented on the test pit logs at the depth tested.

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 ON 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

Sampler Symbol Descriptions

- 2.4-inch I.D. split barrel
- 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
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

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	7/30/2018	Total Depth (ft)	8.5	Logged By	SAH	Excavator	Kelly's Excavating	Groundwater not observed
				Checked By	CRN	Equipment	Komatsu WB 140 (Backhoe)	Caving not observed

Surface Elevation (ft)	230	Easting (X)	1283035	Coordinate System	WA State Plane North
Vertical Datum	NAVD88	Northing (Y)	174068	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 2 inches sod (grass)			
					SM	Approximately 3 inches gray silty fine to medium sand with occasional organic matter (roots) (medium dense, moist) (fill)			
229	1		1		SP-SM	Gray fine to medium sand with silt and gravel and occasional deleterious debris (wood, metal pipe fragment, concrete, bricks) (medium dense, moist)			
					AC	Approximately 3-inch layer hot mix asphalt			
228	2				SM	Brown silty fine to coarse sand with gravel and quarry spalls (dense, moist)			
227	3		2 SA		SP-SM	Gray fine to medium sand with silt and occasional gravel (dense, moist)	11	8	Pilot Infiltration Test completed at approximately 3¼ feet
226	4				ML/CL	Gray to brown-gray with iron oxide staining alternating laminations of silt and clay with occasional gravel (stiff, moist) (recessional lacustrine deposits)			
225	5								
224	6		3 HA				37		
223	7		4						
222	8				ML	Gray silt with fine sand (stiff, moist)			
			5 MC				28		

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 Aerial Imagery. Vertical approximated based on Google Earth.

Log of Test Pit TP-1 (PIT-1)

GEOENGINEERS

Project: Thorndyke Elementary School Improvements
Project Location: Tukwila, Washington
Project Number: 23537-001-00

Date Excavated	7/30/2018	Total Depth (ft)	10	Logged By	SAH	Excavator	Kelly's Excavating	See "Remarks" section for groundwater observed
				Checked By	CRN	Equipment	Komatsu WB 140 (Backhoe)	See "Remarks" section for caving observed

Surface Elevation (ft)	220	Easting (X)	1283731	Coordinate System	WA State Plane North
Vertical Datum	NAVD88	Northing (Y)	174067	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 inches sod (grass)			
219	1				SM	Approximately 10 inches gray silty fine to medium sand with gravel and occasional organic matter (roots) (dense, moist) (fill)			
218	2				SM	Gray silty fine to coarse sand with gravel and occasional cobbles and deleterious debris (wood debris) (very dense, moist)			
217	3				SM	Gray-brown silty fine sand with occasional gravel and deleterious debris (PVC pipe fragments at approximately 4 feet) (very dense, moist)			
216	4	▯	1 / SA				10	40	
215	5	▯	2 / MC		ML	Dark brown silt with organic matter and occasional sand and gravel (medium stiff, moist)	69		Minor caving observed at approximately 5 feet Includes woody debris
214	6	▯	3		SM	Gray silty fine to medium sand with gravel and occasional organic matter and includes lenses of dark brown organic silt (medium dense, moist)			
213	7								
212	8					Grades to with occasional cobbles			
211	9	▯	4		SM	Gray-brown silty fine to medium sand with occasional gravel and cobbles (medium dense, wet)			Slow groundwater seepage observed at approximately 9 feet on south side of test pit
210	10								

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 Aerial Imagery. Vertical approximated based on Google Earth.

Log of Test Pit TP-2

GEOENGINEERS

Project: Thorndyke Elementary School Improvements
Project Location: Tukwila, Washington
Project Number: 23537-001-00

Figure A-3
Sheet 1 of 1

Date Excavated	7/30/2018	Total Depth (ft)	9.5	Logged By	SAH	Excavator	Kelly's Excavating	Groundwater not observed
				Checked By	CRN	Equipment	Komatsu WB 140 (Backhoe)	Caving not observed

Surface Elevation (ft)	230	Easting (X)	1283097	Coordinate System	WA State Plane North
Vertical Datum	NAVD88	Northing (Y)	173939	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 inches sod (grass)			
					SM	Approximately 6 inches gray-brown silty fine to coarse sand with gravel and occasional organic matter (roots) (medium dense, moist) (fill)			
229	1				SM	Gray-brown silty fine to medium sand with gravel and occasional cobbles (very dense, moist) (glacial till)			
228	2		1						
227	3				SM	Gray silty fine to coarse sand with gravel and occasional cobbles (very dense, moist)			
226	4		2 SA				10	14	Pilot infiltration test completed at approximately 4 feet
225	5								
224	6		3						
223	7				SM	Gray silty fine to medium sand with occasional gravel and cobbles (very dense, moist)			
222	8		4						
221	9		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 Aerial Imagery. Vertical approximated based on Google Earth.

Log of Test Pit TP-3 (PIT-2)

GEOENGINEERS

Project: Thorndyke Elementary School Improvements
Project Location: Tukwila, Washington
Project Number: 23537-001-00

Figure A-4
Sheet 1 of 1

Date Excavated 7/30/2018	Total Depth (ft) 8	Logged By SAH Checked By CRN	Excavator Kelly's Excavating Equipment Komatsu WB 140 (Backhoe)	Groundwater not observed Caving not observed	

Surface Elevation (ft)	220	Easting (X)	1283678	Coordinate System	WA State Plane North
Vertical Datum	NAVD88	Northing (Y)	173939	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 inches sod (grass)			
					SM	Approximately 6 inches gray silty fine to coarse sand with gravel and occasional organic matter (roots) (medium dense, moist) (fill)			
219	1				SM	Gray-brown silty fine sand with gravel and occasional cobbles and deleterious debris (glass) (very dense, moist)			
218	2		1 / SA				8	33	
					SM	Gray-brown silty fine to medium sand with occasional gravel and cobbles (very dense, moist)			
217	3								
216	4		2						
					ML/CL	Stratified brown sandy silt with occasional gravel and gray clay with occasional gravel and organic matter (stiff, moist)			
215	5		3 / MC				25		
					SM	Gray silty fine to coarse sand with gravel (medium dense, moist)			
214	6								
213	7					Grades to wet			
212	8		4						1-inch diameter by approximately 8 feet long metal pipe remnant encountered between about 7 and 8 feet depth.

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 Aerial Imagery. Vertical approximated based on Google Earth.

Log of Test Pit TP-4

GEOENGINEERS

Project: Thorndyke Elementary School Improvements
Project Location: Tukwila, Washington
Project Number: 23537-001-00

Figure A-5
Sheet 1 of 1

Date Excavated 7/30/2018	Total Depth (ft) 11	Logged By SAH Checked By CRN	Excavator Kelly's Excavating Equipment Komatsu WB 140 (Backhoe)	Groundwater not observed See "Remarks" section for caving observed

Surface Elevation (ft) 220 Vertical Datum NAVD88	Easting (X) 1283838 Northing (Y) 173955	Coordinate System WA State Plane North 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 inches sod (grass)			
				SM	Approximately 8 inches gray silty fine to medium sand with gravel and occasional organic matter (roots) (medium dense, moist) (fill)			
219	1			SM	Gray-brown silty fine to coarse sand with gravel and occasional cobbles and organic matter (roots) (very dense, moist)			
218	2		1					
217	3			SM	Gray-brown silty fine to medium sand with occasional gravel and cobbles (medium dense, moist)			
216	4		2					
215	5		3 MC	ML	Gray and brown silt with sand and occasional gravel and organic matter (roots) (medium stiff, wet)	32		Minor caving observed at approximately 5¼ feet on north and south side of test pit
214	6		4	SM	Gray silty fine to medium sand with gravel and occasional cobbles and organic matter (medium dense, moist)			
213	7							
212	8							
211	9							
210	10							
209	11		5	ML	Brown-gray with occasional iron oxide staining silt (very stiff, moist) (recessional lacustrine deposits)			

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 Aerial Imagery. Vertical approximated based on Google Earth.

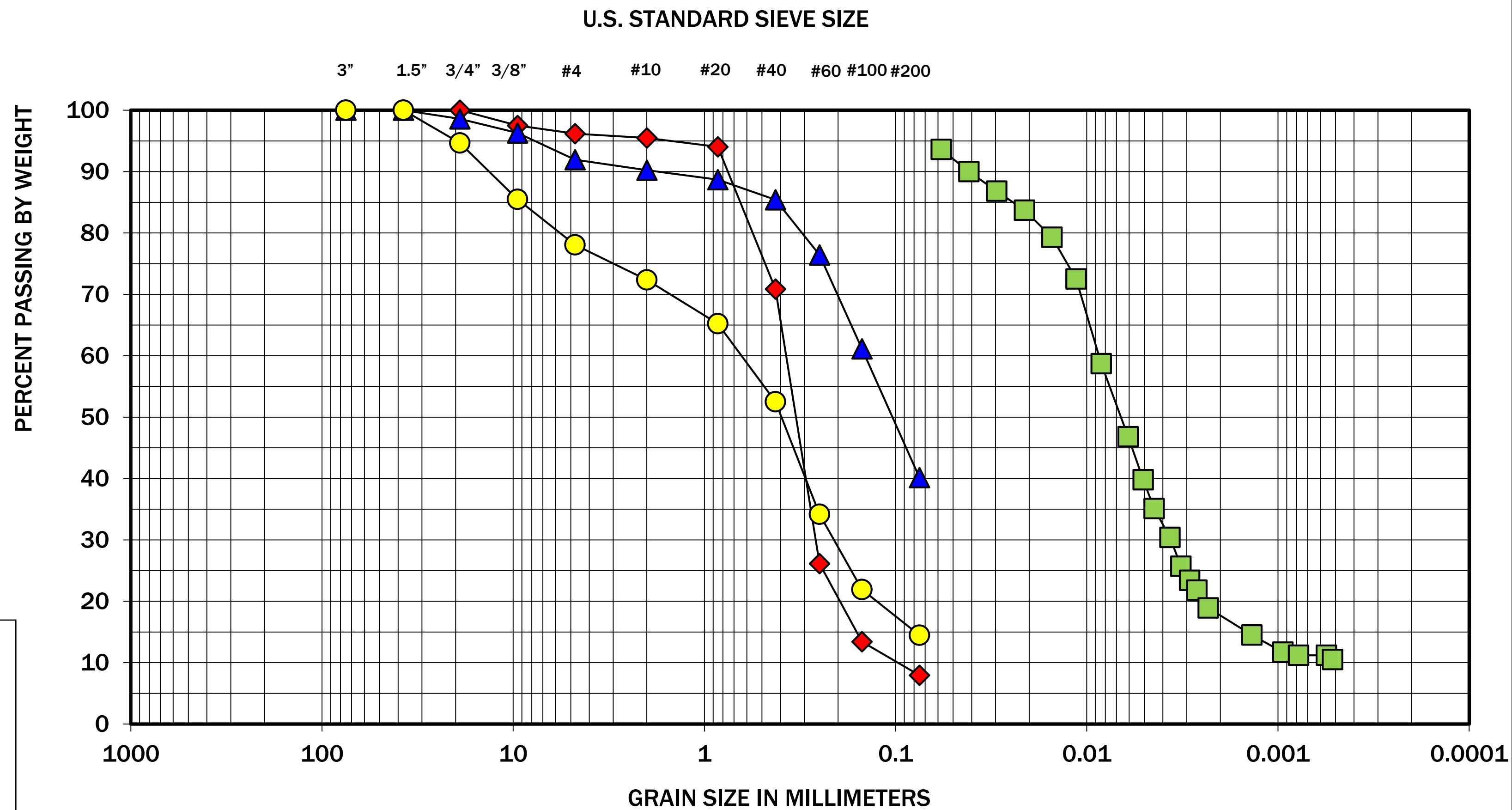
Log of Test Pit TP-5

GEOENGINEERS

Project: Thorndyke Elementary School Improvements
Project Location: Tukwila, Washington
Project Number: 23537-001-00

Figure A-6
Sheet 1 of 1

123537-001-00 Date Exported: 8/8/18



<table>
<tr><td rowspan="2">COBBLES</td><td colspan="2">GRAVEL</td><td colspan="3">SAND</td><td rowspan="2">SILT OR CLAY</td></tr>
<tr><td>COARSE</td><td>FINE</td><td>COARSE</td><td>MEDIUM</td><td>FINE</td></tr>
</table>

Symbol	Test Pit Number	Depth (feet)	Moisture (%)	Laboratory Soil Description
◆	TP-1 (PIT-1)	3.25	1	Fine to medium sand with silt (SP-SM)
■	TP-1 (PIT-1)	5.5	37	Silt (ML)
▲	TP-2	3.5	10	Silty fine sand with occasional gravel (SM)
●	TP-3 (PIT-2)	3.75	10	Silty fine to medium sand with gravel (SM)

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 D 6913. The Hydrometer analysis results were obtained in general accordance with ASTM D422

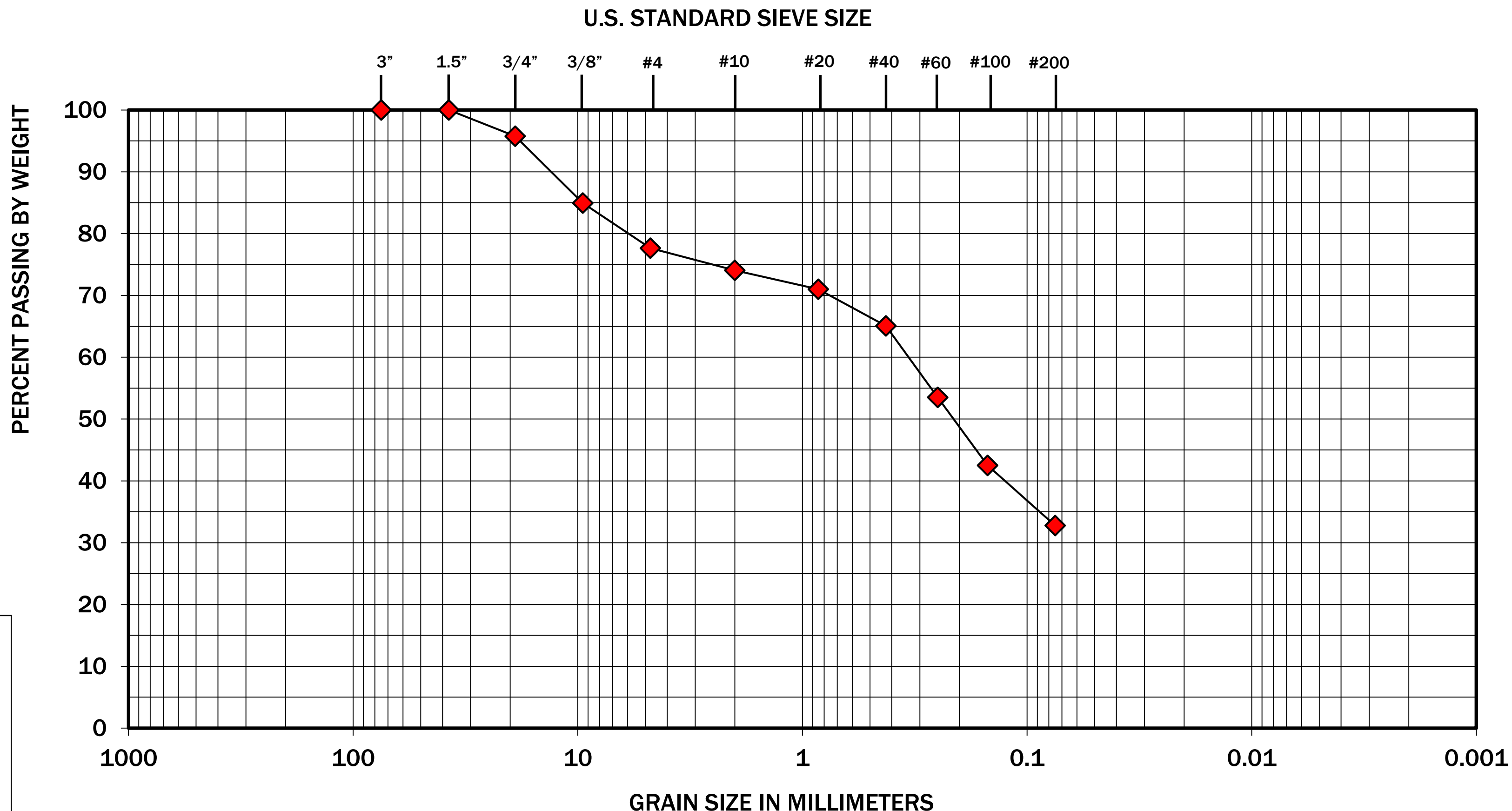
Sieve-Hydrometer Analysis Results

Thorndyke Elementary School Improvements
Tukwila, Washington

GeoEngineers

Figure-A-7

123537-001-00 Date Exported: 8/8/18



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Test Pit Number	Depth (feet)	Moisture (%)	Laboratory Soil Description
◆	TP-4	1.5	8	Silty fine sand with gravel (SM)

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 D 6913.

Sieve Analysis Results

Thorndyke Elementary School Improvements
Tukwila, Washington

GEOENGINEERS

Figure-A-8

APPENDIX B
Report Limitations and Guidelines for Use

APPENDIX B
REPORT LIMITATIONS AND GUIDELINES FOR USE[1][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 Tukwila School District and 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 our Agreement with Tukwila School District, No. 406 dated July 24, 2018 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 Thorndyke Elementary School in Tukwila, 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.

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

- the function of the proposed structure;

[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

- 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.

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.

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.