GEOTECHNICAL ENGINEERING SERVICES
WASTEWATER TREATMENT PLANT IMPROVEMENTS
MBR UPGRADE AND EXPANSION
ARLINGTON, WASHINGTON

JANUARY 9, 2008

FOR
CITY OF ARLINGTON

GEOENGINEERS

File No. 5430-004-00 T1

Geotechnical Engineering Services
Wastewater Treatment Plant Improvements
MBR Upgrade and Expansion
Arlington, Washington
File No. 5430-004-00

January 9, 2008

Prepared for:

City of Arlington
c/o Kennedy/Jenks Consultants
32001 32nd Avenue South, Suite 100
Federal Way, Washington 98001

Attention: Christopher W. Kelsey, PE

Prepared by:

GeoEngineers, Inc.
2924 Colby Avenue
Everett, Washington 98201
(425) 252-4565

DEBRA C. OVERBAY
STATE OF WASHINGTON
29247
REGISTERED PROFESSIONAL ENGINEER
EXPIRES 7/23/09

Debra C. Overbay, PE
Senior Engineer

Thomas A. Tobin, PE
Principal

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INTRODUCTION AND PROJECT UNDERSTANDING

This report presents the results of our design geotechnical engineering services for the proposed improvements to the City of Arlington WWTP located in the northwest area of the city. The facility is located on the north side of Burke Avenue and east side of SR 9 as shown in the Vicinity Map, Figure 1. Existing site conditions are shown on the Site Plan, Figure 2. Main access to the facility is from Haller Avenue one block north of Burke Avenue.

We understand that the proposed MBR Upgrade/Expansion project will include:

- Construction of a membrane bioreactor (MBR) system that includes tanks adjacent to a new treatment building,
- expansion of the Support Building for additional Blowers/UV,
- new aerobic digesters and odor control structures,
- expansion of the existing Solids Handling building,
- expansion of the Headworks building,
- construction of a new Equipment building, and
- construction of a new Lab/Office building.

The MBR building will be located in the north end of the site adjacent to the administration building, and the remaining new structures and aerobic digesters will be located in the south and east portion of the site. Specific system dimensions, details, and loading have not been finalized. We expect the buildings will be one to two-story structures with concrete framing and reinforced concrete masonry unit (CMU) block walls. Structural loads imparted to the building foundation are expected to be light to moderate.

The purpose of our geotechnical engineering services is to review existing subsurface data and complete supplemental subsurface explorations as a basis for providing design geotechnical recommendations for the project. Our scope of services included drilling four hollow-stem auger borings, completing laboratory testing on the samples obtained from the explorations, and providing geotechnical recommendations for earthwork and subgrade preparation, excavation and shoring, foundation design, subsurface walls and drainage, and seismic design.

REGIONAL GEOLOGY

The complex geologic conditions in the Stillaguamish River valley is the result of several episodes of mountain building, volcanism, interglacial erosion, scour by the glaciers, deposition of glacial and non-glacial sediments, and post-glacial deposition and erosion. The Fraser Glaciation is the most recent continental glaciation. Erosion and deposition during and following the Fraser Glaciation have resulted in the modern topography of the Stillaguamish River valley.

The last glaciation in the vicinity of the site was known as the Vashon Stade of the Fraser Glaciation. The Vashon ice sheet flowed south from Canada through the Puget Sound lowlands and locally extended into and across various major drainages. At its maximum extent, approximately 15,000 years ago, the glacial ice was likely more than 3,000 feet thick in the vicinity of the site. The Vashon ice sheet likely receded from the area approximately 12,000 to 13,000 years ago. Erosion and deposition during and following the Vashon Stade have resulted in the modern topographic features of the area.

Geologic mapping of the area indicates younger alluvium is present north of the site near the river, recessional outwash of the Marysville Sand Member is located within the site, and recessional outwash of the Arlington Gravel Member is located south and east of the site. Younger alluvium typically consists of fine-grained sand and silt in the lower courses of recent channels, and coarse sand and gravel in the upper bars. Recessional outwash of the Marysville Sand Member typically consists of well-stratified sand with some fine gravel and some areas of silt and clay. The Arlington Gravel Member deposits typically consist of well-stratified gravel and sand. Localized zones of fill should also be anticipated from previous site development.

SITE CONDITIONS

SURFACE CONDITIONS

Existing surface conditions at the WWTP site consist primarily of asphalt concrete with some small strips of lawn and isolated areas of concrete. Main access to the site is off of Haller Avenue as shown on the Site Plan, Figure 2. Existing structures and tanks are also presented in the site plan.

The site slopes down to the north and west from the southeast corner of the site with an overall site relief of approximately 25 feet. Burke Avenue is elevated 10 to 20 feet above the south border of the site. The roadway embankment is retained by an MSE wall (mechanically stabilized earth wall that appears to be Reinforced Earth™). SR 9 is located along the western boundary and is also elevated above the site. The embankment height of SR 9 is on the order of 20 feet and the embankment slopes down to the property at an inclination of approximately 2H:1V (horizontal to vertical).

SUBSURFACE EXPLORATION

Subsurface soil and groundwater conditions were evaluated by reviewing existing subsurface data and by drilling four hollow-stem auger borings. Three of the borings were drilled with a truck-mounted rig and boring B-2 was advanced with a portable rig. The borings were advanced to a depth of 18.5 to 19 feet each. The approximate locations of the explorations are shown on the attached site plan, Figure 2.

The explorations were continuously monitored by a geotechnical engineer from our firm who examined and classified the soils encountered, obtained representative soil samples, and observed groundwater conditions. All samples were brought back to our laboratory for additional classification, moisture testing and sieve analyses. Additional details of the field exploration program and laboratory testing are provided in the appendix.

SOIL CONDITIONS

Borings B-1, B-2 and B-3 were advanced through the existing pavement and encountered 3 to 4 inches of asphalt concrete overlying 6 to 8 inches of crushed rock base course. A 2-inch-thick layer of concrete was encountered in boring B-1 between the asphalt and base course. The remaining boring, boring B-4, was advanced through 10 inches of crushed rock surfacing. Beneath the pavement section or gravel surfacing, the borings encountered a four- to seven-foot thickness of medium dense granular fill. The fill content varies from silty sand with gravel (SM) in borings B-1 and B-2, to gravel with silt and sand (GP-GM) in boring B-3, and to sand with silt and gravel (SP-SM) in boring B-4. Loose to medium dense silty sand and medium stiff to stiff silt (likely alluvium) underlies the fill in borings B-2 and B-3 to a depth of about 11 to 13 feet. Medium dense to dense recessional outwash (sand and gravel with variable silt content) underlies the alluvium in these borings and the fill in borings B-1 and B-4.

GROUNDWATER CONDITIONS

Groundwater seepage was encountered during drilling at a depth of 10 to 11 feet in borings B-1, B-3 and B-4. Groundwater was encountered at a depth of 4 feet in boring B-2. The groundwater may be locally higher near the adjacent roadway embankments if some perched groundwater occurs seasonally. Groundwater levels will likely fluctuate in response to precipitation, seasonal variation, river elevations, and other factors.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

Based on the results of our subsurface exploration program, it is our opinion that the proposed WWTP improvements may be constructed satisfactorily as planned with respect to geotechnical issues provided adequate excavation and foundation preparation methods are implemented. Our explorations typically encountered medium dense to dense fill and outwash deposits. Loose alluvial deposits or fill was encountered at a greater depth in boring B-3. Subgrade improvement recommendations beneath new footings and slabs are recommended in following sections to mitigate for potential variability in the fill and alluvial deposits. A summary of the primary site preparation and design considerations for the proposed project is provided below. The summary is presented for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report.

- The existing fill, alluvium, and outwash deposits contain sufficient fines such that they are moisture-sensitive soils that will become easily disturbed when wet. If practical, we recommend site development be accomplished during extended periods of dry weather when the surficial soils will be less susceptible to disturbance. Alternatively, additional excavation and replacement of portions of the foundation and slab subgrade soils may be necessary in wet conditions.

- A high groundwater condition should be anticipated during the wet season from precipitation and surface water runoff from the adjacent elevated areas. We anticipate that groundwater encountered during shallow excavations can be adequately handled by routing to collection ditches and using sump pumps. Construction dewatering for deeper excavations, if applicable, is discussed in the “Excavation” section of this report.

- Temporary slopes should be inclined at 1.5H:1V (horizontal to vertical) or flatter depending on localized sloughing. Recommendations for temporary shoring where space constraints limit the use of open cuts are provided in a following section.

- Shallow foundations are suitable for support of proposed new structures and additions. We recommend an allowable soil bearing pressure of 3,000 psf (pounds per square foot) be utilized for design of building footings bearing on a minimum 2-foot thickness of compacted structural fill.

- The MBR tank and treatment building can be supported on shallow footings and concrete slab-on-grade floors or mat foundations. Some overexcavation of unsuitable fill soils and backfilling with structural fill will be required.

- Buoyancy should be considered in design of structures below the static groundwater surface.

- We recommend a minimum 12-inch-thick ¾-inch-minus crushed rock subbase underlie new mat foundations. The crushed rock should contain a minimum of 25 percent retained on the U.S. No. 4 sieve and no more than 3 percent passing the U.S. No. 200 sieve. The underlying subgrade should be compacted to a minimum 95 percent of the MDD prior to subbase placement. We recommend a geotechnical engineer from our firm evaluate the exposed subgrade to confirm soil conditions are as expected. A modulus of subgrade reaction of 200 pci (pounds per cubic inch) is appropriate for design.

- Crushed rock subbase should also underlie slabs-on-grade. We recommend a 6-inch-thick crushed rock subbase layer underlie new slabs-on-grade. The subbase layer should have the same gradation requirement as described above for mat foundations. The slab subgrade should be compacted to a minimum 95 percent of the maximum dry density prior to placing the subbase.

- Lateral loads may be resisted by friction on the base of footings and the mat slabs and passive resistance on the sides of the footings. Detailed parameters for lateral resistance are provided in a following section.

- We recommend Site Class D in accordance with IBC 2006 be utilized for seismic design. Based on the soil consistency and groundwater table, a small portion of the subsurface soils are susceptible to liquefaction. We estimate that ground settlement from the result of liquefaction during a moderate earthquake to be less than about 1 inch. However, lateral spreading of the river bank located approximately 300 feet north of the site may occur. This is discussed in more detail in a following section.

- Where new pavements are required, we recommend a pavement design section consisting of a minimum 3 to 4 inches of ½-inch HMA (PG 58-22) overlying 6 inches of crushed surfacing base course. A 3-inch HMA thickness is appropriate for general site access and a 4-inch HMA thickness is appropriate for areas that will experience repeated truck traffic or frequent turning. The base course should be placed on the existing fill soils following compaction to a minimum of 95 percent of the maximum dry density in accordance with ASTM D 1557.

EXCAVATIONS

Temporary Cut Slopes

We recommend that temporary unsupported cut slopes greater than 4 feet deep be inclined no steeper than 1.5H:1V. This applies to fully dewatered conditions. Flatter slopes will be necessary if seepage is present on the cut face. Temporary unsupported cut slopes should encroach no closer that 10 feet laterally from existing structures, pavements or improvements. Some sloughing and raveling of the cut slopes should be expected. Temporary covering, such as heavy plastic sheeting, should be used to protect these slopes during periods of rainfall. Surface water runoff from above cut slopes should be prevented from flowing over the slope face by using curbs, berms, drainage ditches, swales or other appropriate methods.

If temporary cut slopes experience excessive sloughing or raveling during construction, it may become necessary to modify the cut slopes to maintain safe working conditions and protect adjacent facilities or structures. Slopes experiencing problems can be flattened or additional dewatering can be provided if the poor slope performance is related to groundwater seepage.

Shored Excavations

Because of the diversity of available shoring systems and construction techniques, the design of temporary shoring is most appropriately left up to the contractor proposing to complete the installation. However, we recommend that the shoring be designed by an engineer licensed in Washington, and that PE stamped shoring plans and calculations be submitted prior to construction. The following paragraphs present recommendations for the type of shoring system and design parameters that we conclude are appropriate for the subsurface conditions at the project.

The majority of the soils within the depths of the proposed excavations consist of medium dense fill and outwash deposits. Trenches excavated in these soils can be retained using conventional trench boxes or sheet piles with appropriate internal bracing. We recommend that the excavation for the below-grade structures be supported with sheet pile shoring.

The design of temporary shoring should allow for lateral pressures exerted by the adjacent soil, and surcharge loads due to traffic, construction equipment, temporary stockpiles adjacent to the excavation, or other surcharge conditions if present. Lateral load resistance can be mobilized through the use of internal braces, tiebacks, anchor blocks and passive pressures on shoring members that extend below the bottom of the excavation.

Temporary shoring of excavations using internal bracing above groundwater level can be designed using active soil pressures. We recommend that this temporary shoring be designed using a lateral pressure equal to an equivalent fluid density of 35 pcf (pounds per cubic foot), for conditions with horizontal backfill adjacent to the excavation. If the ground within 5 feet of the excavation rises at an inclination of 1½H:1V or steeper, the shoring should be designed using an equivalent fluid density of 75 pcf. For adjacent slopes flatter than 1½H:1V, soil pressures can be interpolated between this range of values. Other conditions, such as surcharges from adjacent facilities, should be evaluated on a case by case basis.

The lateral pressure for design of the sheet pile shoring that extends below groundwater will depend on the method of dewatering used by the contractor. If dewatering is accomplished around the exterior of the shored excavation, we recommend that the shoring be designed using a lateral pressure equal to an equivalent fluid density of 35 pcf. If the ground around the excavation is inclined, the lateral pressure should be increased, as discussed above. If dewatering is accomplished within the interior of the shored excavation, we recommend that the shoring below the groundwater depth be designed for both soil and full hydrostatic pressures. For this case, the shoring below the groundwater level should be designed using a lateral pressure equal to an equivalent fluid density of 80 pcf. We recommend assuming that groundwater is approximately 4 feet below the ground surface.

The passive soil resistance acting on the embedded portion of the shoring can be evaluated using a lateral pressure equal to an equivalent fluid density of 150 pcf. If portions of the shoring use passive elements such as anchor or reaction blocks, available soil resistance can be estimated using passive soil pressures assuming an equivalent fluid density of 300 pcf above the water table and 150 pcf below the water table. The passive soil pressures presented above include a factor of safety of 1.5.

These lateral soil pressures do not include traffic or construction surcharges. Surcharge loads should be added to the above values where appropriate. If soft or loose soil conditions are encountered at the base of the excavation, the shoring should extend sufficiently below the bottom of the excavation to prevent base failure of the excavation. If saturated sands extend to below the base of the excavation, groundwater levels outside the excavation should be lowered by pumping to reduce the potential for base failure within the excavation. It is often impractical to extend the shoring to a suitable depth to reduce the potential for base failure.

GeoEngineers will be available to consult with the project team and the shoring designer, and to review the shoring and dewatering plans prior to construction. This will allow us to evaluate if the designs are consistent with the intent of our recommendations, and to provide supplemental recommendations if needed.

Excavation Dewatering

Groundwater was encountered at depths ranging from 4 to 11 feet below the ground surface at the site. Therefore, some form of groundwater control will be required for some of the excavations at the wastewater treatment plant site.

We recommend that excavations be dewatered to at least 2 feet below the bottom of the excavation. This may require the use of deep wells. The dewatering at the site must remain in operation until such time that the designer determines that positive buoyancy of the structure has been achieved.

Dewatering can be accomplished from outside of the sheet pile shoring or from within the shoring. As discussed above, dewatering from outside of the shoring will lower the pressures on the shoring and may reduce the overall cost of the shoring system. However, dewatering from outside of the shoring may not completely lower the water within the shoring and additional internal dewatering may be necessary.

Regardless of whether dewatering is accomplished from outside of the sheet pile shoring or from within the shoring, we recommend that a number of observation wells be established both outside and inside the shoring to monitor the groundwater levels.

In our opinion, the contractor should be responsible for designing and installing the appropriate dewatering system needed to complete the work. Appropriate discharge points should be designated by the contractor. Also, the contractor will need to obtain the necessary discharge permits from regulatory agencies. We recommend that details of the dewatering system be reviewed by GeoEngineers prior to construction.

Shoring and Dewatering Construction Considerations

Cobbles or boulders were not observed in the borings completed for this project; however, cobbles and boulders are known to exist in alluvial formations and recessional outwash. If such oversized material exists below the site, this material may obstruct the sheet pile installation. The shoring contractor should make provisions to realign the sheeting, and/or penetrate through obstructions as necessary to complete the installation.

In conjunction with the shoring and dewatering, a monitoring program should be performed by the contractor to determine the effects of the construction on the adjacent treatment plant structures and nearby roadway embankments. As a minimum, we recommend that horizontal and vertical surveying points be established on the shoring, a distance of 10 feet outside the shoring, and on the adjacent structures. The survey points on the existing structures should be read prior to the start of the construction activities, and the survey points on and outside the shoring should be read prior to excavating any materials. When the dewatering and/or excavation begin, surveys should be made at least weekly until the excavation backfilling operations begin. More frequent readings are recommended during critical construction activities, or if significant movements are noted.

Extraction of the shoring following completion of the backfill placement often leads to localized settlement of the adjacent ground. Consideration should be given to delaying placement of the upper 2 or 3 feet of backfill in the shoring area until after the shoring is extracted. This will allow for a greater degree of compaction in the surficial areas and the opportunity to work soils into the voids left when the sheets are extracted.

Contractor Responsibility

All temporary cut slopes and shoring must comply with the provisions of Title 296 WAC, Part N, "Excavation, Trenching and Shoring." The contractor performing the work must have the primary responsibility for protection of workmen and adjacent improvements, deciding whether or not to use shoring, and for establishing the safe inclination for open-cut slopes.

These discussions regarding excavation shoring and dewatering are presented to provide the project owner and designers with general information regarding the type of shoring and dewatering that may be necessary, and the level-of-effort necessary to construct these facilities. This information is not intended to provide final design parameters for the contractor who will design and install these construction facilities.

It is the contractor's responsibility to review the factual data presented in this report, perform any additional investigations and tests deemed necessary to characterize the soil and groundwater conditions at this site, and to develop independent conclusions regarding the design, construction, and operation of the shoring and dewatering systems.

EARTHWORK

Clearing and Site Preparation

We recommend removing and wasting the existing asphalt pavement, vegetation and other debris located within the excavation areas for the structures or in the locations of new facilities. This material should not be incorporated into fill or backfill placed at the project.

Reuse of On-Site Soils

The surficial fill soils on the site consist of silty sand, gravel with silt, and sand with silt. These soils contain moderate to significant fines (that portion passing the U.S. No. 200 sieve) and are therefore moisture sensitive. As the amount of fines increases, soil becomes increasingly more sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve. Based on the moisture content data, it is our opinion that these soils will be suitable for reuse during the dry summer months. However, the on-site soils are not suitable for use as structural fill during wet weather construction. We recommend clean import sand and gravel (containing less than 5 percent fines) be included in the construction budget for earthwork and backfill activities that occur during the winter season.

Subgrade Preparation

We recommend that new footings be supported on a minimum 2-foot zone of compacted structural fill, and new floor slabs and mat foundations be supported on a minimum 18 inches of structural fill. The upper 6-to 12-inches beneath slabs-on-grade and mat foundations should consist of a crushed rock material as described in subsequent sections. Detailed recommendations for foundation support are provided in a following section.

Structural Fill Material Quality

All fill and backfill used on the project should be placed as structural fill. Structural fill material should be free of debris, organic contaminants or rock fragments larger than 6 inches. We recommend using material such as crushed rock or pit run sand and gravel.

The workability of material for use as structural fill will depend on the gradation and moisture content of the soil. If structural fill is imported during wet weather we recommend that imported structural fill material have no more than 5 percent fines. If the material is too wet when delivered to the site, or if it becomes overly wet from rain, it must be aerated and dried out prior to placement as fill. If prolonged dry weather prevails during the construction period, the on-site soils or an imported structural fill with a somewhat higher fines content will be workable.

Fill Placement

Unless specified otherwise in this report, the following general requirements apply to all fill and backfill placement.

1. All structural fill must be placed in thin lifts so that uniform compaction can be achieved throughout the entire lift thickness. In general, granular soils with less than 5 percent fines can be placed in lifts of about 12 inches or less (loose thickness). A soil with higher fines content will necessitate thinner lifts. Each lift must be compacted prior to placing the subsequent lift.
2. All structural fill must be compacted to at least 90 percent of the maximum dry density determined by the ASTM D1557 test procedure. Where structures will be supported on the backfill, all of the fill must be compacted to at least 95 percent of the maximum dry density (ASTM D1557). If pavements will be supported by the backfill, the uppermost 24 inches of subgrade soils below the pavements must also be compacted to at least 95 percent of the maximum dry density (ASTM D1557).
3. Prior to compaction, the material should be moisture conditioned to within about 3 percent of optimum moisture content.
4. Compaction must be achieved by mechanical means. No jetting, ponding, or flooding will be allowed for compaction.

During structural fill placement, a suitable number of in-place density tests should be performed concurrently with the filling to check that the required compaction is being achieved. The frequency of tests will be dependant on the total fill thickness placed, fill content and performance, and season of construction.

Permanent Slopes

We recommend that all permanent cut and fill slopes be constructed no steeper than 2H:1V. To achieve uniform compaction, we recommend that fill slopes be overbuilt slightly and subsequently cut back to expose well compacted fill.

To minimize erosion, newly constructed slopes should be planted or hydroseeded shortly after completion of grading. Until the vegetation is established, some sloughing and ravelling of the slopes should be expected. This may require localized repairs and reseeding. Temporary covering, such as clear heavy plastic sheeting, jute fabric, loose straw or excelsior matting could be used to protect the slopes during periods of rainfall.

FOUNDATION SUPPORT

General

We recommend that the proposed new buildings and building additions be supported on shallow or mat foundations overlying a zone of compacted structural fill. Where existing loose fill soils are present at footing subgrade level, we recommend excavation and replacement to provide a minimum of 2 feet below the footing subgrade elevation. If excessively soft soils are exposed at subgrade level, additional excavation and replacement with structural fill will be necessary. The structural fill zone should form a prism below each footing which extends a minimum distance of 2 feet beyond the edges of the footing. We recommend that a representative from our firm evaluate the soils exposed in the footing excavations before placement of structural fill to determine that the footing subgrade is acceptable. Additional recommendations for spread footings and mat foundations are presented below.

Spread Footings

We recommend that isolated spread footings be a minimum of 24 inches wide and that continuous strip footings be a minimum of 16 inches wide. Exterior footings should be founded at least 18 inches below the lowest adjacent finished grade. Interior footings should be founded a minimum of 12 inches below lowest adjacent soil grade.

Footings can be designed for an allowable bearing capacity of 3,000 psf (pounds per square foot) for the combination of dead and long-term live loads. This allowable bearing capacity may be increased by one-third to account for short-term live loads such as induced by wind or seismic forces.

We recommend that all completed footing excavations be observed by a representative of our firm prior to reinforcing steel and structural concrete placement. Our representative will confirm that the bearing surface has been prepared in a manner consistent with our recommendations and that the subsurface conditions are as expected.

We estimate that the postconstruction settlement of structures supported on shallow foundations over 2 feet of compacted structural fill may be on the order of 1 inch. Differential settlements measured over a distance of approximately 25 feet may be on the order of ½ inch. We expect that settlements for these conditions will tend to occur rapidly after the loads are applied.

Immediately prior to placing concrete, all debris and soil slough that accumulated in the footings during forming and reinforcing steel placement must be removed. Debris or loose soil not removed from the footing excavations will result in increased settlement.

Mat Foundations

Concrete structural mat foundations may have flat bottoms or may be thickened below the perimeter and interior walls or areas of concentrated loading. We recommend that the bottom of the mat around the perimeter of the structure be founded at least 18 inches below the adjacent finished grade. A minimum 12-inch thickness of crushed rock subbase should underlie the mat foundation. The crushed rock should contain a minimum of 25 percent retained on the U.S. No. 4 sieve and no more than 3 percent passing the U.S. No. 200 sieve. The structural mat foundation can be evaluated assuming a subgrade modulus of 200 pounds per cubic inch (pci). Local bearing pressures below concentrated loads can be evaluated assuming an allowable soil bearing pressure of 3,000 pounds per square foot (psf). This bearing value considers combined dead and long-term live loads, and may be increased by up to one-third to account for short-term live loads such as wind or seismic forces.

We recommend that a representative of GeoEngineers observe the final subgrade below structural mat foundations to evaluate if the subgrade conditions are as expected, and to provide recommendations for design changes should the conditions encountered during construction differ from those anticipated.

We estimate postconstruction foundation settlements of ½ to 1 inch for the assumed loading conditions. Differential settlements across the mat foundation should be less than ½ inch in 25 feet assuming the mat is designed to distribute relatively uniform areal loading.

SLAB SUPPORT

All slab subgrade areas should be excavated and prepared as recommended in the *Site Preparation* section of the report before placing any fill. The upper 6 inches of fill placed to form the slab subgrade should consist of ¾-inch minus free draining sand and gravel with a minimum of 25 percent retained on the U.S. No. 4 sieve and no more than 3 percent passing the U.S. No. 200 sieve. If fill placement and slab construction will proceed during extended periods of dry weather, the imported fill placed below the upper 6 inches of slab subgrade may contain an increased percentage of fines, provided the fill can be compacted as recommended in this report.

A vapor barrier should be utilized where control of moisture in the slab is critical (e.g., where an adhesive is used for tiled or carpeted floors). We recommend a vapor barrier consisting of polyethylene sheeting with bonded seams.

SUBSURFACE WALLS

Below-grade walls of the new improvements will likely be designed for restrained conditions and should be designed for at-rest earth pressures. We recommend the buried walls be designed for an equivalent fluid pressure of 55 pcf. This assumes that the below grade portions of the walls have adequate wall drainage and a footing drain at the base of the walls. The equivalent fluid density should be increased by 15 pcf for a 2H:1V sloping backfill and 22 pcf for a 1½H:1V sloping backfill. If drainage is not provided behind the walls, full hydrostatic pressures should be included in the design. For this condition, we recommend using a design lateral pressure based on an equivalent fluid density of 85 pcf. Surcharge loads should also be applied as appropriate. We recommend a uniform surcharge of 250 psf be utilized where vehicle traffic will be adjacent to buried walls.

Backfill behind walls should be compacted to between 90 and 92 percent of ASTM D-1557. Measures should be taken to prevent overcompaction of the backfill against the wall (e.g. use lighter compaction equipment and thinner lifts).

The recommended equivalent fluid density assumes a free-draining condition behind the wall. If any walls are placed where water may have access to the fill behind, drainage should be provided by placing an 18- to 24-inch-wide zone of sand and gravel containing less than five percent fines against the wall. A perforated drainpipe should be embedded in the free-draining sand and gravel zone along the base of retaining walls to remove any water which collects in this zone. The drainpipe should be tightlined to an appropriate discharge point.

LATERAL RESISTANCE

Lateral loads may be resisted by passive resistance on the sides of buried foundation elements and by friction on the base. For foundations supported in accordance with our recommendations, the allowable frictional resistance may be computed using a coefficient of friction of 0.4 applied to vertical dead-load forces.

The allowable passive resistance may be computed using an equivalent fluid density of 300 pounds pcf if medium dense native soils or structural fill extends out from the face of the foundation element for a distance at least equal to three times the height of the element. The above coefficient of friction and passive equivalent fluid density values can be combined and include a factor of safety of about 1.5. The allowable passive and friction values can be increased by one-third for wind and seismic loads.

BUOYANCY DESIGN

Buoyancy uplift should be evaluated in design of new structures that will extend below groundwater level. We recommend that the buoyancy of the structure be evaluated on the basis of the dead weight of the structure alone without any soil resistance to buoyancy uplift. The soil resistance should be excluded because of the possibility that the backfill around the below-grade structure could be effected by seismically induced liquefaction.

SEISMIC DESIGN CONSIDERATIONS

Seismicity

The site is located within the Puget Sound region, which is seismically active. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca and North American plates. The Juan de Fuca plate is subducting beneath the North American plate. It is thought that the resulting deformation and breakup of the Juan de Fuca plate might account for the deep focus earthquakes in the region. Hundreds of earthquakes have been recorded in the Puget Sound area. In recent history, four of these earthquakes were large events: (1) in 1946, a Richter magnitude 7.2 earthquake occurred in the Vancouver Island, British Columbia area; (2) in 1949, a Richter magnitude 7.1 earthquake occurred in the Olympia area; (3) in 1965, a Richter magnitude 6.5 earthquake occurred between Seattle and Tacoma; and (4) recently in 2001, a Richter magnitude 6.8 occurred near Olympia.

Research is presently underway regarding historical large magnitude subduction-related earthquake activity along the Washington and Oregon coasts. Geologists are reporting evidence that suggests several large magnitude earthquakes (Richter magnitude 8 to 9) have occurred in the last 1,500 years, the most recent of which occurred about 300 years ago. No earthquakes of this magnitude have been documented during the recorded history of the Pacific Northwest. Local design practice in Puget Sound and local building codes are beginning to consider the possible effect of a very large subduction earthquake in the design of structures.

Site Class and Spectral Response

We recommend the project site be classified as Site Class D as defined in 2006 IBC. The parameters for the 2006 IBC are summarized in the following table:

Spectral Response Accelerations

(SRA) and Site Coefficients	Short Period	1 Second Period
Mapped SRA	$S_S = 1.05$	$S_1 = 0.36$
Site Coefficients	$F_a = 1.08$	$F_v = 1.68$
Max. Considered Earthquake SRA	$S_{MS} = 1.13$	$S_{M1} = 0.60$
Design SRA	$S_{DS} = 0.75$	$S_{D1} = 0.40$

Note: 1) Soil Profile for Site Class D: Stiff Soil Profile

Liquefaction Potential

Liquefaction refers to the condition where vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent loss of strength in the deposit of soil so affected. In general, soils that are susceptible to liquefaction include very loose to medium dense, clean to silty sands that are below the water table. Our analysis indicates that the majority of the outwash deposits that underlie the site area have a low risk of liquefying under a magnitude 7.5 design earthquake. Ground surface settlement due to liquefaction is estimated to be less than 1 inch.

Lateral Spreading

Lateral spreading occurs when loose wet soils liquefy and move horizontally, either toward an open face (such as a riverbank) or along a gradual slope. We have used an empirical model to predict free-field ground displacements that might be associated with lateral spreading at the site. The empirical model incorporates earthquake, geological, topographical and soil factors that affect ground displacement. The model was developed from compiled data collected at sites where lateral spreading was observed. The key parameters are the earthquake magnitude and distance, the thickness of the liquefied zone, the grain size distribution of the liquefied deposit, and the ratio of the free face height to the distance between the point of measurement and the toe of the free face. The results of our analysis indicate that the site soils and geometry at this site have the potential to generate relatively small lateral displacements during an earthquake. We estimate that free-field lateral displacements at the site may be on the order of ½ foot.

Other Considerations

We strongly recommend that all connections to the building be flexible to allow differential movements to occur between the building and the adjacent ground during and after an earthquake. In particular, it is important that gas, sewer and water lines be fitted with flexible connections at the building. We also recommend that automatic shut-off values, triggered by seismic accelerations, be installed in gas and water lines leading to the building.

DRAINAGE CONSIDERATIONS

We recommend that exterior surfaces be sloped so that surface drainage flows away from the structures. Because of the relatively high seasonal groundwater in portions of the site, surface water may tend to pond in low-lying areas during the winter season. Grading in all areas should be accomplished to avoid concentration of runoff onto fill, cut slopes, natural slopes steeper than 10 percent or other erosion-sensitive areas.

A perimeter footing drain system should be incorporated into the design of new buildings. We recommend that the drainage system consist of a zone of free-draining backfill against the side of the footing surrounding a perforated pipe placed a minimum of 12 inches below the adjacent slab elevation. The free-draining backfill should consist of sand or sand and gravel containing no more than 3 percent passing the U.S. No. 200 sieve based on the 3/4-inch fraction. The perforated drainpipe should have a minimum diameter of 4 inches and should be rigid PVC, not the flexible ADS variety. The pipe should be sloped to drain by gravity and routed to a suitable discharge point so that water discharged from the pipe does not cause erosion. Roof downspout lines must not be tied into the perimeter footing drain system.

PAVEMENT RECOMMENDATIONS

Pavement subgrade areas should be stripped and proofrolled, or otherwise evaluated, as recommended in the *Site Preparation* section of the report. Where the existing soils are loose or wet and cannot be compacted, it will be necessary to excavate and replace these soils. We recommend the following minimum pavement sections for the project:

Pavement Section	Asphalt Surfacing Thickness (inches)[1]	Crushed Rock Base Course (inches)[2]
Light Traffic Areas	3	6
Repeated Truck Traffic and Areas of Heavy Turning	4	6

[1] We recommend asphalt surfacing consist of ½-inch HMA (PG 58-22) in accordance with WSDOT Sections 5-04 and 9-03.

[2] Crushed rock base course should meet WSDOT specification 9-03.9(3). We recommend the subgrade be evaluated to confirm soil conditions are as anticipated prior to placement of the crushed rock. The above pavement recommendations assume placement of structural fill as previously recommended and an R value of 45 (CBR of approximately 15).

UTILITY TRENCHES

We recommend trench excavation, pipe bedding, and trench backfilling be completed using the general procedures described in Division 7 of the 2008 WSDOT Standard Specifications or other suitable procedures specified by the project civil engineer. The native deposits and fill soils encountered at the site are generally of low corrosivity based on our experience in similar soil conditions.

Prior to the installation of the pipe, the pipe bedding should be shaped to fit the lower part of the pipe exterior with reasonable closeness to provide continuous support along the pipe. Pipe bedding material should be placed in layers and tamped around the pipe to obtain complete contact. In areas where a trench box is used, the bedding material should be placed before the trench box is advanced.

Utility trench backfill should consist of structural fill and should be placed in lifts of 8 inches or less (loose thickness) such that adequate compaction can be achieved throughout the lift. Sand backfill, containing less than 5 percent fines, may be compacted in loose lifts not exceeding 12 inches when placed below five feet of the finished ground surface. Each lift must be compacted prior to placing the subsequent lift. Prior to compaction, the backfill should be moisture conditioned to within 3 percent of the optimum moisture content, if necessary. The backfill should be compacted to the minimum criteria discussed in the structural fill section of this report.

EROSION CONTROL MEASURES

Potential sources or causes of erosion and sedimentation depend upon construction methods, slope length and gradient, amount of soil exposed and/or disturbed, soil type, construction sequencing and weather. The project impact on erosion-prone areas can be reduced by implementing an erosion and sedimentation control plan. We recommend the plan incorporate basic planning principles including:

- Scheduling grading and construction to minimize soil exposure,
- Retaining existing vegetation whenever feasible,
- Revegetating or mulching denuded areas,
- Directing runoff away from denuded areas,
- Minimizing 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, and
- Inspecting and maintaining control measures frequently.

Temporary erosion protection should be used and maintained in areas with exposed or disturbed soils to help minimize erosion and reduce transport of sediment to adjacent areas and receiving waters. Permanent erosion protection should be provided by re-establishing vegetation by hydroseeding or landscape planting.

Until the permanent erosion protection is established and the site is stabilized, site monitoring should be performed by qualified personnel to evaluate the effectiveness of the erosion control measures and 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.

LIMITATIONS

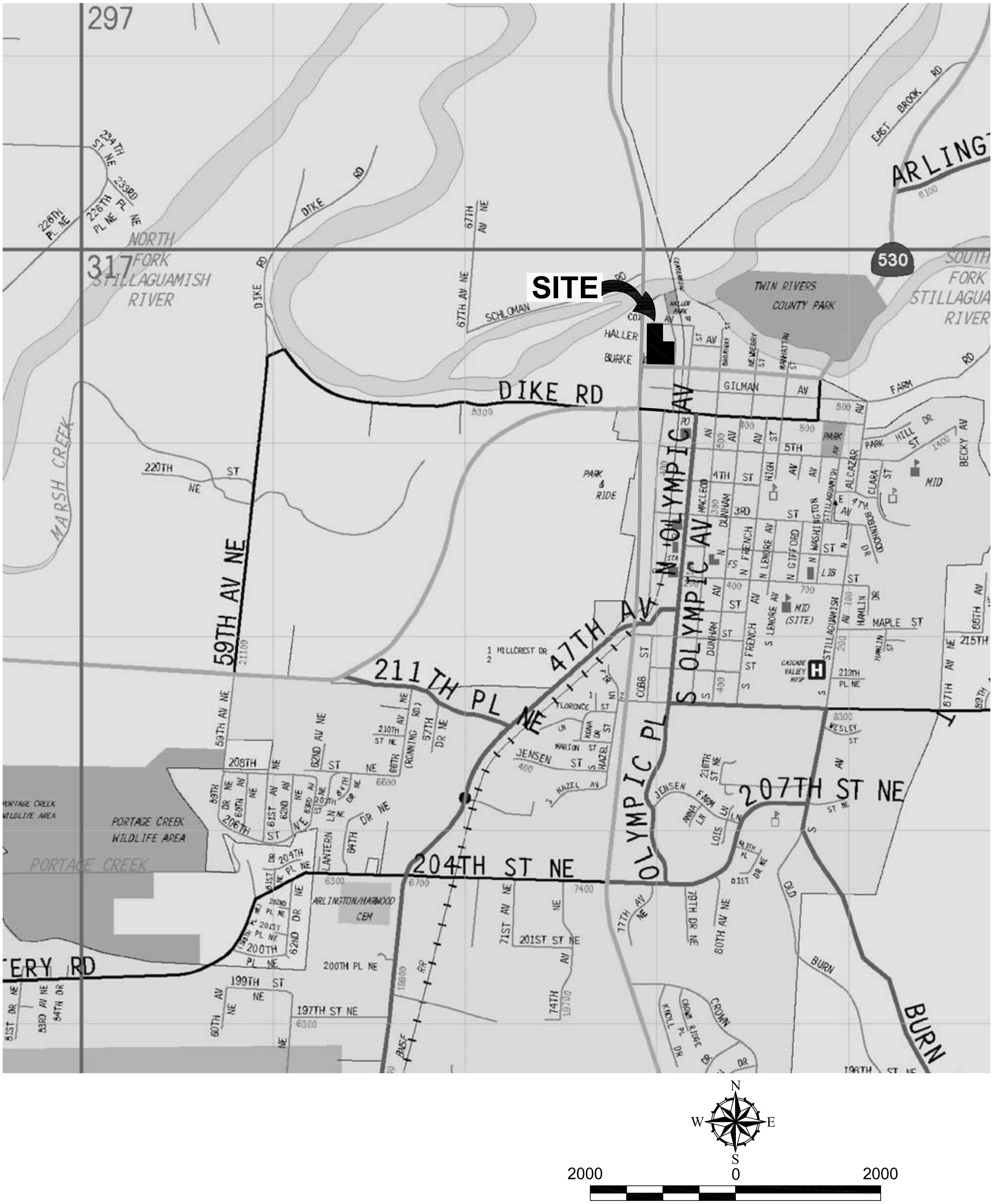
We have prepared this report for use by City of Arlington and members of the design team for use in design of the MBR Upgrade and Expansion project at the WWTP in Arlington, Washington.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted geotechnical practices in this area at the time the report was prepared. No warranty or other conditions, express or implied, should be understood.

Please refer to the appendix titled Report Limitations and Guidelines for Use for additional information pertaining to use of this report.

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

We appreciate this opportunity to be of service to City of Arlington and Kennedy/Jenks Consultants on this project. Please call if you have any questions regarding this report or we can provide additional assistance.

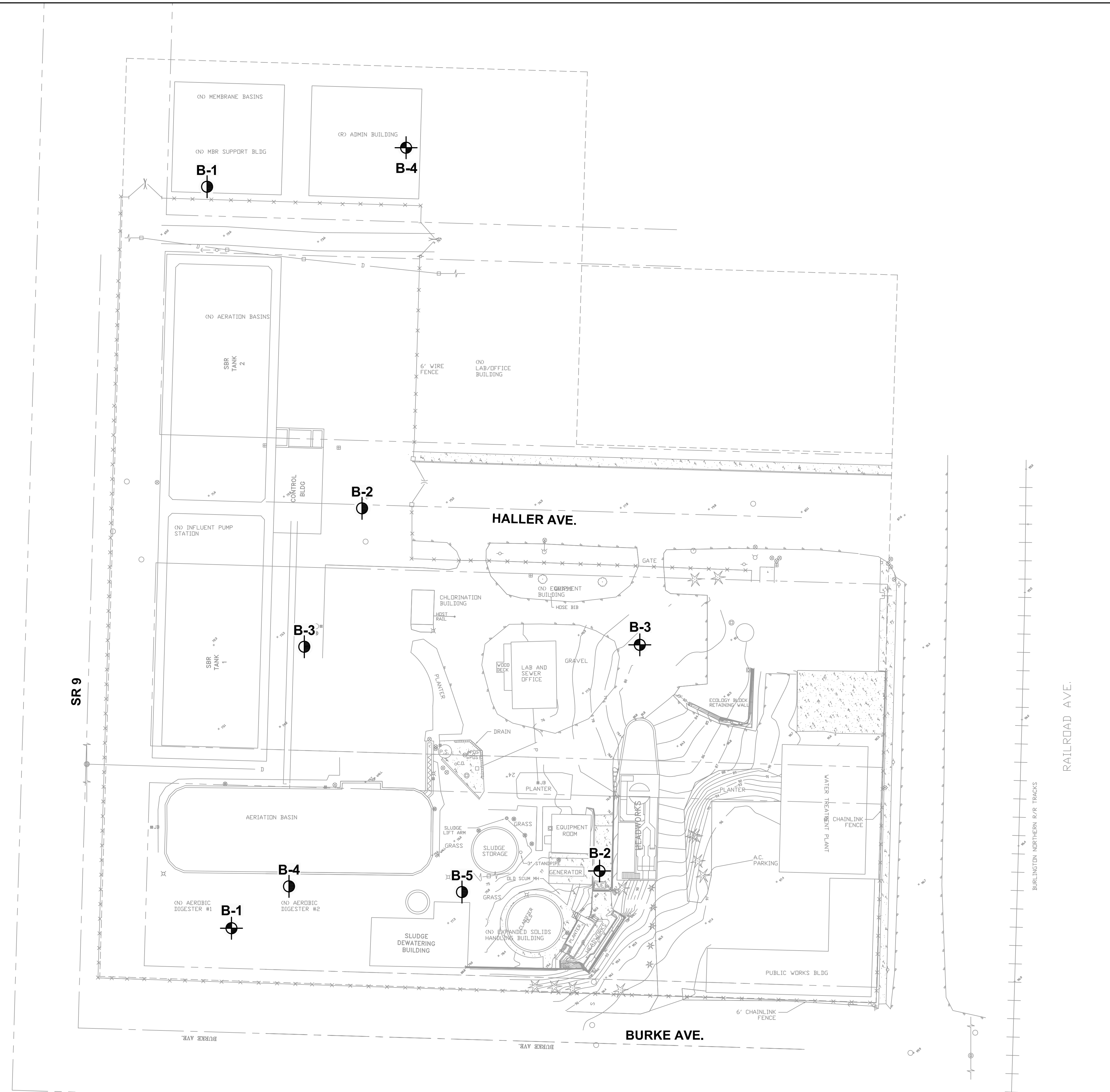


Vicinity Map

City of Arlington
WWTP Improvements

GEOENGINEERS

Figure 1



LEGEND:

- **B-1** Boring (GeoEngineers)
- **B-1** Boring by Others

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. can not 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.

Reference: CAD file entitled "X-SITE.dwg" provided by Kennedy/Jenks on 8/31/07.

N W E S

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FEET

Site Plan

City of Arlington
WWTP Improvements

GEOENGINEERS

Figure 2

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APPENDIX A

FIELD EXPLORATION AND LABORATORY TESTING

APPENDIX A
FIELD EXPLORATION AND LABORATORY TESTING

EXPLORATION PROGRAM

Subsurface soil and groundwater conditions were evaluated by drilling four borings at the approximate locations shown in Figure 2. The borings were completed to depths of 18.5 to 19 feet below the existing ground surface. The exploration locations were field located by taping and pacing from existing site features.

The explorations were continuously monitored by an engineering geologist from our firm who examined and classified the soils encountered, obtained representative soil samples, observed groundwater conditions, and prepared a detailed log of each exploration. Soils were visually classified in general accordance with ASTM D 2488-90, which is described in Figure A-1. An explanation of our boring log symbols is also shown in Figure A-1.

The samples were obtained using a Dames & Moore sampler driven into the soil with a 300-pound hammer and an SPT sampler driven into the soil with a 140-pound hammer. The number of blows required to drive the sampler the last 12 inches or other indicated distances are recorded on the boring logs. The logs of the borings are presented in Figures A-2 through A-5. The exploration logs are based on our interpretation of the field and laboratory data and indicate the various types of soils encountered. They also indicate the depths at which these soils or their characteristics change; although the change might actually be gradual.

LABORATORY TESTING

All soil samples were brought to our laboratory for further examination. Selected samples were tested to determine their moisture content and grain size characteristics. The results of the moisture content and percent fines tests are presented on the logs. The results of the sieve analyses results are presented on Figure A-6.

SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Measured groundwater level in exploration, well, or piezometer

Groundwater observed at time of exploration

Perched water observed at time of exploration

Measured free product in well or piezometer

Stratigraphic Contact

Distinct contact between soil strata or geologic units

Gradual change between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

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

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

Laboratory / Field Tests

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

Sheen Classification

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

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

KEY TO EXPLORATION LOGS

GeoEngineers

FIGURE A-1

Date(s) Drilled	07/17/07	Logged By	BHC	Checked By	DCO
Drilling Contractor	Boart Longyear	Drilling Method	Hollow-stem Auger	Sampling Methods	D&M
Auger Data	4" ID	Hammer Data	300 lb hammer/30 in drop Rope and Cathead	Drilling Equipment	B-59 Truck
Total Depth (ft)	19	Surface Elevation (ft)		Groundwater Level (ft. bgs)	10
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES: Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
0						AC	4 inches asphalt concrete			
						CC	2 inches concrete			
						CR	6 inches crushed rock			
						SM	Gray silty fine to medium sand with coarse sand and gravel (medium dense, moist) (fill)			
		18	17	1				13	104	
						GM	Brown and gray coarse gravel with silt and sand (medium dense to dense, moist)			
5		18	19	2						
		18	58	3						Gravel in shoe Blow count overstated
10					▼	SP-SM	Gray and brown medium to coarse sand with silt and gravel (dense, wet)			
		18	32	4				12		SA
15						GP	Grayish brown coarse gravel with sand and trace silt (dense, wet)			
		11	46	5						
20										

Note: See Figure A-1 for explanation of symbols.

LOG OF BORING B-1

GEOENGINEERS	Project:	City of Arlington WWTP Improvements
	Project Location:	Arlington, Washington
	Project Number:	5430-004-00 Task 100

Figure A-2
Sheet 1 of 1

Date(s) Drilled	08/22/07	Logged By	ABA	Checked By	DCO
Drilling Contractor	Geologic Drill	Drilling Method	Hollow-stem Auger	Sampling Methods	SPT
Auger Data	2¼" ID, 4¼" OD	Hammer Data	140 lb hammer/30 in drop automatic	Drilling Equipment	Limited Access Acker Rig
Total Depth (ft)	18.5	Surface Elevation (ft)		Groundwater Level (ft. bgs)	4
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES: Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
0							AC	3 inches asphalt concrete			
							CR	6 inches crushed rock			
							SM	Tan with orange silty fine to medium sand with gravel (medium dense, moist) (fill)			
		14	18	1					15		
					▼						
5		18	12	2			SM	Tan-gray silty very fine sand (medium dense, wet) (fill?)			
		18	12	3			ML	Grayish brown silt with sand and occasional gravel (stiff, moist)	23		
10		18	12	4							
							SM	Gray silty fine to medium sand with gravel (dense to very dense, moist to wet)			
15		3	74	5							Blow counts may be overstated due to gravel
		8	50/5"	6							
20											

Note: See Figure A-1 for explanation of symbols.

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LOG OF BORING B-2

GEOENGINEERS

Project: City of Arlington WWTP Improvements
Project Location: Arlington, Washington
Project Number: 5430-004-00 Task 100

Date(s) Drilled	07/17/07	Logged By	BHC	Checked By	DCO
Drilling Contractor	Boart Longyear	Drilling Method	Hollow-stem Auger	Sampling Methods	D&M
Auger Data	4" ID	Hammer Data	300 lb hammer/30 in drop Rope and Cathead	Drilling Equipment	B-59 Truck
Total Depth (ft)	18.6	Surface Elevation (ft)		Groundwater Level (ft. bgs)	10
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES: Recovered (in)	Blows/foot	Sample Number	Water Level	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
0					AC	3 inches asphalt concrete			
					CR	8 inches crushed rocked base course			
					GP-GM	Brown coarse gravel with silt and sand (medium dense, moist) (fill)			
	5	12	1						
5					SM	Dark brown silty fine sand with organic matter (loose, moist)			
	6	7	2				34		
					ML	Brown silt with very fine sand and trace organic matter (medium stiff, moist)			
	6	6	3				27	95	
					SM	Gray silty fine sand (loose, moist to wet)			
10				▼					
					SP	Gray fine to coarse sand with gravel (medium dense to dense, wet)			
	6	20	4				13		
15									
	6	87/9"	5			Grades to very dense			
20									

Note: See Figure A-1 for explanation of symbols.

LOG OF BORING B-3

GeoEngineers

Project: City of Arlington WWTP Improvements

Project Location: Arlington, Washington

Project Number: 5430-004-00 Task 100

Date(s) Drilled	07/17/07	Logged By	BHC	Checked By	DCO
Drilling Contractor	Boart Longyear	Drilling Method	Hollow-stem Auger	Sampling Methods	D&M
Auger Data	4" ID	Hammer Data	300 lb hammer/30 in drop Rope and Cathead	Drilling Equipment	B-59 Truck
Total Depth (ft)	19	Surface Elevation (ft)		Groundwater Level (ft. bgs)	11
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
0							CR	10 inches crushed rock			
							SP-SM	Brown fine to coarse sand with silt and gravel (medium dense, moist) (fill)			
		6	15	1					5		SA
							SM	Brown silty fine to medium sand with gravel (dense to very dense, moist)			
5		6	84	2							
							GM	Brown-gray silty gravel with sand (dense to very dense, moist)			
		6	84	3							
10											
					▼						
		0	50/2"	4							
							SP-SM	Gray-brown fine sand with silt (dense, wet)			
15											
		6	37	5					20	108	
20											

Note: See Figure A-1 for explanation of symbols.

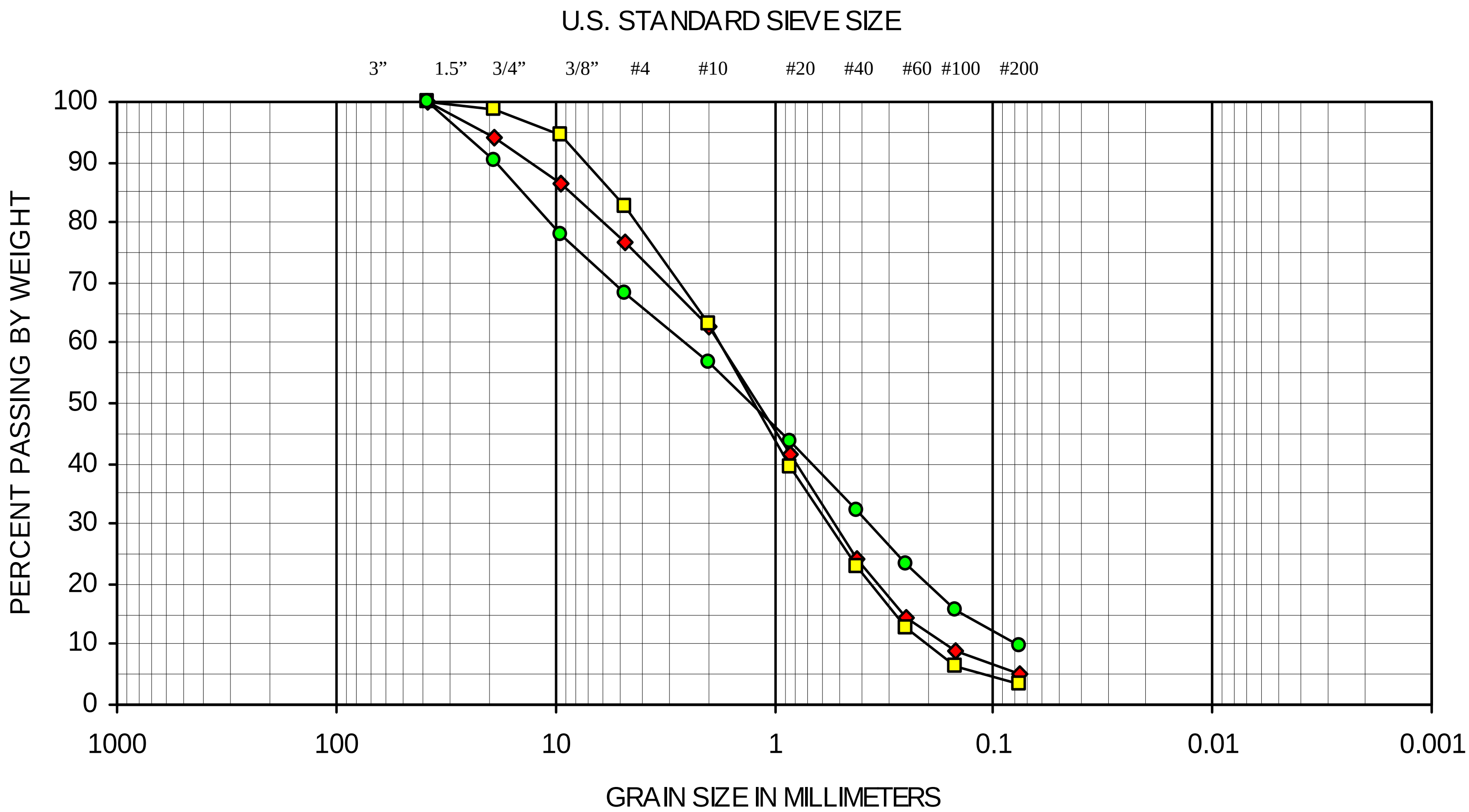
LOG OF BORING B-4

GEOENGINEERS

Project: City of Arlington WWTP Improvements
Project Location: Arlington, Washington
Project Number: 5430-004-00 Task 100

Figure A-5
Sheet 1 of 1

GEOENGINEERS



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	EXPLORATION NUMBER	DEPTH (ft)	SOIL CLASSIFICATION
◆	B-1	12.5	Poorly graded sand with silt and gravel (SP-SM)
□	B-3	12.5	Poorly graded sand with gravel (SP)
●	B-4	2.5	Poorly graded sand with silt and gravel (SP-SM)

SIEVE ANALYSIS RESULTS

FIGURE A-6

GEOENGINEERS

APPENDIX B

REPORT LIMITATIONS AND GUIDELINES FOR USE

APPENDIX B
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

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

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS AND PROJECTS

This report has been prepared for the exclusive use of City of Arlington and their authorized agents. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with whom there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with the Client and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

This report has been prepared for the proposed MBR improvements at the WWTP in Arlington, 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, do not 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;
- elevation, configuration, location, orientation or weight of the proposed structure;
- composition of the design team; or
- project ownership.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

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

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 manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

MOST 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 subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

GEOTECHNICAL ENGINEERING REPORT RECOMMENDATIONS ARE NOT FINAL

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A GEOTECHNICAL ENGINEERING OR GEOLOGIC REPORT COULD BE SUBJECT TO MISINTERPRETATION

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by 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. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

GIVE CONTRACTORS A COMPLETE REPORT AND GUIDANCE

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

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 to adjacent properties.

READ THESE PROVISIONS CLOSELY

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory “limitations” provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these “Report Limitations and Guidelines for Use” apply to your project or site.

GEOTECHNICAL, GEOLOGIC AND ENVIRONMENTAL REPORTS SHOULD NOT BE INTERCHANGED

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

BIOLOGICAL POLLUTANTS

GeoEngineers’ Scope of Work specifically excludes the investigation, detection, prevention, or assessment of the presence of biological pollutants in or around any structure. Accordingly, this report includes no interpretations, recommendations, findings, or conclusions for the purpose of detecting, preventing, assessing, or abating biological pollutants. The term “biological pollutants” includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and/or any of their byproducts.