Geotechnical Engineering Services

Facility Upgrades
Sumner Wastewater Treatment Plant
Sumner, Washington

for
Gray & Osborne, Inc.

April 18, 2013

GeoEngineers
Earth Science + Technology

Geotechnical Engineering Services

Facility Upgrades
Sumner Wastewater Treatment Plant
Sumner, Washington

for
Gray & Osborne, Inc.

April 18, 2013

GeoEngineers

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

Geotechnical Engineering Services

Facility Upgrades
Sumner Wastewater Treatment Plant
Sumner, Washington

File No. 0588-004-03

April 18, 2013

Prepared for:

Gray & Osborne, Inc.
701 Dexter Avenue North, Suite 200
Seattle, Washington 98109

Attention: Doug Welch, PE

Prepared by:

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

Craig A. Jordan
Staff Geotechnical Engineer

Morgan McArthur, PE
Geotechnical Engineer

Dennis (D.J.) Thompson, PE
Associate

MORGAN MCARTHUR
STATE OF WASHINGTON
46390
REGISTERED
PROFESSIONAL ENGINEER

4/18/13

CAJ:MM:DJT:tt

GEOENGINEERS

Table of Contents

LIST OF FIGURES

APPENDICES

1.0 INTRODUCTION AND PROJECT UNDERSTANDING

This report presents the results of our geotechnical engineering services for the proposed facility upgrades to the Sumner Wastewater Treatment Plant (WWTP) in Sumner, Washington. The project site is located near the confluence of the White and Puyallup Rivers, as shown in Figure 1. Our services are provided in accordance with our master subconsultant agreement dated June 20, 2003 and our supplemental signed agreement dated July 15, 2010. GeoEngineers previously provided geotechnical engineering services for a prior expansion of the WWTP. Those services were presented in our geotechnical report dated September 11, 2002.

Our understanding of the proposed facility upgrades are based on information provided by Gray & Osborne, Inc. (G&O) and a site walk with G&O on February 19, 2013. The information provided includes a site plan showing proposed upgrades, which includes a new secondary clarifier, aeration basin, gravity thickener, grit handling and odor control facility, solids storage building and vehicle maintenance building. Additional upgrades include a facility truck canopy and additions to Primary Clarifier No. 3. We understand that overall site grades will remain relatively unchanged. The proposed locations of the new facilities are shown in Figure 2.

Excavation for the proposed secondary clarifier is likely to extend about 30 feet below the existing ground surface (bgs). The excavation for the proposed aeration basin is expected to be about 20 feet bgs. Excavation for the gravity thickener facility is expected to be about 15 feet bgs. Excavations less than 10 feet deep are expected for the grit handling and odor control facility. A foundation/retaining wall about 5 to 6 feet tall will be required for construction of the solids storage building. All of the proposed structures are expected to bear on shallow foundations, mats or slabs-on-grade. Additions to Primary Clarifier No. 3 are anticipated to include the extension of the clarifier sidewalls to an elevation about 5 feet higher than the current walls. For the purposes of this report, we define at-grade structures to be those founded less than about 5 feet below existing grades. Structures extending more than 5 feet below existing grades are defined as below-grade structures.

2.0 SCOPE OF SERVICES

The purpose of our services is to complete a geotechnical evaluation that addresses specific elements of the proposed facility upgrades to the Sumner WWTP. The scope of services completed for this project includes the following tasks:

1. Review our files for geotechnical information in the site vicinity.
2. Explore subsurface soil and groundwater conditions at the site by drilling four borings to depths ranging between 21.5 to 51.5 feet bgs.
3. Perform sieve analysis, percent fines, and moisture content laboratory tests on representative soil samples.
4. Evaluate pertinent physical and engineering characteristics of the soils based on the results of our field exploration, laboratory testing and our experience.

5. Provide recommendations for earthwork and site preparation including suitability of on-site soils for reuse in fill, placement and compaction of fill, and mitigation of unsuitable soil conditions. This includes a discussion of the effects of weather and/or construction equipment on site soils.
6. Perform engineering analyses, and provide conclusions and recommendations for the following:
    - Foundation type for the structures and appropriate design capacities, bearing pressures, lateral resistance, and estimates of expected foundation settlement.
    - Lateral soil pressures on below-grade structures, including design seismic loads.
    - Geotechnical considerations for bearing surface preparation for mat foundations.
    - Geotechnical considerations for slab-on-grade construction.
    - Seismic design criteria based on the 2009 International Building Code (IBC).
    - Geotechnical considerations for shoring design.
    - Dewatering considerations.
7. Evaluate the liquefaction and lateral spreading potential of the soils at the site.
8. Provide consultation and attend meetings.

3.0 SITE CONDITIONS

3.1. General

The site is located to the east of the confluence of the Puyallup and White Rivers in Sumner, Washington. The WWTP currently contains a number of improvements, including administration and control buildings, anaerobic digesters, primary and secondary clarifiers, aeration basins, headworks components, a disinfection facility and other structures and pipes involved in the movement of wastewater through the facility. Underground and overhead utilities include sanitary and storm sewer, water, power and telephone.

3.2. Topography

The topography of the site gently slopes from the northeast to the southwest. The ground surface within the WWTP facility ranges from Elevation 43 to 56 feet. It appears that grading has taken place to level portions of the WWTP facility. A berm approximately 5 feet high has been constructed on the north, south and west sides of the plant. In addition to the berm a vinyl sheet pile wall has been constructed on the south and west sides of the plant. Vegetation at the site consists of coniferous and deciduous trees, small shrubs and landscaped grass area. Topographic data of the site was provided by G&O.

3.3. Mapped Geologic Conditions

Geologic conditions at the project site are shown on the *Geologic Map of the South Half of the Tacoma Quadrangle, Washington* (Walsh, 1987) and the *Geologic Map of the Puyallup 7.5-minute Quadrangle, Washington* (Troost and Booth, in review). Alluvium and mudflow deposits are mapped in the project vicinity. Alluvium typically consists of interbedded deposits of sand and fine-grained soils such as silt and clay, but may also include lake deposits and peat. Mudflow deposits are associated with lahars from Mt. Rainier, most notably the Osceola mudflow. Lahar deposits found at depth are described as comprising unsorted andesitic rock fragments in a clayey or silty sand matrix often with wood debris.

3.4. Previous Studies

As part of this study, we reviewed the following geotechnical reports prepared for the project site:

- Dames & Moore, 1971. "Report of Site Investigation, Proposed Sewage Treatment Plant, Sumner, Washington," May 19, 1971.
- Earth Consultants, Inc., 1981. "Geotechnical Engineering Study, Sumner Waste Water Treatment Plant Addition, Sumner, Washington," April 15, 1981.
- GeoEngineers, Inc., 2002. "Geotechnical Engineering Services, Sumner Wastewater Treatment Plant, Sumner, Washington," September 11, 2002.

4.0 SUBSURFACE CONDITIONS

4.1. Explorations

We explored subsurface conditions at the site by advancing four borings to depths between 21.5 and 51.5 feet bgs on February 25, 2013. We also reviewed explorations from the previous studies described above. Locations of the explorations are shown in Figure 2. Details of our explorations are presented in Appendix A.

4.2. Laboratory Testing

Soil samples obtained during our explorations were transported to GeoEngineers' laboratory for further examination and testing. Representative soil samples were selected for laboratory tests to evaluate the pertinent geotechnical engineering characteristics and to confirm or modify field classifications. Our geotechnical testing program included grain-size analyses, percent fines determination and moisture content determination. Details of our laboratory testing program are presented in Appendix B.

4.3. Subsurface Conditions

4.3.1. General

Based on our explorations and review of previous studies, we classify the soils encountered into the following general soil units: 1) fill, 2) upper alluvium, 3) mudflow deposits and 4) lower alluvium. These units are described in more detail below.

4.3.2. Fill

Fill was encountered in each of the four borings performed for this study. The fill observed in the borings generally consists of loose to medium dense sand and gravel with variable silt and organic content. Depths of the fill ranged from approximately 12.5 to 19 feet bgs in the borings completed for this study. The fill appears to be associated with site grading and backfilling around existing structures.

4.3.3. Upper Alluvium

Alluvium deposits were encountered below the fill in each of the four borings performed for this study. The upper alluvium deposits consist of loose to very dense sand and gravel with variable silt content. The upper alluvium deposit was observed to range from approximately 10 to 13.5 feet thick in the explorations completed for this study. Borings B-3 and B-4 were terminated in the upper alluvium material.

4.3.4. Mudflow Deposits

Mudflow deposits were encountered below the upper alluvium in borings B-1 and B-2 of this study. The mudflow deposits observed typically consists of very loose to medium dense silty sand and gravel. Borings B-1 and B-2 were terminated in the mudflow deposits. Explorations from previous studies penetrated through the mudflow deposits. These explorations indicate that the mudflow deposits range from about 35 to 42 feet in thickness.

4.3.5. Lower Alluvium

Explorations for previous studies indicate that the mudflow deposits are underlain by alluvium deposits consisting of medium dense to dense sand and gravel with variable silt content. We did not observe the lower alluvium unit in explorations performed for this study. Borings that were conducted for the Geotechnical Report dated September 11, 2002 and the Site Investigation report dated May 19, 1971 extended into the lower alluvium deposits and were terminated in these deposits.

4.4. Groundwater

At the time of our explorations, we observed groundwater at depths between about 12 feet and 17 feet bgs. We also measured the water depths in three existing groundwater monitoring wells at the time of our explorations. The water depths were measured to be about 12.5 feet bgs in well 1, 12.4 feet bgs in well 2, and 7.5 feet in well 3. Based on topography information provided by G&O, these depths correspond to Elevation 35 to 37.5 feet. We interpret this to be representative of the regional groundwater table at the time of our explorations.

Groundwater at the site is expected to vary depending on season, precipitation and other factors. Because the site is located immediately adjacent to the Puyallup and White Rivers, and the upper alluvium soil is relatively permeable, we expect groundwater levels to vary with the water level in the rivers. We understand that the 100-year flood elevation at the project site is at Elevation 48.5 feet, which is at or above the ground surface in a portion of the facility. If structures are to be designed for flood conditions, we recommend using a design groundwater level at Elevation 48.5, or at the ground surface, whichever is more conservative.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1. Earthwork

5.1.1. General

We anticipate that site development and earthwork will initially include clearing and stripping of surface vegetation, removing portions of the existing vinyl sheet pile wall, excavating for below-grade structures and utilities, and placing and compacting fill and backfill materials. We expect that the majority of site grading can be accomplished with conventional earthmoving equipment in proper working order. The following sections provide recommendations for earthwork, site development and fill materials.

5.1.2. Clearing, Stripping and Demolition

Based on our project understanding and site observations, we anticipate stripping, clearing and demolition will be required to remove the existing organic-rich sod, vegetation, and existing pavements from structural areas. We estimate that the depth of stripping will generally be on the order of 3 to 6 inches to remove the sod and organic-rich soil in the existing landscape areas. Greater stripping depths may be required to remove localized zones of loose or organic soil, or if stripping operations cause excessive disturbance to bearing surface soil. Material from the stripping operations should be disposed of off site or used for landscaping purposes, if practical.

Any remaining below-grade elements from previous site development should also be removed from structural areas. Abandoned, below-grade utilities should also be removed from structural areas; alternatively, below-grade utilities can be abandoned in place by completely filling the utilities with lean mix concrete or controlled density fill (CDF).

5.1.3. Excavations

5.1.3.1. TEMPORARY SLOPES

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, based on our observations and explorations, temporary cut slopes in on-site soils should be inclined no steeper than about 1½H:1V (horizontal:vertical). This guideline assumes that all surface loads are kept a minimum distance of at least one-half the slope height away from the top of the slope and that significant seepage is not present on the slope face. Flatter slopes will be necessary where significant seepage occurs, where soils are disturbed or if voids are created during excavation. Sloughing and raveling of temporary cut slopes should be expected. Temporary covering with heavy plastic sheeting should be used to protect slopes during periods of wet weather.

If 1½H:1V or flatter slopes are not feasible because of site constraints, temporary shoring could be required. Combinations of slopes and temporary shoring may also be considered. The following section discusses the use and design of temporary shoring.

5.1.3.2. TEMPORARY SHORING

5.1.3.2.1. GENERAL

Temporary shoring could be necessary for portions of the excavations for below-grade structures, to avoid disturbance of the existing ground around the excavation, preserve lateral support of existing facilities and to protect the personnel working within the excavation.

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 and stamped by a Professional Engineer (PE) licensed in Washington, and that shoring plans and calculations be submitted for review prior to construction. The following paragraphs present recommendations for the type of shoring system and design parameters appropriate for the subsurface conditions at the site.

We recommend GeoEngineers be retained 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 whether the designs are consistent with the intent of our recommendations and to provide supplemental recommendations in a timely manner.

5.1.3.2.2. GROUNDWATER CONSIDERATIONS FOR SHORING

Groundwater was observed between Elevations 35 and 37.5 during our site explorations. Therefore, we anticipate groundwater control will be required for some of the excavations at the WWTP site. The groundwater levels are expected to fluctuate as a function of season; therefore, less dewatering effort will likely be required during the drier summer and early fall months.

As discussed above, we anticipate that groundwater level will fluctuate at the site. Shoring design should consider appropriate groundwater levels, depending on the anticipated season of construction. If construction is expected to proceed in winter, it may be appropriate to assume that groundwater is at or near the ground surface. If summer construction is planned, a lower groundwater elevation may be appropriate. For final shoring design, we recommend the contractor confirm groundwater elevations at the excavation locations, and that groundwater elevations be monitored throughout construction.

We recommend that excavations be dewatered to at least 2 feet below the bottom of the excavation. This dewatering will likely necessitate the use of wells. The dewatering at the site must remain in operation until such time that the designer determines that the structures can resist buoyant and hydrostatic pressures.

Dewatering can be accomplished from outside of the excavation or from within the excavation. As discussed below, 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 excavation may not completely lower the water within the excavation and additional internal dewatering may be necessary.

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

We recommend the contractor be made 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 should obtain discharge permits from regulatory agencies, if necessary. We recommend that we be retained to review details of the dewatering system prior to construction. This will allow us to evaluate whether the designs are consistent with the intent of our recommendations, and to provide supplemental recommendations in a timely manner.

5.1.3.2.3. SHORING ABOVE GROUNDWATER

Trenches or excavations above the water table can be retained using conventional trench boxes or sheet piles with appropriate internal bracing. We recommend that temporary shoring above groundwater be designed using an active lateral pressure equal to an equivalent fluid density of 35 pounds per cubic foot (pcf), for conditions with horizontal backfill adjacent to the excavation. If the ground within 5 feet of the top of the excavation rises at an inclination of 1½H:1V or steeper, the shoring should be designed using an equivalent fluid density of 80 pcf. For adjacent slopes flatter than 1½H:1V, soil pressures can be interpolated between this range of values.

If portions of the shoring use passive elements such as anchor or reaction blocks, available soil resistance can be estimated using a passive soil pressure assuming an equivalent fluid density of 300 pcf above the water table. The upper foot of soil should be neglected when estimating passive earth pressures. This passive soil pressure includes a factor of safety of 1.5.

5.1.3.2.4. SHORING BELOW GROUNDWATER

The lateral pressure for design of 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 as described in the "Shoring Above Groundwater" section. Sloping backfill pressures described above should also be considered, where appropriate. If the ground around the top of 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 for conditions with horizontal backfill adjacent to the excavation. If the ground within 5 feet of the top of the excavation rises at an inclination of 1½H:1V or steeper, portions of the shoring below groundwater should be designed using an equivalent fluid density of 145 pcf. For adjacent slopes flatter than 1½H:1V, soil pressures can be interpolated between this range of values.

The passive soil resistance acting on the embedded portion of the shoring and passive elements such as anchor blocks can be evaluated using a lateral pressure equal to an equivalent fluid density of 150 pcf. The upper foot of soil should be neglected when estimating passive earth pressures. The passive soil pressures presented above include a factor of safety of 1.5.

If dewatering is performed from inside the excavation and the toe of the shoring is not embedded deep enough, seepage of water below the shoring could produce piping or heave in the sand that could potentially destabilize the base of the excavation. We recommend that the shoring extend below the bottom of the excavation a sufficient distance to prevent piping or heave. Assuming that groundwater within the excavation is lowered to at least 2 feet below the excavation bottom depth, the shoring should be extended at least 10 feet below the excavation bottom depth to prevent piping or heave. This consideration must be evaluated during design of the shoring and dewatering system.

5.1.3.2.5. SURCHARGE LOADS

The design of temporary shoring should include a surcharge load to account for traffic, construction equipment, and temporary stockpiles adjacent to the excavation. 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. For traffic loading, we recommend that temporary shoring walls be designed for a uniform surcharge pressure of 250 pcf. Higher surcharge pressures should be used if the designer deems them appropriate. Other surcharge loads, including equipment, stockpiles, and existing nearby structures should be included as appropriate.

5.1.4. Permanent Cut and Fill Slopes

In general, we recommend that permanent cut and fill slopes be constructed at a maximum inclination of 2H:1V. Where 2H:1V permanent slopes are not feasible, retaining structures should be considered. Slopes should be vegetated as soon as practical to reduce the surface erosion and sloughing. Temporary protection should be used until permanent protection is established. In order to achieve uniform compaction, we recommend that fill slopes be overbuilt and subsequently cut back to expose well-compacted fill. Fill placement on slopes steeper than 5H:1V should be benched into the slope face. The configuration of benches will depend on the equipment being used and the slope geometry.

5.1.5. Bearing Surface and Subgrade Preparation

5.1.5.1. AT-GRADE STRUCTURES AND ROADWAYS

To provide subgrade protection and a uniform bearing surface, we recommend that at-grade structures be supported on 1 foot of crushed rock or quarry spalls. The bearing surface soils for at-grade structures should be overexcavated 1 foot deeper than the design foundation elevation. The overexcavation should also extend 1 foot beyond the perimeter of the foundation. A geotextile fabric for separation should be placed on the overexcavated bearing surface soils prior to placing structural fill. The excavation should be backfilled to foundation bearing surface elevation with washed free-draining crushed rock or 2- to 4-inch minus quarry spalls as defined below, in the "Fill Materials" section.

Foundation bearing surfaces for at-grade structures and roadway subgrades should be thoroughly compacted to a uniformly firm and unyielding condition on completion of stripping and before placing structural fill or foundations. We recommend that foundation bearing surfaces and roadway subgrades be proof-rolled or probed, as appropriate, to identify areas of yielding or soft soil. Proof-rolling should be accomplished with a heavy piece of wheeled construction equipment such as a loaded dump truck or loader. If soft or otherwise unsuitable areas are revealed during proof-rolling or probing 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 compacted; or 2) the unsuitable soils be removed and replaced with compacted structural fill, as needed.

We recommend that prepared bearing surfaces be observed by a member of our firm, prior to placement of fill, reinforcing steel or pavement base, to verify that the procedures comply with the intent of our recommendations and the project plans and specifications. Foundation bearing surfaces should not be exposed to standing water. If water pools on the bearing surfaces, it should be removed before placing structural fill or reinforcing steel.

We recommend that project plans include a contingency for partial overexcavation of soft or otherwise unsuitable site soils (beyond the recommended 1 foot) and replacement with compacted structural fill. Overexcavation at the location of structures should extend to the depth necessary for the proposed use of the structure as determined by our firm representative. For estimating purposes, typical overexcavation depths could be on the order of 2 to 3 feet total. The overexcavation should extend beyond the perimeter of the structure or foundation an equal distance to the depth of overexcavation. Other options for remediation of soft bearing surfaces instead of or in conjunction with overexcavation may include stabilization methods such as the use of a geotextile fabric or other geotextile products and/or placement of quarry spalls.

5.1.5.2. BELOW-GRADE STRUCTURES

We recommend that excavations for below-grade structures be completed with as little bearing surface disturbance as possible. Because the bearing surface is likely to be wet and soft/loose, even with proper dewatering it could be difficult if not impossible to compact the bearing surface to a firm and unyielding condition.

We recommend that the bearing surface soils for below-grade structures be overexcavated 2 feet deeper than the design foundation elevation. The overexcavation should also extend 2 feet beyond the perimeter of the foundation. A geotextile fabric for separation should be placed on the overexcavated bearing surface soils prior to placing structural fill. The excavation should be backfilled to foundation bearing surface elevation with washed free-draining crushed rock or 2- to 4-inch minus quarry spalls as defined below, in the "Fill Materials" section. If, during construction, it becomes difficult to place the geotextile fabric directly on the overexcavated soil surface, it may be necessary to stabilize the soils with quarry spalls prior to placing fabric.

5.1.5.3. UTILITY TRENCH SUBGRADE

We recommend that excavations for utilities be completed with as little subgrade disturbance as possible. Because the subgrade is likely to be wet and soft/loose for deeper utilities, even with proper dewatering it could be difficult if not impossible to maintain grade and provide a firm base upon which to place the utility and backfill. If loose or otherwise unsuitable areas are revealed, we recommend that: 1) the loose soil be compacted, if practical; 2) the unsuitable soils be removed and replaced with 1 to 2 feet of quarry spalls "seated" into the underlying subgrade using equipment such as a vibratory plate mounted on an excavator. We are available to provide assistance evaluating utility trench subgrades, if requested.

5.1.6. Wet Weather Construction and Bearing Surface Protection

Portions of the on-site soil contain a high percentage of fines (material passing the U.S. Standard No. 200 sieve) and are moisture sensitive. When the moisture content of the soil is more than a few percent above the optimum moisture content, this soil may become muddy and unstable and it will be difficult or impossible to meet the recommended compaction criteria. Disturbance of near-surface soil should be expected if earthwork is completed during periods of wet weather.

The wet weather season generally begins in October and continues through May in this area; however, periods of wet weather may occur during any month of the year. The optimum earthwork period for this type of soil is typically June through September. If wet weather earthwork is unavoidable, we recommend that:

- Structural fill placed during the wet season or during periods of wet weather consist of select granular fill as defined in this report.
- The ground surface should be sloped to direct surface water away from the work area. The ground surface should be graded such that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should also be implemented to remove surface water from the work area.
- Earthwork activities should not take place during periods of heavy precipitation.
- Slopes with exposed soil should be covered with plastic sheeting or otherwise protected from erosion.
- Measures should be taken to prevent on-site soil and soil stockpiles from becoming wet or unstable. The site soil should not be left uncompacted and exposed to moisture. Sealing the surficial soil by rolling with a smooth-drum roller prior to periods of precipitation should reduce the extent that the soil becomes wet or unstable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with materials not susceptible to wet weather disturbance.
- Construction activities should be scheduled so that the length of time that soil is left exposed to moisture is minimized.
- A minimum 1-foot-thick layer of quarry spalls should be used in high traffic areas of the site to protect the bearing surface soil from disturbance. Additional thickness may be required, depending on subgrade conditions.
- Contingencies should be included in the project schedule and budget.

5.2. Fill Materials

5.2.1. Structural Fill

5.2.1.1. GENERAL

For the purposes of this report, materials placed to support structures and pavements and placed as backfill around below-grade structures are classified as structural fill. We provide recommendations for structural fill with varying characteristics depending upon its intended use. Import structural fill must be free of debris, organic material and rock fragments larger than 6 inches. The workability of material used as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines increases, soil becomes increasingly sensitive to small changes in moisture content and adequate compaction may become difficult or impossible to achieve. Weather and the time of year should be considered when evaluating material for use as structural fill.

5.2.1.2. FREE-DRAINING CRUSHED ROCK

Structural fill placed for the working pad beneath below-grade structures should consist of free-draining crushed rock. Washed, crushed rock passing the ½ inch sieve and retained on the U.S. No. 4 sieve (1-1/2 inch by #4 crushed rock) and permeable ballast conforming to Washington State Department of Transportation (WSDOT) Standard Specification 9-03.9(2) are appropriate. Alternative crushed rock gradations may be considered provided the material is free-draining.

5.2.1.3. SELECT BORROW

Structural fill placed as backfill around below-grade structures and below at-grade structures should meet the criteria for select borrow as described in Section 9-03.14(2) of the WSDOT Standard Specifications. Select borrow will be suitable for use as structural fill or as structural backfill during dry weather conditions only. If structural fill is placed during wet weather, the structural fill should consist of free-draining crushed rock described above or select granular fill as described below. It is our opinion that the on-site fill and upper alluvium may be considered as an alternative to imported select borrow, provided the soil can is properly moisture conditioned as described below in the "Use of On-site Material" section of this report.

5.2.1.4. SELECT GRANULAR FILL

If construction is performed during wet weather, we recommend imported structural fill consist of 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 3/4-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), 9-03.10 (Aggregate for Gravel Base), 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 3/4-inch fraction) and the maximum particle size is 6 inches.

5.2.2. Re-Use of On-site Material

Based on our explorations, it is our opinion that the on-site fill and upper alluvium may be considered for use as structural fill during dry weather construction. Most of the near-surface soils at the site have a relatively high fines content and are moisture sensitive, which could make them difficult or impossible to compact when wet. In our opinion, on-site soils with more than about 5 to 10 percent fines are not suitable for re-use as fill during periods of wet weather. Even during periods of prolonged dry weather, on-site soil may require drying to reduce the moisture content to near optimum. Soil excavated from below groundwater should not be considered for re-use unless a soil drying plan is implemented.

5.2.3. Trench Backfill

In general, we recommend that imported trench backfill have gradation characteristics similar to "Select Borrow" as described above and in Section 9-03.14(2) of the WSDOT Standard Specifications. If construction is performed during wet weather, we recommend using imported trench backfill consisting of select granular fill as described above. On-site fill and upper alluvium may be considered for use as trench backfill provided the soil can be moisture conditioned for proper compaction. If water is present in trenches during backfilling, free-draining crushed rock, as described above, should be used for trench backfill.

5.2.4. Pipe Bedding

Trench backfill for the bedding and pipe zone should consist of well-graded granular material with a maximum particle size of 3/4 inch and less than 5 percent passing the U.S. Standard No. 200 sieve. The material should be free of roots, debris, organic matter and other deleterious material. Alternative bedding materials may be required by local jurisdictions and/or by the pipe manufactures.

5.2.5. Quarry Spalls

We recommend that quarry spalls consist of 2- to 4-inch washed crushed stone that meets the quality characteristics indicated in Section 9-13 of the WSDOT Standard Specifications. Alternative stone size ranges may be appropriate, depending on the application.

5.3. Fill Placement and Compaction

5.3.1. General

Structural fill should be compacted at a moisture content near optimum. The optimum moisture content varies with the soil gradation and should be evaluated during construction. Material containing more than 5 percent fines can be difficult or impossible to compact when wet.

Fill and backfill material should be placed in uniform, horizontal lifts and uniformly densified with vibratory compaction equipment. The maximum lift thickness will vary depending on the material and compaction equipment used, but should generally not exceed 12 inches in loose thickness.

5.3.2. Structural Fill Beneath Foundations and Slabs

Structural fill beneath foundations and slabs should be placed on a bearing surface prepared in accordance with the “Bearing Surface and Subgrade Preparation” section. All structural fill beneath footings should be compacted to at least 95 percent of the maximum dry density (MDD) (ASTM International [ASTM] D 1557), and should extend beyond all edges of the footing a distance equal to the thickness of structural fill. For example, if footings are supported on 2 feet of structural fill, the structural fill should extend at least 2 feet beyond the edge of the footing on all sides.

5.3.3. Utility Trench Backfill

We recommend that the initial lift of fill over utility pipes be thick enough to reduce the potential for damage during compaction but generally should not be greater than about 18 inches. In addition, rock fragments greater than about 1 inch in maximum dimension should be excluded from this lift.

In building areas, trench backfill should be uniformly compacted in horizontal lifts to at least 95 percent of the MDD based on ASTM D 1557. Backfill placed more than 2 feet below subgrade in pavement areas should be compacted to at least 90 percent of the MDD (ASTM D 1557).

Backfill placed within 2 feet of subgrade in pavement areas should be compacted to at least 95 percent of the MDD (ASTM D 1557). In nonstructural areas, trench backfill should be compacted to a firm and unyielding condition.

5.3.4. Backfill Around Below-Grade Structures

We recommend that all backfill around below-grade structures be placed as structural fill. Backfill material should be placed in uniform, horizontal lifts and uniformly densified with vibratory compaction equipment. The maximum lift thickness will vary depending on the material and compaction equipment used, but generally should not exceed 12 inches in loose thickness. All structural fill placed around below-grade structures should be compacted to at least 90 percent of the MDD determined by ASTM Test Method D 1557 (modified Proctor). We recommend structural fill placed within 2 feet of pavement subgrade be compacted to at least 95 percent of the MDD (ASTM D 1557). If backfill around below-grade structures is intended to support foundations or slabs, we recommend it be compacted to at least 95 percent of the MDD (ASTM D 1557).

5.4. Foundation Design for At-Grade Structures

5.4.1. General

Foundation bearing surfaces should be prepared as described above in the “Bearing Surface Preparation” section of this report. The following paragraphs describe specific design considerations for spread footings as well as mats and slabs-on-grade. We also include recommendations for Primary Clarifier No. 3.

5.4.2. Primary Clarifier No. 3

We understand that upgrades to Primary Clarifier No. 3 will include extension of the clarifier sidewalls to an elevation about 5 feet higher than the current walls. We assume the existing mat foundation for the primary clarifier was constructed in general accordance with the recommendations provided in the Soils Report conducted by Earth Consultants, Inc. dated April 15, 1981. Based on this assumption, it is our opinion that the following recommendations for new foundations, including bearing pressures, lateral load resistance, settlement, and modulus of subgrade reaction are also appropriate for re-evaluation of the existing mat foundation at Primary Clarifier No. 3.

5.4.3. Spread Footings

5.4.3.1. FOUNDATION SUPPORT AND MINIMUM SIZE

Proposed at-grade structures can be satisfactorily founded on continuous wall or isolated column footings supported on bearing surfaces prepared as recommended. The exterior footings should be established at least 18 inches below the lowest adjacent grade for frost protection. Interior footings can be founded a minimum of 12 inches below the lowest adjacent grade. Isolated column and continuous wall footings should have minimum widths of 24 and 18 inches, respectively.

5.4.3.2. SOIL BEARING PRESSURE

The footings for planned at-grade structures are anticipated to bear on at least 1 foot of compacted free-draining crushed rock or quarry spalls. It is our opinion that an allowable soil bearing pressure of 2,500 pounds per square foot (psf) is appropriate for design of isolated footings with a minimum width of 2 feet and continuous footings with a minimum width of 18 inches. The allowable soil bearing values apply to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

5.4.3.3. LATERAL LOAD RESISTANCE

Lateral loads on foundation elements may be resisted by passive pressure on the sides of footings and other below-grade structural elements and by friction on the base of footings. Passive resistance may be estimated using an equivalent fluid density of 300 pcf, assuming that the footings and below-grade elements are backfilled with structural fill and there is no groundwater present. The upper 1 foot of should be neglected when estimating lateral load resistance. Frictional resistance may be estimated using 0.4 for footings bearing on select granular fill and free-draining crushed rock. The above values include a factor of safety of about 1.5.

5.4.3.4. SETTLEMENT

For the proposed at-grade structures, we estimate that settlement of footings designed and constructed as recommended should be less than 1 inch. We estimate differential settlements of ½ inch or less between comparably loaded isolated footings or along 25 feet of continuous footings. Most of the settlement should occur as loads are being applied. Loose or soft soil below footings or disturbance of foundation bearing surfaces during construction could result in more settlement than predicted.

5.4.4. Mats and Slabs-on-Grade

Slabs and mat foundations should be founded on prepared bearing surfaces and subgrades as recommended in this report. An allowable bearing capacity of 2,500 psf is appropriate for design. A modulus of subgrade reaction of 200 pounds per cubic inch (pci) may be used for designing mats and slabs-on-grade. This value is for a 1-foot by 1-foot square plate. The coefficient of subgrade reaction for a foundation varies based on its minimum width according to the following equation:

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

where k_s is the coefficient of subgrade reaction, k_{s1} is the coefficient of subgrade reaction for a 1-foot by 1-foot plate, and B is the minimum width or lateral dimension of the mat. For mats/slabs-on-grade designed and constructed as recommended, we estimate settlements of less than 1 inch. We estimate that differential settlement of the floor slabs will be 1/2 inch or less over a span of 25 feet.

We recommend that slabs-on-grade be underlain by a minimum 6-inch-thick capillary break layer to reduce the potential for moisture migration into the slab. The capillary break material should consist of well-graded sand and gravel or crushed rock with a maximum particle size of 3/4 inch and less than 3 percent fines. Alternatively, a crushed rock base course material conforming to 9-03.9(3) of the WSDOT Standard Specifications is suitable. The capillary break material should be placed in one lift. Where dry slabs are required (for example, where adhesives are used to anchor carpet or tile to the slab) a waterproof liner should be placed as a vapor retarder below the slab.

5.5. Conventional Retaining and Foundation Walls

5.5.1. General

The recommendations provided in this section are for walls constructed for minor grade separation and are not intended for retaining walls more than about 6 feet high or below-grade walls. Walls for below-grade structures should be designed in accordance with the "Below-Grade Structures" section of this report.

5.5.2. Lateral Earth Pressures

Lateral earth pressures acting on retaining walls will depend on the type and density of the wall backfill, whether or not the wall backfill is drained, the amount of lateral wall movement which occurs as backfill is placed, and the inclination of the ground surface. For walls free to yield at the top at least one thousandth of the wall height (i.e., wall height times 0.001), soil pressures will be less than if movement is restrained. We recommend that walls free to yield at the top be designed using an equivalent fluid density of 35 pcf for the drained condition. Restrained walls (walls not allowed to rotate at least 0.001 times wall height) should be designed using an equivalent fluid density of 55 pcf for the drained condition. Lateral resistance values for retaining wall footings should be designed in accordance with the "Lateral Load Resistance" section of this report.

For seismic loading conditions, a rectangular earth pressure equal to 7*H psf, where H is the height of the wall (in feet), should be added to the active pressures provided above. If the wall is designed for an at-rest condition, but is assumed to move during seismic conditions, then it is appropriate to combine the seismic surcharge with the active pressure.

The above-recommended lateral soil pressures do not include traffic surcharges, the effects of sloping backfill surfaces or hydrostatic forces. The recommended equivalent fluid densities presented assume that material behind the wall consists of structural fill for a horizontal distance behind the wall equal to the wall height. Over-compaction of fill placed directly behind retaining walls should be avoided. We recommend use of hand-operated compaction equipment and maximum 6-inch loose lift thickness when compacting fill within about 3 feet of retaining walls.

5.5.3. Drainage

Drainage systems should be constructed to collect water and prevent the buildup of hydrostatic pressure against retaining walls. 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. Free-draining backfill should conform to the WSDOT Standard Specification 9-03.12(2) "Gravel Backfill for Walls." The free-draining backfill zone should extend to within about a foot of the full height of the wall. A perforated rigid, smooth-walled drain pipe with a minimum diameter of 4 inches should be placed along the base of the wall 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 to reduce erosion potential.

Cleanouts should be provided to allow routine maintenance. In general, roof downspouts, perimeter drains or other types of drainage systems should not be connected to retaining wall drain systems. If external drainage systems are connected to retaining wall drains, the retaining wall drains must be designed to accommodate the additional volume of water, and features such as cleanouts and settling basins should be provided to allow maintenance of the drains and prevent fines from entering into the retaining wall drain system.

5.6. Below-Grade Structures

5.6.1. Soil Bearing Pressure

An allowable soil bearing pressure of 2,500 pounds per square foot (psf) may be used for design of mat and slab foundations for below-grade structures, provided the bearing surface is prepared as recommended in the “Bearing Surface and Subgrade Preparation” section of this report. The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

5.6.2. Lateral Earth Pressures

We recommend that permanent below-grade walls be designed for at-rest soil pressures and for full hydrostatic pressures below the design groundwater level. For this condition, we recommend using an equivalent fluid density of 90 pcf. If portions of the below-grade walls will be above the groundwater, we recommend an equivalent fluid density of 55 pcf. These lateral earth pressures are appropriate for level backfill conditions only. If the walls of below-grade structures are to be designed for flood conditions, we recommend assuming a groundwater level at the ground surface.

If the walls of below-grade structures are to be designed for seismic forces, we recommend that the seismic loading be approximated using a uniform lateral pressure equal to 7*H psf, where H is the depth (in feet) below grade of the structure. This seismic lateral pressure is in addition to the static soil and hydrostatic pressures. In our opinion, it is conservative to design structures for simultaneous flood and seismic conditions.

These lateral soil pressures do not include traffic or other surcharges. Typically, below-grade walls are designed for a surcharge pressure for traffic loading. For traffic loading, we recommend that below-grade walls be designed for a uniform surcharge pressure of 250 pcf. Higher surcharge pressures and other surcharge loads should be used where appropriate.

5.6.3. Buoyancy and Uplift

We anticipate that below-grade structures will extend below the design groundwater level; therefore, buoyancy and uplift forces must be considered. Uplift forces can be resisted by the dead weight of the structure and friction along the sides of the structure. In addition to dead weight and frictional resistance, the structure may be constructed with footings that extend beyond the structure walls so that the weight of overlying soil resists a portion of the uplift. Figure 3 shows our recommended procedure for evaluating uplift resistance.

If it is not feasible to design below-grade structures to resist buoyant forces when groundwater level is high, pressure relief valves (PRVs) may be incorporated into the base or walls of the structure to allow groundwater to enter the structure when empty. Perimeter or base drainage zones should be incorporated at the locations of the PRVs to allow for rapid dissipation of hydrostatic pressure. Drainage zones at PRV locations should consist of at least 4-inch-diameter perforated pipe surrounded on all sides by 6 inches of drain material. The drain material and pipe should be enclosed in a non-woven geotextile fabric for underground drainage to prevent fine soil from migrating into the drain material. We recommend that the drainpipe consist of either heavy-wall solid pipe (SDR-35 PVC, or equal) or rigid corrugated smooth interior polyethylene pipe (ADS N-12, or equal). We do not recommend using flexible tubing for drainpipes. The drain material should consist of pea gravel or "Gravel Backfill for Drains" per WSDOT Standard Specifications Section 9-03.12(4).

5.7. Seismic Design Considerations

5.7.1. International Building Code (IBC) Parameters

We understand seismic design will be performed using the procedure outlined in the 2009 IBC. The design parameters provided below may be used for design.

TABLE 1. SEISMIC DESIGN CRITERIA

2009 IBC Seismic Design Parameters	
Spectral Response Acceleration at Short Periods (S_S)	1.18g
Spectral Response Acceleration at 1-Second Periods (S_1)	0.40g
Site Class	D
Design Peak Ground Acceleration (PGA)	0.32g
Design Spectral Response Acceleration at Short Periods (S_{DS})	0.81g
Design Spectral Response Acceleration at 1-Second Periods (S_{D1})	0.43g

5.7.2. Liquefaction

Soil liquefaction refers to the condition by which vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent reduction in soil shear strength. The evaluation of liquefaction potential is complex and dependent on numerous parameters, including soil type, grain-size distribution, soil density, depth to groundwater, in-situ static ground stresses, earthquake magnitude, peak ground acceleration (PGA), earthquake-induced ground stresses and excess pore water pressure generated during seismic shaking. In general, soils that are susceptible to liquefaction include very loose to medium dense, clean to silty sands and very soft to stiff non-plastic silts that are below the water table.

We evaluated the liquefaction potential of the site soil using simplified methods (Seed, et al., 2003 and Idriss and Boulanger, 2008), which are based on comparing the cyclic resistance ratio (CRR) of a soil layer (the cyclic shear stress required to cause liquefaction) to the cyclic stress ratio (CSR) induced by an earthquake. The factor of safety (FS) against liquefaction is determined by dividing the CRR by the CSR. For this project we evaluated liquefaction hazards, including settlement and related effects, when the FS against liquefaction was calculated as less than 1.2.

Based on our liquefaction analysis, it is our opinion that there is potential for liquefaction of the site soils during the design earthquake (M = 7.0, PGA = 0.32g). We estimate that liquefaction-induced settlement could range from about 10 to 20 inches at the ground surface and 5 to 15 inches at the bottom of the proposed secondary clarifier and proposed aeration basin as a result of the design level earthquake. Areas of liquefaction can be relatively discontinuous, and separated by layers of non-liquefied soil; therefore, we estimate that liquefaction-induced differential settlement could be on the order of 5 to 10 inches per 100 feet.

5.7.3. Lateral Spreading

Lateral spreading involves lateral displacements of large volumes of liquefied soil. Lateral spreading can occur on near-level ground as blocks of surface soils displace relative to adjacent blocks. Lateral spreading also occurs as blocks of surface soils are displaced toward a nearby slope (free face) by movement of the underlying liquefied soil. The banks of the Puyallup and White Rivers represent free face conditions for this site. Therefore, the topography of the site and underlying soil conditions indicate that lateral spreading is a strong possibility at the wastewater treatment plant site.

Lateral spreading analysis using an empirical model to predict free-field ground displacements that might be associated with lateral spreading at the site was conducted for the geotechnical report dated September 11, 2002. In our opinion, the previous analysis is still valid for the proposed facility upgrades.

The results of the previous analysis indicate that the soils at this site have the potential to generate lateral displacements during the design earthquake with a magnitude of 7.0. We estimate that free-field lateral displacements at the site may be on the order of 2 to 3 feet around the perimeter site where the ground surface slopes toward the rivers, and 1 to 2 feet in the center site for the design earthquake event.

5.7.4. Ground Rupture

Because of the anticipated site location with respect to the nearest known active crustal faults and the presence of thick glacial and alluvial deposits overlying bedrock, it is our opinion that the risk of ground rupture at the site due to crustal faulting is low.

5.7.5. Liquefaction and Lateral Spreading Mitigation

The potential effects of liquefaction and lateral spreading have been discussed with G&O. Based on magnitude of liquefaction settlement and lateral spreading estimated for this site, severe foundation settlement or lateral movement of the existing and proposed structures and piping could occur during the design earthquake.

Because the majority of structures at the wastewater treatment plant are not intended for human occupancy, we understand that current code standards do not require that these structures be designed to resist the effects of liquefaction and lateral spreading. We understand that the City of Sumner has elected to accept the risk of seismic damage to the facility; therefore, the new structures can be supported on shallow foundations or mat foundations. We recommend that any structure intended for human occupancy be designed to resist the estimated settlement and lateral displacement discussed above.

6.0 LIMITATIONS

We have prepared this report for the exclusive use of the City of Sumner, Gray & Osborne, Inc. and their authorized agents for the Sumner Wastewater Treatment Plant facility upgrades project located in Sumner, Washington.

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

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

7.0 REFERENCES

Dames & Moore. 1971. "Report of Site Investigation, Proposed Sewage Treatment Plant, Sumner, Washington," May 19, 1971.

Earth Consultants, Inc. 1981. "Geotechnical Engineering Study, Sumner Waste Water Treatment Plant Addition, Sumner, Washington," April 15, 1981.

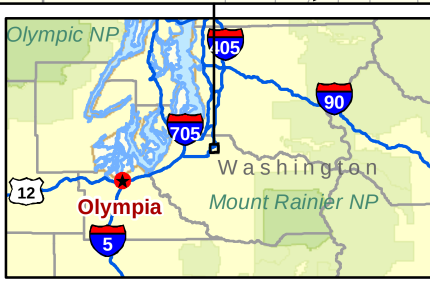
GeoEngineers, Inc. 2002. "Geotechnical Engineering Services, Sumner Wastewater Treatment Plant, Sumner, Washington," September 11, 2002.

Idriss, I. M., and Boulanger, R. W. 2008. Soil Liquefaction During Earthquake, Earthquake Engineering Research Institute, EERI Publication MNO-12.

R. B. Seed, K. O. Cetin, R. E. S. Moss, A. Kammerer, J. Wu, J. Pestana, M. Riemer, R. B. Sancio, J. D. Bray, R. E. Kayen, and A. Faris. 2003. "Recent advances in soil liquefaction engineering: A unified and consistent framework." Keynote presentation, 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, Long Beach, CA.

Troost, K.G., in review, Geologic map of the Puyallup 7.5-minute quadrangle, Washington: U.S. Geological Survey Miscellaneous Field Investigation, scale 1:24,000.

Walsh. 1987. "Geologic Map of the South Half of the Tacoma Quadrangle, Washington."



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.
3. It is unlawful to copy or reproduce all or any part thereof, whether for personal use or resale, without permission.

Data Sources: ESRI Data & Maps, Street Maps 2005

Transverse Mercator, State Plane South, North American Datum 1983
North arrow oriented to grid north

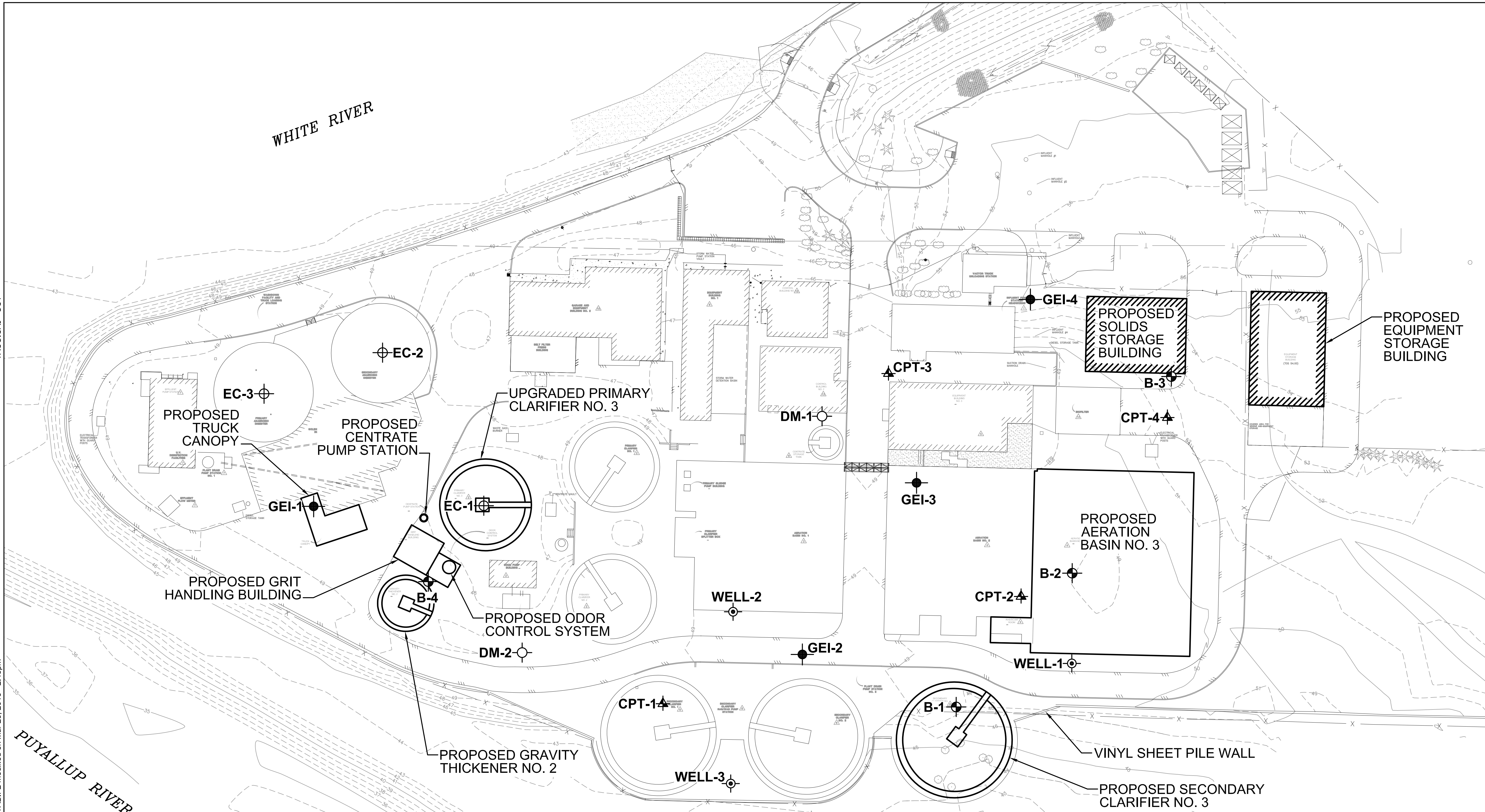
Vicinity Map

Sumner WWTP - Facility Upgrades
Sumner, Washington

GeoEngineers

Figure 1

Office: TACO Path: P:\0\0588004\GIS\058800403_F1.mxd Map Revised: 14 March 2013 syi



TACO:CAJ : SCY

P:\0\0588004\03\CAD\058800403_F2.dwg\TAB:F2 modified on Mar 25, 2013 - 2:48pm

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.

Reference: Drawing provided by Gray & Osborne, Inc.

Legend

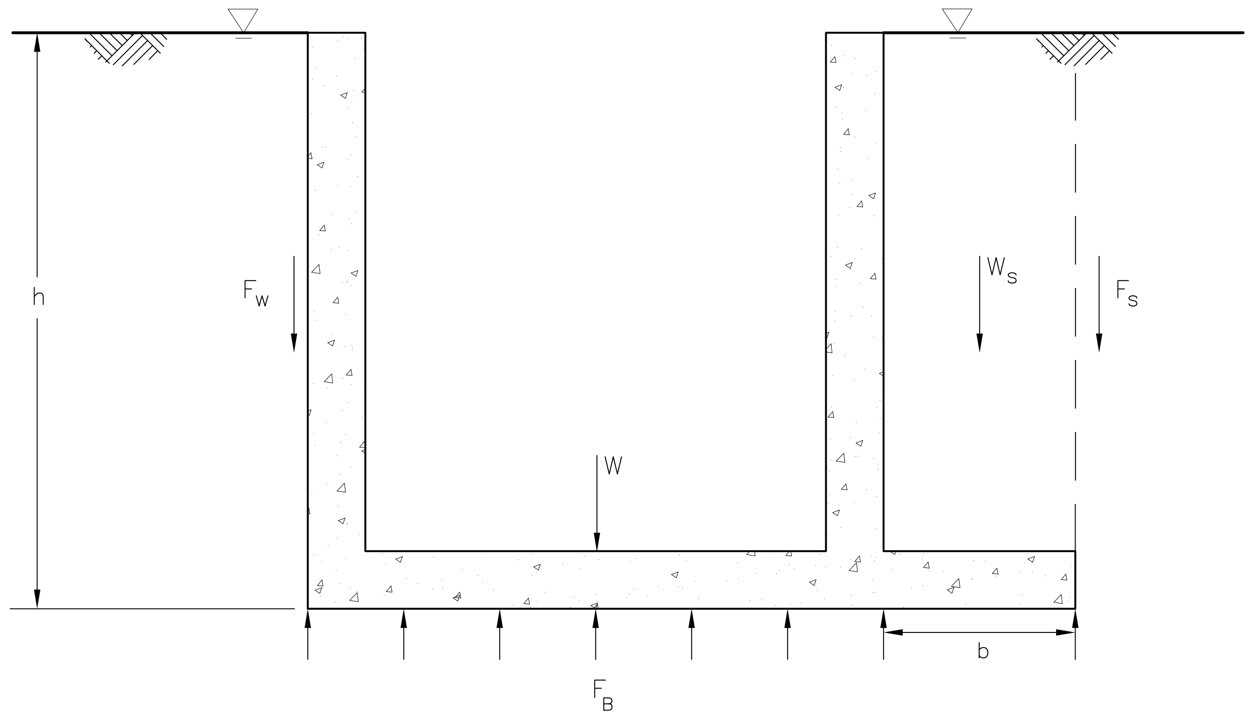
- B-1 Boring number and approximate location (this study)
- WELL-1 Existing well number and approximate location (installation date unknown)
- GEI-1 Boring number and approximate location (GeoEngineers, Inc., 2002)
- CPT-1 Cone penetration test sounding number and approximate location (GeoEngineers, Inc., 2002)
- EC-1 Boring number and approximate location (Earth Consultants, 1981)
- DM-1 Boring number and approximate location (Dames & Moore, 1971)

Site Plan

Sumner WWTP - Facility Upgrades
Sumner, Washington

GEOENGINEERS

Figure 2



NOT TO SCALE

ASSUMED SOIL PARAMETERS

Total Soil Unit Weight, γ_T = 125pcf
Bouyant Soil Unit Weight, γ_b = 63pcf
Soil Friction Angle, ϕ = 34°
Wall Friction Angle, δ = 28°
Earth Pressure Coefficient, K_o = 0.441

FACTORS OF SAFETY

$$FS = \frac{W + F_W * P}{F_B}$$

(for case where no protruding structure is present)

$$FS = \frac{W + (F_S + W_S) * P}{F_B}$$

(for case where protruding structure is present around perimeter of structure)

KEY TO SYMBOLS/FORCES

h = Depth to base of structure (feet)
b = Width of structure covered with structural fill (feet)
W = Weight of structure (pounds)
W_S = Effective weight of structural fill directly above protruding structure
= $b\gamma_b h$ (in pounds per foot linear of wall)
P = Perimeter around base of structure (feet)
F_b = Bouyant force acting on structure (pounds)
= 62.4pcf * (volume of structure below groundwater table in cubic feet)
F_W = Shearing resistance between wall and soil (DO NOT USE if Fs is used)
= $7h^2$ (in pounds per linear foot of wall)
F_S = Shearing resistance of soil
= $9h^2$ (in pounds per linear foot of wall)

Recommended Procedure for Evaluating Uplift Resistance

Sumner WWTP - Facility Upgrades
Sumner, Washington

GEOENGINEERS

Figure 3

APPENDIX A
Subsurface Explorations

APPENDIX A
SUBSURFACE EXPLORATIONS

Subsurface Explorations

We explored subsurface conditions at the site on February 25, 2013 by advancing four borings to depths between about 21.5 and 51.5 feet below surrounding site grades. Exploration locations were selected during our site walk of the site with Gray & Osborne, Inc. (G&O) using GPS equipment. Locations of the explorations are provided in Figure 2. The elevations presented on the boring logs are based on the use of a hand level and topographical data provided by G&O. The locations and elevations of the explorations should be considered approximate.

The field explorations were completed under the direction of our personnel. We obtained disturbed soil samples from the borings using a 1.5-inch-inside-diameter split-spoon SPT sampler driven into the soil using a 140-pound hammer free-falling a distance of 30 inches. Soil samples were visually classified in general accordance with the system described in Figure A-1. The samples were placed in sealable plastic bags then returned to our laboratory for further analysis. Logs of the explorations are presented as Figures A-2 through A-5.

Existing Wells

The water levels inside the existing wells were measured within one hour of drilling and measuring the water level in boring B-1. Water levels were measured using an e-tape water measurement tool in the borings and a weighted tape in the existing wells.

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

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.

ADDITIONAL MATERIAL SYMBOLS

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

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 or geologic units

Approximate location of soil strata change within a geologic soil unit

Material Description Contact

Distinct contact between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

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
PI	Plasticity index
PP	Pocket penetrometer
PPM	Parts per million
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

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	2/25/2013	2/25/2013	51.5	CAJ	MM	Holocene Drilling	Hollow Stem Auger

Surface Elevation (ft) Vertical Datum	51	Hammer Data	140 lb/30 in. drop Autohammer
Drilling Equipment	B-59 Truck Mounted	System Datum	Gray and Osborne Topo
Easting (X) Northing (Y)		Groundwater: Date Measured / Depth to Water (ft) / Elevation (ft)	2/25/2013 / 14.0 / 37
Notes:			

Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
0				SOD	4 inches sod			
				TS	6 inches topsoil, dark brown silty sand with organics (very soft, moist) (fill)			
				SM	Dark brown silty fine sand with gravel (dense, moist) (fill)			
	17	39	1					
5	15	40	2 / MC			8		
	15	31	3A,B	SP-SM	Dark gray fine sand with silt and occasional gravel (dense, moist) (fill)			
				GM	Dark brown silty fine to coarse gravel with sand (very dense, moist) (fill)			
10	17	60	4					
				SM	Dark gray silty fine sand with occasional fine gravel (medium dense, moist) (upper alluvium)			
15	14	29	5 / %F		Grades to wet	20		%F=20
				SP	Dark gray fine to medium sand with trace silt (medium dense, wet) (upper alluvium)			
20	4	31	6					
25	12	6	7A,B / %F			24		%F=3
				SM	Gray silty fine to coarse sand with occasional gravel (loose, wet) (mudflow)			
30	9	3	8A,B / SA			21		%F=24
35								

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

Log of Boring B-1

GeoEngineers

Project: Facility Upgrades - Sumner Wastewater Treatment Plant
Project Location: Sumner, Washington
Project Number: 0588-004-03

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
15	35		10	5		9A,B							
10	40		0	4		10							
5	45		16	7		11A,B / %F					18		%F=21
0	50		14	12		12				Grades to medium dense			

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

Log of Boring B-1 (continued)

GeoEngineers

Project: Facility Upgrades - Sumner Wastewater Treatment Plant
Project Location: Sumner, Washington
Project Number: 0588-004-03

Figure A-2
Sheet 2 of 2

Drilled	Start	End	Total Depth (ft)	31.5	Logged By	CAJ	Driller	Holocene Drilling	Drilling Method	Hollow Stem Auger
	2/25/2013	2/25/2013			Checked By	MM				

Surface Elevation (ft) Vertical Datum	49	Hammer Data	140 lb/30 in. drop Autohammer	Drilling Equipment	B-59 Truck Mounted
Easting (X) Northing (Y)		System Datum	Gray and Osborne Topo	Groundwater	

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
2/25/2013	12.7	36.3

Notes:

Elevation (feet)	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample Sample Name Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0					GP	Gray fine to coarse gravel with silt and sand (crushed rock) (fill)			
						SM	Dark gray silty fine sand (loose, moist) (fill)			
		3	9	1						
45										
	5	4	6	2A,B MC				14		
40						SP-SM	Dark gray fine to coarse sand with silt and gravel (medium dense, moist) (fill)			
	10	6	28	3						
		12	21	4	▼		Grades to wet			
35						SP	Dark gray fine to coarse sand with gravel and trace silt (dense, wet) (upper alluvium)			
	15	12	43	5A,B						
30										
	20	10	22	6 SA			Grades to medium dense	17		%F=4
25										
	25	18	2	7A,B %F		SM	Gray silty fine to coarse sand with occasional gravel (very loose, wet) (mudflow)	20		%F=25
20										
	30	18	2	8						

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

Log of Boring B-2

GEOENGINEERS

Project: Facility Upgrades - Sumner Wastewater Treatment Plant
Project Location: Sumner, Washington
Project Number: 0588-004-03

Figure A-3
Sheet 1 of 1

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	2/25/2013	2/25/2013	21.5	CAJ	MM	Holocene Drilling	Hollow Stem Auger

Surface Elevation (ft) Vertical Datum	53	Hammer Data	140 lb/30 in. drop Autohammer	Drilling Equipment	B-59 Truck Mounted
Easting (X) Northing (Y)		System Datum	Gray and Osborne Topo	Groundwater	

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
2/25/2013	17.3	35.7

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0								SOD	4 inches sod			
									SM	Dark brown silty fine sand (loose, moist) (fill)			
50			16	5		1 / MC					12		
	5		14	18		2A,B / MC					7		
									SP-SM	Dark brown fine to coarse sand with silt and gravel (medium dense, moist) (fill)			
45									GP-GM	Dark brown fine to coarse gravel with silt and sand (dense, moist) (fill)			
	10		13	45		3							
40													
	15		14	50		4				Grades to very dense			
							▼						
35									SP-SM	Gray fine to coarse sand with silt and gravel (medium dense, wet) (upper alluvium)			
	20		14	20		5							

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

Log of Boring B-3

GeoEngineers

Project: Facility Upgrades - Sumner Wastewater Treatment Plant

Project Location: Sumner, Washington

Project Number: 0588-004-03

Figure A-4

Sheet 1 of 1

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	2/25/2013	2/25/2013	21.5	CAJ	MM	Holocene Drilling	Hollow Stem Auger

Surface Elevation (ft) / Vertical Datum	48.5	Hammer Data	140 lb/30 in. drop Autohammer
Drilling Equipment	B-59 Truck Mounted		
Easting (X) / Northing (Y)		System Datum	Gray and Osborne Topo
Notes:			

Groundwater: Date Measured	Depth to Water (ft)	Elevation (ft)
2/25/2013	13.8	34.7

FIELD DATA

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0					SOD	Dark brown sandy silt with organics (roots) (very soft, moist) (fill)			
						GM	Dark brown silty fine to coarse gravel with sand (medium dense, moist) (fill)			
45		14	23	1						
	5	13	15	2						
						SM	Dark brown silty fine sand with occasional organics (fine roots) (medium dense, moist) (fill)			
40										
	10	11	3	3 / %F			Grades to very loose	19		%F=26
						ML	Dark brown sandy silt with occasional organics (fine roots) (soft, wet) (upper alluvium)			
35		16	2	4 / SA	▼			34		%F=61
	15	16	7	5A,B			Grades to medium stiff?			
						SP	Dark gray fine to coarse sand with gravel and trace silt (loose, wet) (upper alluvium)			
30										
	20	10	30	6			Grades to dense			

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

Log of Boring B-4

GEOENGINEERS

Project: Facility Upgrades - Sumner Wastewater Treatment Plant
Project Location: Sumner, Washington
Project Number: 0588-004-03

Figure A-5
Sheet 1 of 1

Appendix B
Laboratory Testing

APPENDIX B
LABORATORY TESTING

General

Soil samples obtained from the explorations 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 or modify our field classifications.

Our testing program consisted of the following:

- Three – Grain-size distribution analyses (SA)
- Five – Percent passing U.S. No. 200 sieve (%F)
- Four – Moisture tests (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.

Grain-Size

Grain-size analyses were performed on selected samples in general accordance with ASTM Test Method D 422. This test provides a quantitative determination of the distribution of particle sizes in soils. Grain-size analyses are used to classify soil and to aid in evaluating index properties. Figure B-1 presents the results of the grain-size analyses.

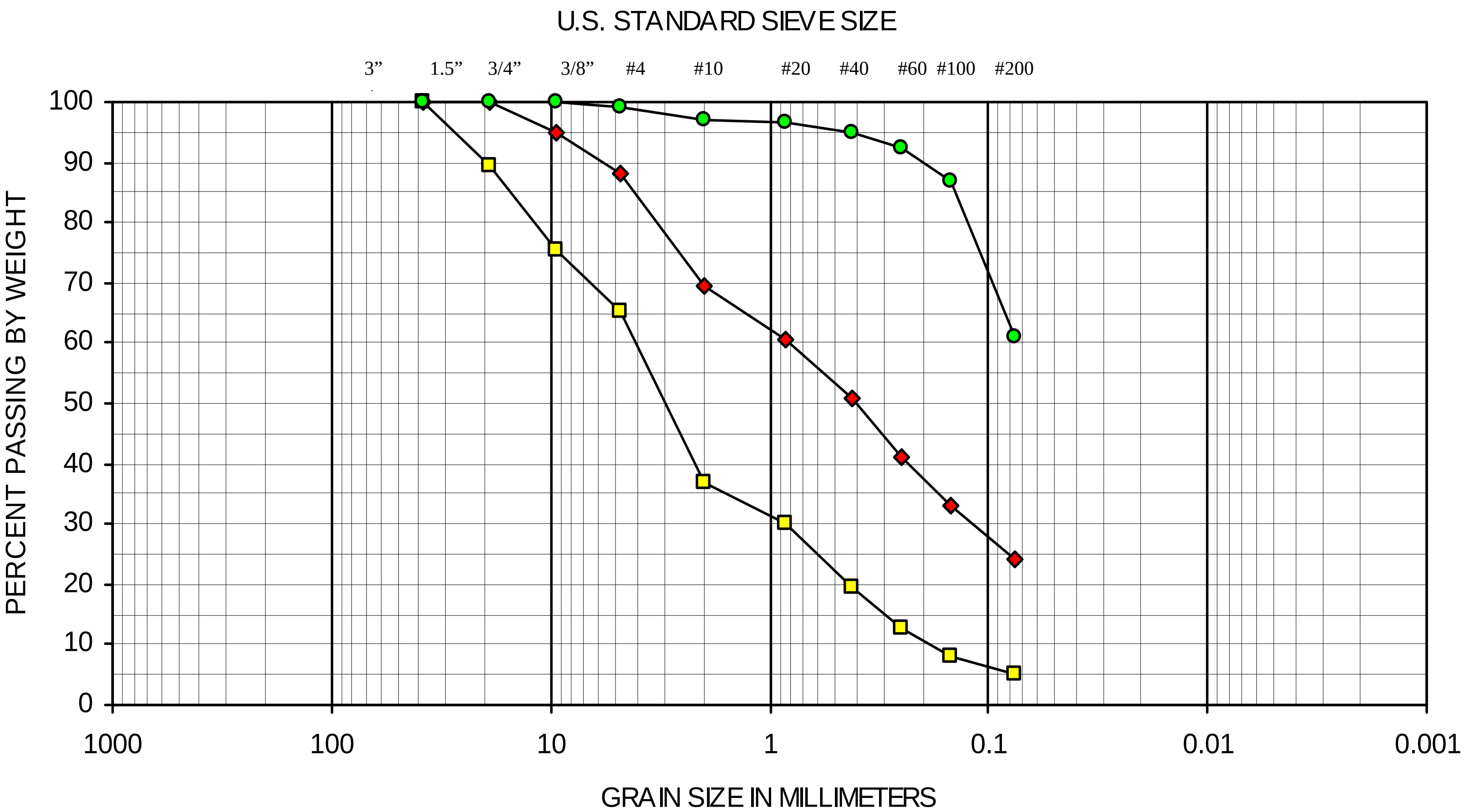
Percent Passing the U.S. No. 200 Sieve

Selected samples were "washed" through the U.S. No. 200 sieve to estimate the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve (fines). The tests were conducted in general accordance with ASTM D 1140. The test results are presented on the exploration logs in Appendix A at the respective sample depths.

Moisture Content

The moisture content of selected samples was determined in general accordance with ASTM D 2216. The test results are used to aid in determining the moisture content of the soil, soil classification and correlation with other pertinent engineering soil properties. The test results are presented on the exploration logs in Appendix A at the respective sample depths.

GEOENGINEERS



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	EXPLORATION NUMBER	DEPTH (ft)	MOISTURE (%)	SOIL CLASSIFICATION
◆	B-1	30	21	Brown silty sand (SM)
■	B-2	20	17	Brown sand with gravel (SP)
●	B-4	12.5	34	Brown silt with sand (ML)

SIEVE ANALYSIS RESULTS

FIGURE B-1

APPENDIX C
Report Limitations and Guidelines for Use

APPENDIX C
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

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

Geotechnical Services are Performed for Specific Purposes, Persons and Projects

This report has been prepared for the exclusive use of the City of Sumner, Gray & Osborne, Inc. (G&O) 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 Facility Upgrades at the Sumner Wastewater Treatment Plant located in Sumner, 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;

---

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

- 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.

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.

Topsoil

For the purposes of this report, we consider topsoil to consist of generally fine-grained soil with an appreciable amount of organic matter based on visual examination, and to be unsuitable for direct support of the proposed improvements. However, the organic content and other mineralogical and gradational characteristics used to evaluate the suitability of soil for use in landscaping and agricultural purposes was not determined, nor considered in our analyses. Therefore, the information and recommendations in this report, and our logs and descriptions should not be used as a basis for estimating the volume of topsoil available for such purposes.

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.