Phase 1 Geotechnical Engineering Services

Totem Lake Connector
NE 124th Street/124th Avenue NE
Kirkland, Washington

for
City of Kirkland
c/o COWI North America, Inc.

July 14, 2017

GeoEngineers

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INTRODUCTION AND PROJECT DESCRIPTION

This report presents the results of our preliminary geotechnical engineering services in support of Phase 1 design of the proposed Totem Lake Connector project located at NE 124th Street and 124th Avenue NE in Kirkland, Washington. The location of the project site is shown on the Vicinity Map, Figure 1.

The Totem Lake Connector will consist of a non-motorized bridge conforming to AASHTO Shared-Use Path Guidelines, spanning the intersection of NE 124th Street/124th Avenue NE and Totem Lake Boulevard. The preliminary bridge alignment and site features are shown in the Site Plan, Figure 2. The bridge will provide an elevated connection between segments of the existing Cross Kirkland Corridor (CKC) trail. The CKC is an approximately 5 ¾ mile long multipurpose trail on a former BNSF railroad grade that extends north from 108th Avenue NE near State Route 520 to Slater Avenue NE.

The initial concept for the bridge includes:

- an embankment for the south approach ramp flanked by retaining walls;
- the bridge spanning over NE 124th Street and Totem Lake Boulevard with a “touchdown” support in the triangular property bounded by these roadways; and
- a spiral ramp located just northeast of Totem Lake Boulevard extending over the park and wetland associated with Totem Lake, transitioning back to the trail alignment.

Our services were completed in general accordance with a Subconsultant Agreement between COWI North America, Inc. and GeoEngineers dated January 2017.

Our scope of services includes:

- reviewing existing geologic and geotechnical information available for the site and surrounding areas;
- completing explorations at the site to characterize the subsurface soil and groundwater conditions;
- completing geotechnical laboratory testing on selected soil samples obtained from the explorations;
- providing recommendations for seismic design in accordance with the 2014 AASHTO LRFD Bridge Design Specifications, 7th Edition;
- completing analysis to evaluate the axial and lateral capacity for deep foundations supporting the proposed bridge structure;
- evaluating settlement of ramp fills and bridge foundations;
- developing options for retaining walls for bridge abutment and ramp fills;
- providing recommendations for site preparation, earthwork, pavements, and underground utility construction; and
- preparing draft and final versions of this preliminary geotechnical design report.
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Preliminary subsurface conditions were evaluated along the project alignment by drilling seven borings (B-1 through B-7) to depths ranging from 21½ to 71½ feet below the existing ground surface. The locations of the subsurface explorations completed for this project are shown on Figure 2. Details of the field exploration program and logs of the borings are presented in Appendix A, Field Explorations.

Laboratory Testing

Soil samples were obtained during drilling and taken to GeoEngineers' laboratory for further evaluation. Selected samples were tested for the determination of moisture content, fines content (particles passing the U.S. No. 200 sieve), grain size distribution, and plasticity characteristics. A description of the laboratory testing and the test results are presented in Appendix B.

Previous Site Explorations

In addition to the explorations completed as part of this evaluation, we reviewed logs of available explorations from previous studies along and near the project alignment. The logs of explorations from previous projects referenced for this study are presented in Appendix C, Previous Explorations.

SITE CONDITIONS

Geology

Published geologic information for the project area includes a United States Geological Survey (USGS) map for the Kirkland, Washington quadrangle (Minard 1983). The mapped surface geologic units in the project area include recessional outwash (Qvr), glacial till (Qvt) and transitional beds (Qtb). The recessional outwash is mapped north of NE 124th Street and consists primarily of stratified sand and gravel with varying percentages of silt and clay. These deposits are related to a glacial meltwater channel that extends west to east in the Totem Lake area, and are generally in a loose to medium dense condition.

Glacial till and the transitional beds are mapped south of NE 124th Street. Glacial till generally consists of a non-sorted, non-stratified mixture of clay, silt, sand and gravel with larger constituents up to the size of boulders. The till is very dense and relatively impermeable, but can contain minor amounts of interbedded stratified sand and gravel. The transitional deposits consist of massive to bedded silt, clay and sand with minor amounts of peat and gravel. The transitional bed deposits are generally in a very stiff to hard condition due to being overridden by glaciers.

Surface Conditions

The project alignment extends in a southwest-to-northeast direction and is parallel to the Cross Kirkland Connector (CKC) multi-purpose trail along most of its length. The south approach will begin about 500 feet southwest of NE 124th Street on the existing trail grade, and extend up through the slope cut to the proposed bridge alignment. The proposed bridge alignment is located roughly 15 to 30 feet away and parallel to the existing trail alignment.
Numerous underground utilities exist along and across the alignment, the most notable of which are a fiber optic line on the east side of the CKC trail and a deep sanitary sewer line that crosses the western part of the spiral ramp area.

For the purposes of discussion, we have divided the site into three general areas below.

**South Alignment**

The southern part of the alignment and CKC trail is located within a through cut made for the former Eastside Rail Corridor (ERC) that was first developed in 1904 as part of the Lake Washington Belt Line. The line was initially used for hauling coal and lumber, and eventually for agricultural and industrial use. The Spirit of Washington Dinner Train also used the tracks from 1993 to 2007. In late 2009, BNSF sold the line to the Port of Seattle and the CKC came into public ownership.

Most of the trail is surfaced with fine gravel; however, a short asphalt paved section is present where the trail approaches the existing streets. Elevations along the south portion of the trail segment range from about Elevation 150 feet to about Elevation 146 feet. (Elevations in the report refer to NAVD 88 datum). Drainage ditches with depths up to about 3 feet are present on both sides of the existing CKC trail.

The existing through-cut within the south portion of the alignment contains cut slopes ranging from a few feet high to as high as 15 feet. The cut slope inclination is typically 1½H:1V (horizontal to vertical) and is vegetated with low brush.

Adjacent development includes a gas station and roadway to various commercial properties (Office Max, motel and fitness club) on the northwest, and a public storage facility on the southeast.

**Central Alignment**

NE 124th Street is a four-lane, high-volume arterial street that crosses the central alignment in a west to east direction. A triangular-shaped traffic island is located on the north side of NE 124th Street and south side of Totem Lake Boulevard NE, also a high-volume arterial. A retail store (Rite Aid) is located west of the island and turn lane from Totem Lake Boulevard NE to 124th Street.

The traffic island is nearly level with ground surfaces at approximately Elevation 145 feet. A small metal signal building remaining from the railroad era is located within the western part of the island and grass covers the remaining area.

**North Alignment**

North of Totem Lake Boulevard NE, the bridge will transition to a spiral ramp connecting to the existing CKC trail. The trail in this area is supported on a fill embankment placed for the former railroad. A short asphalt paved section of trail transitions to a gravel surfaced segment that continues northeast to 128th Place NE. A retail tire store (Discount Tire) is present on the southeast side of the existing trail and a wetland bordering Totem Lake is present to the northwest below the trail embankment. High voltage power lines cross over the trail in a south to north direction.

The existing ground surface ranges from about Elevation 141 feet on the trail surface to about Elevation 125 feet at the toe of the slope near the edge of a large wetlands. Slope inclinations range from about 2H:1V for the railroad embankment to about 4H:1V near the toe of the slope. Vegetation in the spiral ramp
area includes heavy underbrush and deciduous trees of varying diameters. Some of the brush and smaller trees were cleared to provide access for the drill rig used to complete the borings. Clearing and boring locations were limited to outside the wetlands for Phase I explorations.

**Subsurface Conditions**

We evaluated subsurface soil and groundwater conditions along the project alignment by drilling seven borings (B-1 through B-7) to depths ranging from 21⅓ to 71½ feet below the existing ground surface, and by reviewing the logs of selected previous explorations completed near the alignment.

The following sections describe subsurface soil conditions for: (1) the south approach ramp and bridge abutment near NE 124th Street, (2) the bridge touchdown location within the traffic island, and (3) the spiral ramp area and transition to the existing trail grade.

**South Bridge Abutment and South Approach Ramp**

Explorations located south of NE 124th Street include borings B-5 through B-7 drilled for the current study and boring B-93 drilled in 1987 by Converse Consultants.

Boring B-5 was located near the proposed south abutment and encountered 6 inches of gravel trail surfacing over medium dense silty sand to a depth of 7 feet. Stiff to hard silt and clay and dense silty and clayey sand with varying amounts of gravel was encountered below the surficial silty sand. Subsurface soils were similar in the 1987 boring, B-93.

Boring B-6 was drilled along the existing CKC trail near the end of the south approach ramp and boring B-7 was drilled near the future NE 120th Street crossing. Both borings encountered about 6 inches of gravel trail surfacing over either native silt soils (B-6) or silty sand fill (B-7) that could be related to utility installation. The fill is in a dense condition and extends to a depth of about 4½ feet in boring B-7. The fill is underlain by native stiff to hard silt containing thin lenses of peat. A layer of dense silty sand was encountered between the silt layers in boring B-6 at a depth of 13 to 18 feet.

**Bridge Touchdown in Traffic Island**

Boring B-4 was drilled within the traffic island and encountered 6 inches of topsoil over loose to medium dense silty sand with gravel fill. The fill extends to a depth of about 7 feet and is underlain by loose to medium dense silty sand. Below this depth, the boring encountered variable soil conditions consisting of alternating layers of medium dense to dense silty and clayey gravel with cobbles, and stiff to very stiff silt and clay.

At a depth of 53 feet, the boring encountered very dense clayey gravel with cobbles. This unit is underlain by very dense silty sand with gravel that extends to the bottom of the boring at 66½ feet.

**Spiral Ramp and Transition to Existing Trail**

Borings B-1 through B-3 were drilled in the proposed spiral ramp area outside the wetland area. Borings B-87 and B-89 drilled in 1987 by Converse Consultants were located east of the former railroad grade.

Boring B-1 encountered about 4½ feet of loose silty sand fill associated with the embankment that supports Totem Lake Boulevard to the west. Boring B-3 encountered approximately 17½ feet of very loose to medium
dense sand and silty sand fill with varying amounts of gravel associated with the railroad embankment. Borings B-87 and B-89 also encountered loose silty sand fill to depths of 9 and 2 feet, respectively. The loose surficial sand with silt encountered in boring B-2 may also be fill associated with the railroad embankment.

Peat was encountered below the fill in boring B-87 and extends to a depth of about 19 feet below the ground surface at the time the boring was drilled. No peat was encountered in borings B-1 through B-3 and in boring B-89. It is possible the peat was removed during construction of the railroad embankment.

Loose to medium dense sand with varying amounts of silt, gravel and cobbles underlies the fill and peat, where present. This soil unit extends to depths of about 26 to 43 feet in the recent and previous borings. We interpret these soils to be recessional outwash deposits.

Very stiff to hard silt and clay and medium dense to very dense sand and gravel with varying amounts of silt and cobbles underlie the loose to medium dense sand unit, and extend to the bottom of the borings. These soils could represent glacial till or transitional deposits.

**Groundwater Conditions**

Groundwater was observed during drilling at depths ranging from approximately 6 to 17½ feet below the existing ground surface. The groundwater conditions observed during drilling are presented on the boring logs.

Groundwater conditions observed while completing the explorations represent a short-term condition and may or may not be representative of the long-term groundwater conditions at the site. In lower permeability soils the depth at which the groundwater is initially encountered may be many feet below the long-term groundwater level measure in monitoring wells over an extended time.

Monitoring wells were installed in two of the borings, B-2 and B-4, to depths of 20 and 25 feet, respectively. 1 Alliance Geomatics surveyed the tops of the wells in February 2017.

Table 1 provides a summary of groundwater measurements completed on February 16 and May 3, 2017.

**TABLE 1. SUMMARY OF GROUNDWATER MEASUREMENTS**

<table>
<thead>
<tr>
<th>Well ID</th>
<th>Ground Surface Elevation (feet)</th>
<th>Top of Casing Elevation (feet)</th>
<th>Measured Groundwater Elevation (feet) / Depth Below Ground Surface (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>2/16/2017</td>
</tr>
<tr>
<td>B-2</td>
<td>129.92</td>
<td>129.69</td>
<td>126.58/0.23</td>
</tr>
<tr>
<td>B-4</td>
<td>144.48</td>
<td>144.61</td>
<td>130.51/13.97</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>5/3/2017</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>125.79/4.13</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>127.69/16.79</td>
</tr>
</tbody>
</table>

Additional groundwater measurements will be taken during the design phase of the project to further assess variations in groundwater elevations. Groundwater levels are anticipated to fluctuate as a function of precipitation and season.

The monitoring wells are the property of the City of Kirkland. The wells should be decommissioned by a licensed well driller in accordance with Chapter 173-160 of the Washington Administrative Code (WAC)
CONCLUSIONS AND RECOMMENDATIONS

A summary of primary geotechnical considerations for the 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.

- Based on the borings completed to date, the site is designated as seismic Site Class D per the Washington State Department of Transportation (WSDOT) Geotechnical Design Manual (GDM) and AASHTO LRFD. However, borings were not completed within the wetland area such that Site Class E or F may be appropriate within this portion of the site. Additional explorations will be completed during final design, and a site-specific seismic response analysis may be appropriate depending on the subsurface conditions encountered.

- Effective stress liquefaction analysis was completed to better characterize the liquefaction susceptibility of the site soils and the anticipated settlement resulting from liquefaction. As summarized in a subsequent section, estimated ground settlements resulting from liquefaction of the subsurface soils during the design earthquake range from 0 in the southwest, to 6 to 9 inches in the northeast.

- We understand the “Skipping Stone” design alternative has been selected which includes 10 individual piers and a western and eastern abutment. Based on the preliminary subsurface soil conditions encountered and the bridge demands, large diameter drilled shaft foundations will provide suitable support for the bridge. The west abutment will require two shafts and the east abutment near the end of the spiral ramp will likely require three shafts. The remaining foundations will be single shafts to support the Y-piers. Recommendations for axial, compression and lateral capacities are discussed in subsequent sections.

- The approach embankment from the west will extend from the existing trail and up the adjacent slope to connect with the bridge alignment. MSE walls are planned to support the approach in this area. A shorter embankment is required at the north abutment, extending from the spiral terminus to the existing trail. Lightweight fill may be utilized in this area to mitigate settlement, depending on the subsurface findings in the final borings.

- Green stormwater Infiltration may be feasible in the granular soils encountered above the wetland, depending on the location of the facility, long-term groundwater monitoring results, and other regulatory requirements. A lower infiltration rate is available in the south and central site areas based on the fines content encountered in the preliminary borings. Additional evaluation will be completed during final design to support drainage design.

These and other geotechnical considerations are discussed further, and recommendations pertaining to the geotechnical aspects of the project are presented in the following sections of this report.
Earthquake Engineering

Ground Motion Parameters

The seismic design of the bridge should be completed using the design criteria presented in the 7th Edition of the AASHTO LRFD Bridge Design Specifications (2014). This document references the 2008 USGS National Seismic Hazards Mapping project for determining spectral acceleration coefficients (bedrock) for design. The acceleration coefficients for design are based on the expected ground motion at the project site that has a 7 percent probability of exceedance in a 75-year period (approximate 975-year return period).

Based on the calculated Vs30 from the borings completed, the site is classified as Site Class D. However, it is likely that a greater thickness of soft/loose soils are present in the wetland area resulting in Site Class E. Additional borings will be completed during final design to confirm the site class in this area. Recommended seismic parameters are provided in Table 2.

<table>
<thead>
<tr>
<th>AASHTO Seismic Parameter</th>
<th>Recommended Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site Class</td>
<td>D</td>
</tr>
<tr>
<td></td>
<td>E</td>
</tr>
<tr>
<td>S_s</td>
<td>1.25</td>
</tr>
<tr>
<td>S_1</td>
<td>0.48</td>
</tr>
<tr>
<td>Zero-period Site Factor, F_pga</td>
<td>1.13</td>
</tr>
<tr>
<td>Short Period Site Factor, F_s</td>
<td>1.00</td>
</tr>
<tr>
<td>Long-period Site Factor, F_v</td>
<td>1.52</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>AASHTO Seismic Parameter</th>
<th>Recommended Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short Period Site Factor, F_s</td>
<td>1.00</td>
</tr>
<tr>
<td>Long-period Site Factor, F_v</td>
<td>2.40</td>
</tr>
</tbody>
</table>

Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from liquefaction. In general, structures supported on liquefied soils could suffer foundation settlement, downdrag loads, or lateral movement that could be severely damaging to the structures.

Conditions favorable to liquefaction typically occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Based on our evaluation of the subsurface conditions at the site, we conclude that portions of the subsurface soils exhibit characteristics of liquefiable soils and will likely undergo some level of strength loss during the design-level earthquake event (peak ground acceleration [PGA] value and mean earthquake Magnitude as presented in Table 3 below). The PGA value was determined using the PGA from the 2008 USGS probabilistic seismic hazard deaggregation at 975-year return period multiplied by the site amplification factor presented in AASHTO LRFD Bridge Design Specifications (2014). For the design magnitude, we selected the mean magnitude based on the results of the 2008 USGS probabilistic seismic hazard deaggregation.
TABLE 3. EARTHQUAKE DESIGN PARAMETERS FOR LIQUEFACTION ANALYSIS

<table>
<thead>
<tr>
<th>Return Period</th>
<th>Mean Magnitude</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Site Class D</td>
</tr>
<tr>
<td>975 Years</td>
<td>6.76</td>
<td>0.37</td>
</tr>
</tbody>
</table>

Liquefaction triggering is typically evaluated using semi-empirical methods (i.e. simplified methods) based on in situ field tests such as standard penetration test (SPT), cone penetration test (CPT), or shear wave velocity measurements. The simplified methods of liquefaction evaluation are based on comparing the earthquake induced loading to the soil resistance to triggering liquefaction. The earthquake induced loading is called the cyclic stress ratio (CSR) and the soil resistance is the cyclic resistance ratio (CRR). The borings (B-1 to B-5) completed at the site with SPT measurements were evaluated using the simplified triggering criteria proposed by Youd and Idriss (2001) based on a mean magnitude 6.76 design earthquake event (975 years) with a PGA of 0.42g and 0.37g.

Table 4 summarizes the depth ranges of liquefiable soil across project site based on the simplified liquefaction evaluation method used in this study for Site Classes D and E.

TABLE 4. POTENTIAL LIQUEFIABLE ZONES

<table>
<thead>
<tr>
<th>Boring</th>
<th>Depth Range (feet) (Site Class D)</th>
<th>Depth Range (feet) (Site Class E)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>0 to 22.5</td>
<td>0 to 17.5</td>
</tr>
<tr>
<td>B-2</td>
<td>0 to 27.5</td>
<td>0 to 27.5</td>
</tr>
<tr>
<td>B-3</td>
<td>12.5 to 28 and 42.5 to 47.5</td>
<td>12.5 to 28</td>
</tr>
<tr>
<td>B-4</td>
<td>22.5 to 27.5</td>
<td>22.5 to 27.5</td>
</tr>
<tr>
<td>B-5</td>
<td>No Liquefiable Soils</td>
<td>No Liquefiable Soils</td>
</tr>
</tbody>
</table>

Liquefaction-Induced Ground Settlement

The magnitude of liquefaction-induced ground settlement was computed using the Youd and Idriss (2001) simplified approach described previously. Reconsolidation settlement (volumetric strain) is estimated as a function of the factor of safety of liquefaction triggering (serving as a proxy for the maximum accumulated shear strain).

Table 5 below summarizes the range of estimated settlement across the project site for the analysis methods used in this study. The settlement ranges represent estimated ground settlement from the soils encountered from the ground surface to the depth of boring.

TABLE 5. LIQUEFACTION-INDUCED GROUND SETTLEMENT

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Depth of Boring (feet)</th>
<th>Estimated Settlement (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>71.5</td>
<td>6-9</td>
</tr>
<tr>
<td>B-2</td>
<td>71.0</td>
<td>5-8</td>
</tr>
<tr>
<td>Boring No.</td>
<td>Depth of Boring (feet)</td>
<td>Estimated Settlement (inches)</td>
</tr>
<tr>
<td>-----------</td>
<td>------------------------</td>
<td>------------------------------</td>
</tr>
<tr>
<td>B-3</td>
<td>61.5</td>
<td>3.6</td>
</tr>
<tr>
<td>B-4</td>
<td>66.5</td>
<td>1.2</td>
</tr>
<tr>
<td>B-5</td>
<td>31.5</td>
<td>0</td>
</tr>
</tbody>
</table>

**Ground Rupture**

The closest mapped fault in the vicinity is located roughly 1 mile north, and is designated as a “Class B” fault by Washington Department of Natural Resources. Class B faults or fault systems are those which Quaternary-age deformation is suspected, but insufficient evidence has been gathered to support this determination. The uncertain traces mapped are part of the Southern Whidbey Island fault zone. Based on the available data, the risk of adverse impacts resulting from seismically induced slope instability, differential settlement, or surface displacement due to faulting is considered to be low.

**Bridge Foundation**

Based on the subsurface soil conditions encountered and bridge demands, large diameter shaft foundations will provide suitable support for the bridge. The west abutment will require two shafts and the east abutment near the end of the spiral ramp will likely require three shafts. The remaining foundations will be single shafts to support the Y-piers. Mechanically stabilized earth (MSE) walls will be constructed to retain the west approach ramp, and smaller walls may also be necessary at the east ramp between the east abutment and the existing trail. Foundation recommendations are discussed in detail below.

**Drilled Shafts**

**Axial Capacity**

We understand the bridge will be supported on 14 or 15 large diameter drilled shafts. Interior piers will be supported on single shafts and the abutments will be supported on two or three shafts. We evaluated axial shaft capacity using the methods presented in the 7th Edition of the AASHTO LRFD Bridge Design Specifications (2014 with the 2016 Interim Revisions). For Phase 1 evaluation, we developed a single, simplified soil profile using specific soil information at borings B-1, B-4, and B-5 to represent the north, central, and south portions of the bridge. Three diameters (4, 5, and 6 feet) were included for analyzing preliminary capacities.

The compressive and uplift axial shaft capacities for Service, Strength and Extreme Limit loading states are provided in Figures 3 through 11, Axial Resistance Plots. The capacity values presented in this report assume that 8 feet of permanent steel casing will be installed in the upper portion of the shaft. The axial and lateral soil strengths used to evaluate the capacity of the drilled shafts assume the range of soil liquefaction estimated from the simplified liquefaction analysis using Site Class D PGA. Axial reduction factors for group effects should be considered if multiple shafts are spaced closer than three shaft diameters on center.

We have incorporated the resistance factors presented in Table 6 into our drilled shaft foundation capacity charts. The capacity charts also assume that single drilled shafts (i.e. no redundant shafts) are present at each bent by applying 20 percent reduction in capacity. The service limit state condition assumes 1 inch of settlement at the top of the drilled shaft.
TABLE 6. LRFD DRILLED SHAFT FOUNDATION RESISTANCE FACTORS

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Skin Friction</th>
<th>End Bearing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength</td>
<td>0.55</td>
<td>0.50</td>
</tr>
<tr>
<td>Service</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Extreme</td>
<td>0.8</td>
<td>1.00</td>
</tr>
<tr>
<td>Uplift</td>
<td>0.45</td>
<td>-</td>
</tr>
</tbody>
</table>

**Lateral Capacity**

We understand that lateral capacity analyses of drilled shafts will be evaluated using the p-y curve method (LPile). Geotechnical design parameters for use in the evaluation of the lateral capacity of drilled shafts are presented in Table 7, which is located at the end of the report text. Additional parameters will be developed during final design when additional borings are completed. Preliminary LPile analyses runs were also provided to the structural engineer.

Shafts spaced closer than five shaft diameters (as measured center-to-center) apart will experience lateral group effects that will result in a lower lateral load capacity for trailing rows of shafts with respect to leading rows of shafts for an equivalent deflection. We recommend that the lateral load capacity for trailing shafts in a shaft group spaced less than five pile diameters apart be reduced in accordance with the factors in Table 8.

TABLE 8. SHAFT P-MULTIPLIERS, P_M, FOR MULTIPLE ROW SHADING

<table>
<thead>
<tr>
<th>Shaft Spacing¹ (in terms of shaft diameter)</th>
<th>P-Multipliers, P_m², ³</th>
<th>( P_m ) for Row 1 (leading row)</th>
<th>( P_m ) for Row 2 (1st trailing row)</th>
<th>( P_m ) for Row 3 and higher (2nd trailing row)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3D</td>
<td>0.85</td>
<td>0.65</td>
<td>0.55</td>
<td></td>
</tr>
<tr>
<td>5D</td>
<td>1.0</td>
<td>0.85</td>
<td>0.80</td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**

1. The \( P_m \) multipliers in the table above are a function of the center to center spacing of shafts in the group in the direction of loading expressed in multiples of the shaft diameter, \( D \).
2. The values of \( P_m \) were developed for vertical shafts only.
3. The \( P_m \) multipliers are dependent on the shaft spacing and the row number in the direction of the loading to establish values of \( P_m \) for other shaft spacing values, interpolation between values should be conducted.

**Construction Considerations**

Based on our understanding of the preliminary bridge demand loading, drilled shafts will likely extend to a depth of 60 to 70 feet below existing site grade within the spiral ramp area, and 40 to 50 feet along the remaining alignment. The drilled shafts will extend well below the static groundwater elevation. Based on the depth below grade, location of the static groundwater elevation, and proximity to utilities, we recommend that shafts be installed with temporary casing that extends to the dense silty sand or very stiff to hard silts and clay excavated using oscillator or rotator methods. This technology incorporates high torque casing oscillators and rotators to advance heavy wall steel casing into the ground concurrent with the excavation without any vibration or ground loss. A temporary work platform founded on a geogrid reinforced mat, or reaction piles may be necessary to support the oscillator rig and associated drilling equipment.
Drilled shafts should be excavated with equipment that reduces the amount of loose cuttings or slough at the bottom of the drilled hole. Slough and loose cuttings should be removed from the hole prior to placing the concrete. We recommend the drilled shafts be constructed using the tremie methods for concrete placement. Nondestructive testing of shafts using Cross-Hole Sonic Logging (CSL) and/or Thermal Integrity Profiling (TIP) is recommended for all drilled shafts completed for the project.

Though not encountered in our recent explorations, cobbles, boulders, and debris could be encountered within the soil profile. The contractor should be prepared to advance through and/or remove cobbles, boulders, and debris if encountered during drilled shaft construction.

**Bridge Approach**

Based on the preliminary plan, the south approach will begin about 500 feet southwest of NE 124th Street on the existing trail grade, and extend up through the slope cut to the existing bridge alignment. The bridge alignment is located roughly 15 to 30 feet away and parallel to the existing trail. A variable height MSE wall is planned to retain the ramp up the slope and to the south abutment. Maximum embankment height is anticipated to be about 15 feet.

The north approach will likely include low-height MSE walls to retain the embankment between the existing trail and the abutment within the spiral ramp. These walls are estimated to be less than 4 feet high and extend about 50 to 75 feet.

Based on the subsurface conditions encountered in our preliminary borings, conventional fill and construction methods are likely feasible for design and construction of the walls. Additional borings will be completed within the spiral ramp area to confirm soft compressible peat is not present. Lightweight fill or EPS geofoam may be considered for the north approach if necessary. Additional description of the MSE wall construction is provided below and additional design parameters will be provided during final design.

**MSE or Structural Earth Walls**

MSE (mechanically-stabilized earth) or structural earth (SE) walls are planned to retain the approach fill embankments. Global and internal stability of the wall should be evaluated during final design using the procedures outlined in section 15.5.3 of the 2015 WSDOT GDM and Section 11.10 of the 2012 AASHTO LRFD Bridge Design Specifications. We recommend a horizontal seismic force, $k_h$, equal to 0.5 times the PGA be considered when evaluating the wall components for seismic design.

Minimum embedment of MSE walls will be governed by the wall height, slope in front of the wall, retained soil and loading condition. The required embedment depth will be evaluated during final design. MSE walls should be designed for reinforcement pullout, reinforcement capacity, connection strength, sliding resistance, bearing resistance and over-turning using the various load and resistance factors consistent with the criteria presented in AASHTO LRFD Specifications. Minimum reinforcing length should be 0.7 times the wall height (top of wall to top of leveling pad) or 6 feet, whichever is greater. Pullout, over-turning, or other internal stability requirements will dictate longer reinforcement lengths for the variable loading and wall configuration.

**Drainage**

Wall drainage consisting of a minimum 6-inch-diameter perforated drain pipe embedded in drainage material should be incorporated into design of the walls. The drainage material and drain pipe should be
wrapped with a geotextile conforming to WSDOT 9-33 to reduce the potential for fines migration. The drain should be installed at the back of the reinforcement zone and be sloped to direct water into a storm drain system or other suitable discharge.

**Earthwork**

**Excavation Considerations**

Fill soils are present along the alignment associated with construction of the railroad embankment and adjacent streets. We anticipate that these soils can be excavated with conventional excavation equipment. The contractor should be prepared to deal with debris, cobbles and boulders, which are frequently encountered in uncontrolled fill soils.

**Clearing and Grubbing**

The existing ground surface along the project corridor is typically vegetated or paved as discussed in the “Surface Conditions” section of this report. Embankment areas covered with vegetation should be cleared and grubbed in accordance with Section 2-01 of the WSDOT Standard Specifications.

**Subgrade Preparation**

Following clearing and grubbing, we recommend the existing slope in the south approach area be cut into a series of horizontal benches to key in new fill and provide a horizontal stable surface for the foundation of new walls. Additional details and site preparation recommendations will be evaluated during final design when the final configuration of the alignment and walls are available.

Subgrade stabilization will be required to access the north site and wetland area to complete final explorations and to install drilled shafts. This may include the use of high-strength geotextiles and geogrids, light weight fill, or a stabilized platform with reaction piles. Detailed recommendations for site preparation and access road considerations will be provided during final design.

**Structural Fill Materials**

Materials used to construct the south access ramp and placed behind retaining structures is classified as structural fill for the purpose of this report. Structural fill material quality varies depending upon its use, as described below:

1. As a minimum, structural fill placed to construct embankments and the trail, to backfill utility trenches and to support foundations should meet the criteria for common borrow, WSDOT 9-03.14(3). Common borrow will be suitable for use as structural fill during dry weather conditions only. If structural fill is placed during wet weather, the structural fill should consist of gravel borrow, WSDOT 9-03.14(1).

2. Structural backfill for walls should meet the criteria for gravel borrow or gravel backfill for walls, WSDOT 9-03.12(2).

3. Structural fill placed to surround collector pipe (drain rock) should meet the criteria for gravel backfill for drains, WSDOT 9-03.12(4).

**On-site Soils**

The soils observed in the explorations generally contain a high percentage of fines (silt and clay) and are moisture-sensitive. Some of the on-site soils may meet the criteria for common borrow and may be suitable for use during dry weather construction only, provided the soil has a moisture content near optimum.
However, the fine-grained soils (silt and clay), or existing fill with wood or other debris do not meet the criteria for common borrow and should not be used. Peat and organic silt soils are unsuitable for use as structural fill.

**Fill Placement and Compaction Criteria**

Structural fill should be mechanically compacted to a firm, non-yielding condition. Structural fill should be placed in loose lifts not exceeding 1 foot in thickness. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. We recommend structural fill placed for the approach ramps be compacted to 95 percent of the maximum dry density (MDD) (ASTM D 1557).

We recommend that a representative of GeoEngineers be present during proof-rolling and/or probing of the exposed subgrade, and during placement of structural fill. GeoEngineers will evaluate the adequacy of the subgrade soils and identify areas needing further work, perform in-place moisture-density tests in the fill to evaluate whether the work is being done in accordance with the compaction specifications, and advise on any modifications to procedure that may be appropriate for the prevailing conditions.

**Weather Considerations**

The on-site soils generally contain a high percentage of fines (silt and clay) and are moisture-sensitive. When the moisture content of these soils is more than a few percent above the optimum moisture content, these soils become muddy and unstable, operation of equipment on these soils will be difficult, and it will be difficult or impossible to meet the required compaction criteria. Additionally, disturbance of near-surface soils should be expected if earthwork is completed during periods of wet weather. The contractor will need to take precautions to protect the subgrade during periods of wet weather.

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

- the ground surface in and around the work area should be sloped so that surface water is directed away from the work area. The ground surface should be graded such that areas of ponded water do not develop. The contractor should take measures to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area;
- erosion control techniques should be implemented to prevent sediment from leaving the site;
- earthwork activities should not take place during periods of heavy precipitation;
- slopes with exposed soils should be covered with plastic sheeting;
- the contractor should take necessary measures to prevent on-site soils and soils to be used as fill from becoming wet or unstable. These measures may include the use of plastic sheeting, sumps with pumps, and grading. The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will help reduce the extent that these soils become wet or unstable; and
- construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practical.
Erosion and Sedimentation Control

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. Implementing an erosion and sedimentation control plan will reduce the project impact on erosion-prone areas. The plan should be designed in accordance with applicable regulatory standards. The plan should incorporate basic planning principles including:

- scheduling grading and construction to reduce soil exposure;
- retaining existing vegetation whenever feasible;
- revegetating or mulching denuded areas;
- directing runoff away from denuded areas;
- reducing the length and steepness of slopes with exposed soils;
- decreasing runoff velocities;
- preparing drainage ways and outlets to handle concentrated or increased runoff;
- confining sediment to the project site; and
- inspecting and maintaining control measures frequently.

In addition, we recommend that slope surfaces in exposed or disturbed soil be restored so that surface runoff does not become channeled. Some sloughing and raveling of slopes with exposed or disturbed soil should be expected.

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

Temporary Cut Slopes

Temporary shallow cut slopes may be utilized around the site during construction. We recommend that temporary cut slopes be inclined no steeper than 1 1/2H:1V. Flatter slopes may be necessary if seepage is present on the cut face or if localized sloughing occurs. The above cut slope recommendation applies to fully dewatered conditions and is not appropriate for deep excavations in the spiral ramp area. If excavations are required in the spiral ramp area, we should be contacted for site specific shoring or appropriate excavation configurations. In other areas, additional recommendations for open cuts include:

- no traffic, construction equipment, stockpiles or building supplies be allowed at the top of the cut slopes within a distance of at least 10 feet from the top of the cut;
- exposed soil along the slope be protected from surface erosion during periods of wet weather using waterproof tarps, visqueen or flashcoating with shotcrete;
- construction activities be scheduled so that the length of time the temporary cut is left open is reduced to the extent practical;
erosion control measures be implemented as appropriate such that runoff from the site is reduced to the extent practical;

surface water is diverted away from the excavation; and

the general condition of the slopes be observed periodically by a geotechnical engineer to confirm adequate stability.

Since the contractor has control of the construction operations, the contractor should be made responsible for the stability of cut slopes, as well as the safety of the excavations.

Construction Vibrations and Pre-Construction Surveys of Adjacent Buildings

We recommend that a detailed pre-construction condition damage survey of nearby structures be completed to document structural and cosmetic building conditions. This should include photographs, videotaping and other means to establish existing conditions and actual vibration induced damages. We also recommend taking base line vibration measurements prior to any construction or rerouting of traffic to determine what average vibrations typical traffic patterns, including railroad traffic, create in the project area.

Survey reference points should be established on nearby buildings and surveys should be completed before and during construction to determine if any settlement of the structures has occurred. We also recommend performing vibration monitoring at or immediately outside these buildings to document actual vibrations experienced during the work.

Addition details regarding vibration monitoring and pre-construction surveys will be provided during final design.

RECOMMENDATIONS FOR FUTURE SERVICES

Throughout this report we have made recommendations for additional field explorations, analyses, and recommendations that should be completed in support of the final design. Generally, these items are listed below:

- Drill final geotechnical borings for the drilled shaft bridge foundation locations. AASHTO and the GDM require that borings be located at each foundation element and that they extend at least 20 feet beneath the final shaft tip.

- Provide drilled shaft axial and lateral capacity charts that are developed for each foundation element based on the new and existing boring and laboratory information.

- Prepare lateral earth pressure diagrams for site walls, and provide wall design parameters for the approach ramps.

- Prepare supporting information and requirements for the development of a plan to monitor vibrations during construction. This includes setting peak particle velocity thresholds for various existing features in the project vicinity.
■ Perform additional evaluations in association with the proposed geotechnical borings in order to further evaluate the potential for green stormwater infiltration potential and to support final design of such elements if necessary.

LIMITATIONS

We have prepared this report for the exclusive use of the City of Kirkland, COWI North America, Inc., and their authorized agents in the preliminary design of the Totem Lake Connector project in Kirkland, Washington. The data and report should be provided to prospective contractors for their bidding or estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

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.

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

Please refer to Appendix D titled “Report Limitations and Guidelines for Use” for additional information pertaining to use of this report.

REFERENCES


Converse Consultants NW, 1988, “Report of Geotechnical Exploration, Redmond Connection Project, King County, Washington.”


Washington State Department of Natural Resources. Subsurface Geology Information System Mapping Application, at https://fortress.wa.gov/dnr/protectiongis/geology/?Theme=subsurf


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

Data Source: Mapbox Open Street Map, 2016
Projection: NAD 1983 UTM Zone 10N

Vicinity Map
Totem Lake Pedestrian Bridge
Kirkland, Washington

Figure 1
Figure 2

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

Data Source: Background from COWI North America, Inc. dated 02/22/17.
Vertical Datum: MLLW (NAVD 88).
The service case assumes 1-inch and 2-inch of shaft settlement. The plots include resistance factors shown on the adjacent table for Strength, Extreme, and Service Limit States. Resistance factors for Strength Limit State do not include a 20 percent reduction for non-redundant shafts.

Unfactored downdrag load for the Extreme Limit State is estimated to be 40 kips. A load factor of 1.0 is recommended to be applied with post-earthquake loading conditions in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual.
Axial shaft resistance was developed in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual and the 2015 WSDOT Geotechnical Design Manual (GDM).

1. The service case assumes 1-inch and 2-inch of shaft settlement.
2. The plots are based on a single shaft and do not consider group effects of closely spaced shafts.
3. The plots include resistance factors shown on the adjacent table for Strength, Extreme, and Service Limit States. Resistance factors for Strength Limit State do not include a 20 percent reduction for non-redundant shafts.
4. Unfactored downdrag load for the Extreme Limit State is estimated to be 50 kips. A load factor of 1.0 is recommended to be applied with post-earthquake loading conditions in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual.
The service case assumes 1-inch and 2-inch of shaft settlement. The plots include resistance factors shown on the adjacent table for Strength, Extreme, and Service Limit States. Resistance factors for Strength Limit State do not include a 20 percent reduction for non-redundant shafts.

Unfactored downdrag load for the Extreme Limit State is estimated to be 60 kips. A load factor of 1.0 is recommended to be applied with post-earthquake loading conditions in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual.

General Notes
1. Axial shaft resistance was developed in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual and the 2015 WSDOT Geotechnical Design Manual (GDM).
2. The axial resistance plots assume a top of shaft Elevation of 128.5 feet and permanent steel casing to a depth of 8 below the ground surface. The geotechnical engineer should re-evaluate axial shaft resistance if permanent steel casing length or top of shaft Elevation changes.
3. The plots are based on a single shaft and do not consider group effects of closely spaced shafts.
4. The service case assumes 1-inch and 2-inch of shaft settlement.
5. The plots include resistance factors shown on the adjacent table for Strength, Extreme, and Service Limit States. Resistance factors for Strength Limit State do not include a 20 percent reduction for non-redundant shafts.

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1. Axial shaft resistance was developed in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual and the 2015 WSDOT Geotechnical Design Manual (GDM).

2. The axial resistance plots assume a top of shaft Elevation of 144.8 feet and permanent steel casing to a depth of 8 below the ground surface. The geotechnical engineer should re-evaluate axial shaft resistance if permanent steel casing length or top of shaft Elevation changes.

3. The plots are based on a single shaft and do not consider group effects of closely spaced shafts.

4. The service case assumes 1-inch and 2-inch of shaft settlement.

5. The plots include resistance factors shown on the adjacent table for Strength, Extreme, and Service Limit States. Resistance factors for Strength Limit State do not include a 20 percent reduction for non-redundant shafts.

6. Unfactored downdrag load for the Extreme Limit State is estimated to be 317 kips. A load factor of 1.0 is recommended to be applied with post-earthquake loading conditions in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual.
1. Axial shaft resistance was developed in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual and the 2015 WSDOT Geotechnical Design Manual (GDM).
2. The axial resistance plots assume a top of shaft Elevation of 144.8 feet and permanent steel casing to a depth of 8 below the ground surface. The geotechnical engineer should re-evaluate axial shaft resistance if permanent steel casing length or top of shaft Elevation changes.
3. The plots are based on a single shaft and do not consider group effects of closely spaced shafts.
4. The service case assumes 1-inch and 2-inch of shaft settlement.
5. The plots include resistance factors shown on the adjacent table for Strength, Extreme, and Service Limit States. Resistance factors for Strength Limit State do not include a 20 percent reduction for non-redundant shafts.
6. Unfactored downdrag load for the Extreme Limit State is estimated to be 396 kips. A load factor of 1.0 is recommended to be applied with post-earthquake loading conditions in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual.

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The service case assumes 1-inch and 2-inch of shaft settlement. The plots include resistance factors shown on the adjacent table for Strength, Extreme, and Service Limit States. Resistance factors for Strength Limit State do not include a 20 percent reduction for non-redundant shafts.

Unfactored downdrag load for the Extreme Limit State is estimated to be 475 kips. A load factor of 1.0 is recommended to be applied with post-earthquake loading conditions in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual.

### General Notes
1. Axial shaft resistance was developed in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual and the 2015 WSDOT Geotechnical Design Manual (GDM).
2. The axial resistance plots assume a top of shaft Elevation of 144.8 feet and permanent steel casing to a depth of 8 below the ground surface. The geotechnical engineer should re-evaluate axial shaft resistance if permanent steel casing length or top of shaft Elevation changes.
3. The plots are based on a single shaft and do not consider group effects of closely spaced shafts.
4. The service case assumes 1-inch and 2-inch of shaft settlement.
5. The plots include resistance factors shown on the adjacent table for Strength, Extreme, and Service Limit States. Resistance factors for Strength Limit State do not include a 20 percent reduction for non-redundant shafts.

### Resistance Factors

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### Subsurface Profile

- Soil Types: SM, GC, ML, GM, CL

### Axial Resistance Plots

- **Strength Limit State**: Factored Axial Resistance (kips)
- **Extreme Limit State**: Factored Axial Resistance (kips)
- **Service Limit State**: Factored Axial Resistance (kips)
1. Axial shaft resistance was developed in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual and the 2015 WSDOT Geotechnical Design Manual (GDM).

2. The axial resistance plots assume a top of shaft Elevation of 146.7 feet and permanent steel casing to a depth of 8 below the ground surface. The geotechnical engineer should re-evaluate axial shaft resistance if permanent steel casing length or top of shaft Elevation changes.

3. The plots are based on a single shaft and do not consider group effects of closely spaced shafts.

4. The service case assumes 1-inch and 2-inch of shaft settlement.

5. The plots include resistance factors shown on the adjacent table for Strength, Extreme, and Service Limit States. Resistance factors for Strength Limit State do not include a 20 percent reduction for non-redundant shafts.

6. Unfactored downdrag load for the Extreme Limit State is estimated to be 0 kips. A load factor of 1.0 is recommended to be applied with post-earthquake loading conditions in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual.

### Resistance Factors

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Side</th>
<th>Uplift</th>
<th>End</th>
<th>Comp.</th>
<th>Uplift</th>
<th>Service</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>0.55</td>
<td>0.45</td>
<td>0.5</td>
<td>1</td>
<td>0.8</td>
<td>1</td>
</tr>
<tr>
<td>Clay</td>
<td>0.45</td>
<td>0.35</td>
<td>0.4</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rock</td>
<td>0.55</td>
<td>0.4</td>
<td>0.5</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
1. Axial shaft resistance was developed in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual and the 2015 WSDOT Geotechnical Design Manual (GDM).

2. The axial resistance plots assume a top of shaft Elevation of 146.7 feet and permanent steel casing to a depth of 8 below the ground surface. The geotechnical engineer should re-evaluate axial shaft resistance if permanent steel casing length or top of shaft Elevation changes.

3. The plots are based on a single shaft and do not consider group effects of closely spaced shafts.

4. The service case assumes 1-inch and 2-inch of shaft settlement.

5. The plots include resistance factors shown on the adjacent table for Strength, Extreme, and Service Limit States. Resistance factors for Strength Limit State do not include a 20 percent reduction for non-redundant shafts.

6. Unfactored downdrag load for the Extreme Limit State is estimated to be 0 kips. A load factor of 1.0 is recommended to be applied with post-earthquake loading conditions in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual.
General Notes

1. Axial shaft resistance was developed in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual and the 2015 WSDOT Geotechnical Design Manual (GDM).

2. The axial resistance plots assume a top of shaft Elevation of 146.7 feet and permanent steel casing to a depth of 8 below the ground surface. The geotechnical engineer should re-evaluate axial shaft resistance if permanent steel casing length or top of shaft Elevation changes.

3. The plots are based on a single shaft and do not consider group effects of closely spaced shafts.

4. The service case assumes 1-inch and 2-inch of shaft settlement.

5. The plots include resistance factors shown on the adjacent table for Strength, Extreme, and Service Limit States. Resistance factors for Strength Limit State do not include a 20 percent reduction for non-redundant shafts.

6. Unfactored downdrag load for the Extreme Limit State is estimated to be 0 kips. A load factor of 1.0 is recommended to be applied with post-earthquake loading conditions in accordance with the 2014-2015 AASHTO LRFD Bridge Design Manual.
APPENDIX A
Field Explorations
APPENDIX A
FIELD EXPLORATIONS

We evaluated subsurface soil and groundwater conditions along the Totem Lake Connector project alignment by drilling seven borings (B-1 through B-7) at the approximate locations shown on the Site Plan, Figure 2.

Geologic Drill Exploration, Inc. completed the drilling on January 30 through February 2, 2017 under subcontract to GeoEngineers. The borings were drilled to depths ranging from about 21½ to 71½ feet below the existing ground surface.

Exploration locations, ground surface elevations and monitoring well elevations were surveyed by Alliance Geomatics following drilling. Exploration locations and elevations should be considered accurate to the degree implied by the methods used.

The explorations were continuously observed by a geotechnical engineer who evaluated and classified the soils encountered, obtained representative soil samples, and observed groundwater conditions. Our representative maintained a detailed log of each exploration.

Disturbed samples of the representative soil types were obtained from the borings using standard penetration test (SPT) sampling procedures. SPT sampling was performed using a 2-inch outside diameter split-spoon sampler driven with a standard 140-pound hammer in accordance with ASTM D 1586. The soils encountered in the borings were typically sampled at 2½- to 5-foot vertical intervals with the SPT split spoon sampler.

Samples were obtained by driving the sampler into the soil with the hammer free-falling 30 inches. The number of blows required to drive the sampler the for each 6 inches of penetration was recorded. The blow count (“N-value”) of the soil was calculated as the number of blows required for the final 12 inches of penetration. Where very dense or hard soils conditions precluded driving the full 18 inches, the number of blows for the partial penetration was noted on the logs.

Soils encountered in the borings were visually classified in the field using ASTM D 2488, which is summarized in Figure A-1. Logs of the borings are provided in Figures A-2 through A-8.

The logs reflect our interpretation of the field conditions and the results of geotechnical laboratory evaluation and testing of samples. They also indicate the depths at which the soil types or their characteristics change, although the change may be gradual. If the change occurred between samples, it was interpreted. The densities noted on the boring logs are inferred from the blow count data and judgment based on the conditions encountered.

The soil samples were logged, sealed in plastic bags and transported to our Redmond geotechnical laboratory. Field soil classifications were further evaluated in our laboratory.

Observations of groundwater conditions were made during drilling and are included on the boring logs. These observations represent a short-term condition and may or may not be representative of the long-term groundwater conditions at the site. Groundwater conditions observed during drilling should be considered approximate.
Monitoring wells (2-inch diameter) were installed in borings B-2 and B-4 to allow measurement of groundwater levels following drilling. We measured groundwater levels in the wells on February 16, 2017. The groundwater level measurements are indicated on the boring logs.

The monitoring wells are the property of the City of Kirkland. The wells should be decommissioned by a licensed well driller in accordance with Chapter 173-160 of the Washington Administrative Code (WAC) when they are no longer needed for data collection. Alternatively, the wells could be kept intact for use during project bidding and then be decommissioned under the construction contract.

Soil cuttings generated from drilling were transported for eventual disposal at an off-site facility.
### Soil Classification Chart

<table>
<thead>
<tr>
<th>MAJOR DIVISIONS</th>
<th>SYMBOLS</th>
<th>TYPICAL DESCRIPTIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>COARSE GRAINED SOILS</strong></td>
<td>GW</td>
<td>WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES</td>
</tr>
<tr>
<td></td>
<td>GP</td>
<td>POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES</td>
</tr>
<tr>
<td></td>
<td>GM</td>
<td>SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES</td>
</tr>
<tr>
<td></td>
<td>GC</td>
<td>CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES</td>
</tr>
<tr>
<td><strong>SAND AND SANDY SOILS</strong></td>
<td>SW</td>
<td>WELL-GRADED SANDS, GRAVELLY SANDS</td>
</tr>
<tr>
<td></td>
<td>SP</td>
<td>POORLY-GRADED SANDS, GRAVELLY SAND</td>
</tr>
<tr>
<td><strong>SANDS WITH FINES</strong></td>
<td>SM</td>
<td>SILTY SANDS, SAND - SILT MIXTURES</td>
</tr>
<tr>
<td><strong>FINE GRAINED SOILS</strong></td>
<td>ML</td>
<td>INORGANIC SILTS; ROCK FLOUR, CLAYEY SILTS WITH APPRECIABLE PLASTICITY</td>
</tr>
<tr>
<td></td>
<td>CL</td>
<td>INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYEY CLAYS, SILT CLAYS, CLEAN CLAY</td>
</tr>
<tr>
<td><strong>SILTS AND CLAYS</strong></td>
<td>OL</td>
<td>ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY</td>
</tr>
<tr>
<td><strong>MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE</strong></td>
<td>MH</td>
<td>INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS</td>
</tr>
<tr>
<td><strong>MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE</strong></td>
<td>CH</td>
<td>INORGANIC CLAYS OF HIGH PLASTICITY</td>
</tr>
<tr>
<td><strong>HIGHLY ORGANIC SOILS</strong></td>
<td>OH</td>
<td>ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY</td>
</tr>
<tr>
<td></td>
<td>PT</td>
<td>PEAT, RUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS</td>
</tr>
</tbody>
</table>

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

### Additional Material Symbols

<table>
<thead>
<tr>
<th>SYMBOLS</th>
<th>TYPICAL DESCRIPTIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>AC</td>
<td>Asphalt Concrete</td>
</tr>
<tr>
<td>CC</td>
<td>Cement Concrete</td>
</tr>
<tr>
<td>CR</td>
<td>Crushed Rock/Quarry Spalls</td>
</tr>
<tr>
<td>SOD</td>
<td>Sod/Forest Duff</td>
</tr>
<tr>
<td>TS</td>
<td>Topsoil</td>
</tr>
</tbody>
</table>

### Groundwater Contact
- Measured groundwater level in exploration, well, or piezometer

### Graphic Log Contact
- Distinct contact between soil strata
- Approximate contact between soil strata

### Material Description Contact
- Contact between geologic units
- Contact between soil of the same geologic unit

### Laboratory / Field Tests
- %F: Percent fines
- %G: Percent gravel
- AL: Atterberg limits
- CA: Chemical analysis
- CP: Laboratory compaction test
- CS: Consolidation test
- DD: Dry density
- DS: Direct shear
- HA: Hydrometer analysis
- MC: Moisture content
- MD: Moisture density
- Mohs: Mohs hardness scale
- OC: Organic content
- PM: Permeability or hydraulic conductivity
- PI: Plasticity index
- PP: Pocket penetrometer
- SA: Sieve analysis
- TX: Triaxial compression
- UC: Unconfined compression
- VS: Vane shear

### Sheen Classification
- NS: No Visible Sheen
- SS: Slight Sheen
- MS: Moderate Sheen
- HS: Heavy Sheen

### Sampler Symbol Descriptions
- 2.4-inch I.D. split barrel
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- Direct-Push
- Bulk or grab
- Continuous Coring

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

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

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

**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**

Figure A-1
Notes:

1/30/2017 71.5 EF

1/30/2017 71.5 EF

Hammer Data

Automatic 140 (lbs) / 30 (in) Drop

Drilling Equipment D-50 Track Rig

Driller noted gravel at 22 feet

Driller noted easier drilling at 27 feet

Groundwater observed at 6 feet at time of exploration

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

Coordinates Data Source: Horizontal approximated based on Aerial Imagery, Vertical approximated based on DEM

Log of Boring B-1

Project: Totem Lake Pedestrian Bridge

Project Location: Kirkland, Washington

Project Number: 0231-090-00
Auger refusal at 42½ feet; moved 3 feet to the north and drilled to 45 feet.

1 foot of heave

Gray fine to coarse sand with gravel and cobbles (dense to very dense, wet)

Gray fine sand with silt and gravel (very dense, wet)

Gray fine to medium sand with gravel (very dense, wet)

Log of Boring B-1 (continued)

Project: Totem Lake Pedestrian Bridge
Project Location: Kirkland, Washington
Project Number: 0231-090-00
Brown fine to medium sand with occasional gravel (loose, wet)

Brown fine to medium sand with silt (medium dense, wet)

Brown fine to medium sand with occasional gravel (loose to medium dense, wet)

Gray silty fine to coarse sand and brown silt with sand, gravel and organic matter (loose/medium stiff, wet)

Orange-brown silty fine to coarse gravel with sand and cobbles (medium dense, wet)

Steel surface monument

Concrete surface seal

3/8-inch bentonite seal

2-inch Schedule 40 PVC well casing

2-inch Schedule 40 PVC screen, 0.020-inch slot width

2-inch Schedule 40 PVC end cap

Log of Boring with Monitoring Well B-2

Project: Totem Lake Pedestrian Bridge
Project Location: Kirkland, Washington
Project Number: 0231-090-00

Figure A-3
Sheet 1 of 2
Gray silt with sand and occasional gravel (very stiff, wet)

Gray silt with sand and gravel (very stiff, wet)

Orange-brown silt with sand and gravel (very stiff, wet)

Orange-brown silty fine to coarse gravel with sand and cobbles (medium dense, wet)

Monitoring well installed in separate boring drilled 5 feet to the southwest.

GEOENGINEERS

Log of Boring with Monitoring Well B-2 (continued)

Project Number: 0231-090-00

Project Location: Kirkland, Washington

Figure A-3

Redmond: Date: 5/22/17 Path: P:\0\0231090\GINT\023109000.GPJ DB\Templates\Lib\Templates\GEOENGINEERS_OF_STD_US_2017.GDT\GEI8_GEOTECH_WELL_%F

FIELD DATA

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<th>Interval</th>
<th>Recovered (in)</th>
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MATERIAL DESCRIPTION

<table>
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<tr>
<th>Moisture Content (%)</th>
<th>Fines Content (%)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

RECORD DATA

- Blows/foot
- Sample Name
- Testing
- Group Classification
- Water Level
- Graphic Log

WELL LOG

Monitoring well installed in separate boring drilled 5 feet to the southwest.
**Log of Boring B-3**

**Project:** Totem Lake Pedestrian Bridge  
**Project Location:** Kirkland, Washington  
**Project Number:** 0231-090-00

### FIELD DATA

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<th>Interval</th>
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<th>Recovered (in)</th>
<th>Blows/foot</th>
<th>Collected Sample</th>
<th>Classification</th>
<th>Group</th>
<th>Moisture Content (%)</th>
<th>Fines Content (%)</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>GP</td>
<td>9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>5</td>
<td>4</td>
<td>8</td>
<td>1 GP</td>
<td>Gray fine gravel with sand (medium dense, moist) (fill)</td>
<td>SP-5M</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>10</td>
<td>10</td>
<td>11</td>
<td>2 SM</td>
<td>Gray fine to medium sand with silt and gravel (loose, moist) (fill)</td>
<td>SM</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>12</td>
<td>12</td>
<td>3</td>
<td>3 SM</td>
<td>Grayish brown silty fine to medium sand with gravel (medium dense, moist) (fill)</td>
<td>SM</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>15</td>
<td>12</td>
<td>5</td>
<td>5 SM</td>
<td>Reddish brown silty fine to medium sand with occasional gravel (very loose to loose, moist) (fill)</td>
<td>SM</td>
<td></td>
<td>16</td>
<td>15</td>
</tr>
<tr>
<td>15</td>
<td>20</td>
<td>13</td>
<td>10</td>
<td>6 SM</td>
<td>Grades to orange-brown</td>
<td>SM</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>25</td>
<td>10</td>
<td>12</td>
<td>7 SM</td>
<td>Brown fine to medium sand with silt (loose to medium dense, wet)</td>
<td>SP-5M</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td>30</td>
<td>8</td>
<td>24</td>
<td>8 ME</td>
<td>Brown silty fine to medium sand with gravel (medium dense to dense, wet)</td>
<td>SM</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>35</td>
<td></td>
<td></td>
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<td></td>
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<td></td>
<td></td>
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</tbody>
</table>

**Notes:** See Figure A-1 for explanation of symbols.  
Coordinates Data Source: Horizontal approximated based on Aerial Imagery, Vertical approximated based on DEM.
<table>
<thead>
<tr>
<th>Interval</th>
<th>Recovered (in)</th>
<th>Blows/foot</th>
<th>Group</th>
<th>Classification</th>
<th>Moisture Content (%</th>
<th>Fines Content (%</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>16</td>
<td></td>
<td>CL</td>
<td>Brownish gray sandy clay with gravel (stiff to very stiff, wet)</td>
<td>18</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>22</td>
<td></td>
<td>SC</td>
<td>Orange-brown clayey fine to coarse sand with gravel (medium dense, wet)</td>
<td>18</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>25</td>
<td></td>
<td>GM</td>
<td>Orange-brown silty fine to coarse gravel with sand and cobbles (medium dense to dense, wet)</td>
<td>18</td>
<td>18</td>
<td></td>
</tr>
</tbody>
</table>

**Log of Boring B-3 (continued)**

Project: Totem Lake Pedestrian Bridge  
Project Location: Kirkland, Washington  
Project Number: 0231-090-00  

Figure A-4  
Sheet 2 of 2
Redmond: Date 5/22/17 Path: P:\0\0231090\01\0\023109000.GPJ DBTemplate\LibTemplate:GEOENGINEERS_DF_STD_US_2017.GDT/GEI8_GEOTECH_WELL_%F

Log of Boring with Monitoring Well B-4

Project: Totem Lake Pedestrian Bridge

Project Number: 0231-090-00

Log of Boring with Monitoring Well B-4

Project Location: Kirkland, Washington

Figure A-5

Dredge Well D-12: BK 734 (ft) well was installed on 2/7/2017 to a depth of 25 ft

Drill & Equipment: D-50 Track Rig

Driller: Automatic 144.18

Data: Surface Elevation (ft) 144.48

Notes: Groundwater seepage encountered at 18 feet during drilling.

Figure A-1 for explanation of symbols.

 Coordinates Data Source: Horizontal approximated based on Aerial Imagery, Vertical approximated based on DEM

Drilled 2/1/2017

Hammer Data

Date Measured

Horizontal Datum

Vertical Datum

DOE Well I.D.: BIK 734

A 2 (in) well was installed on 2/1/2017 to a depth of 25 ft.

2/16/2017

Easting (X)

Northing (Y)

Notes:

Surface Elevation (ft)

Automatic 140 (lbs) / 30 (in) Drop

144.61

140 (lbs) / 30 (in) Drop

144.48

NAVD88

1309737

WA State Plane North

261545.1

3/8-inch bentonite seal

2-inch Schedule 40 PVC well casing

Colorado silica sand backfill

2-inch Schedule 40 PVC screen, 0.020-inch slot width

2-inch Schedule 40 PVC end cap

Drilling Equipment

Top of Casing

Elevation (ft)

Groundwater Depth to Water (ft)

Logged By

HRP

End Checked By Driller

EF

Total Depth (ft)

66.5

Method

Drilling Hollow-stem Auger

Note: See Figure A-1 for explanation of symbols.
**Log of Boring B-5**

**Project:** Totem Lake Pedestrian Bridge  
**Project Location:** Kirkland, Washington  
**Project Number:** 0231-090-00

---

**Notes:**
- See Figure A-1 for explanation of symbols.
- Coordinates Data Source: Horizontal approximated based on Aerial Imagery, Vertical approximated based on DEM

---

**FIELD DATA**

<table>
<thead>
<tr>
<th>Elevation (feet)</th>
<th>Depth (feet)</th>
<th>Thickness (in)</th>
<th>Blows/foot</th>
<th>Sample Name</th>
<th>Testing Group</th>
<th>Sample Name Testing</th>
<th>Material Description</th>
<th>Moisture Content (%)</th>
<th>Fines Content (%)</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>18</td>
<td>12</td>
<td></td>
<td></td>
<td>GP</td>
<td></td>
<td>Gray fine gravel with sand (medium dense, moist) (fill)</td>
<td>15</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>26</td>
<td></td>
<td></td>
<td>SM</td>
<td></td>
<td>Brown silt to medium sand (medium dense, moist)</td>
<td>15</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>14</td>
<td></td>
<td></td>
<td>CL</td>
<td></td>
<td>Light gray lean clay with sand (stiff, moist)</td>
<td>25</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>20</td>
<td></td>
<td></td>
<td>ML</td>
<td></td>
<td>Light brown silt with sand and occasional gravel (very stiff, wet)</td>
<td>26</td>
<td>26</td>
<td>AL (LL = 45; PI = 14)</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>34</td>
<td></td>
<td></td>
<td>SC</td>
<td></td>
<td>Brown clayey fine to coarse sand with gravel (dense, wet)</td>
<td>16</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>23</td>
<td></td>
<td></td>
<td>ML</td>
<td></td>
<td>Gray sandy silt (very stiff, wet)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>9</td>
<td></td>
<td></td>
<td>CL</td>
<td></td>
<td>Gray clay with sand and occasional gravel (stiff, wet)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>12</td>
<td>6</td>
<td></td>
<td></td>
<td>CL</td>
<td></td>
<td>Gray sandy clay with gravel (hard, wet)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

**Notes:**
- Driller noted gravel at 27 feet
### Log of Boring B-6

<table>
<thead>
<tr>
<th>Interval</th>
<th>Recovered (in)</th>
<th>Blows/foot</th>
<th>Collected Sample</th>
<th>Sample Name</th>
<th>Testing</th>
<th>Classification</th>
<th>Group</th>
<th>Moisture Content (%)</th>
<th>Remarks</th>
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</thead>
<tbody>
<tr>
<td>0.00</td>
<td>18</td>
<td>29</td>
<td>GP</td>
<td>Fine gravel with sand (medium dense, moist) (fill)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
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<td></td>
<td></td>
<td>Gray silt with sand (very stiff, moist)</td>
<td>ML</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.00</td>
<td>18</td>
<td>48</td>
<td>ML</td>
<td>Gray silt with lenses of peat (hard, moist)</td>
<td>ML</td>
<td>Gray sandy silt (very stiff to hard, moist to wet)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Gray sandy silt (hard, wet)</td>
<td>ML</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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

Coordinates Data Source: Horizontal approximated based on Aerial Imagery, Vertical approximated based on DEM.
**Log of Boring B-7**

**Project:** Totem Lake Pedestrian Bridge  
**Project Location:** Kirkland, Washington  
**Project Number:** 0231-090-00

---

**MATERIAL DESCRIPTION**

<table>
<thead>
<tr>
<th>Interval</th>
<th>Recovered Sample</th>
<th>Blows/foot</th>
<th>Sample Name</th>
<th>Testing Group</th>
<th>Classification</th>
<th>Graphic Log</th>
<th>Moisture Content (%)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>GP</td>
<td></td>
<td>mind the</td>
</tr>
<tr>
<td>5 - 10</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>GM</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10 - 15</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>SM</td>
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</tr>
<tr>
<td>15 - 20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ML/PT</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20 - 25</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>ML</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**FIELD DATA**

- **Elevation (feet):**
  - 0 to 20

- **Surface Elevation (ft):**
  - 150.47

- **Vertical Datum:**
  - NAVD88

- **Hammer Data:**
  - Automatic
  - 140 (lbs) / 30 (in) Drop

- **Driller:**
  - Automatic

- **Photographic Log:**
  - Gray fine gravel with sand (medium dense, moist) (fill)
  - Brown silt fine to coarse gravel with sand (medium dense, moist) (fill)
  - Gray silt fine to coarse sand with gravel (dense, moist) (fill)
  - Gray sandy silt with gravel and lenses of peat (stiff to hard, moist)

- **Becomes wet**

- **Gray sandy silt (hard, wet)**

---

**Notes:**

- See Figure A-1 for explanation of symbols.
- Coordinates Data Source: Horizontal approximated based on Aerial Imagery, Vertical approximated based on DEM.

---

**Drilled**

- **Start:** 2/2/2017  
- **End:** 2/2/2017

- **Total Depth (ft):** 21.5

- **Logged By:**
  - Geologic Drill Exploration, Inc.

- **Checked By:**
  - HRP

- **EF:**
  - Automatic

- **Drilling Method:**
  - Hollow-stem Auger

- **Drilling Equipment:**
  - D-50 Track Rig

- **Groundwater observed at 12½ feet at time of exploration**
APPENDIX B
Laboratory Testing
APPENDIX B
LABORATORY TESTING

Soil samples obtained from the explorations were transported to our Redmond geotechnical laboratory and evaluated to confirm or modify field classifications, as well as to evaluate engineering properties of the soils. Representative samples were selected for laboratory testing that included moisture content, percent fines, grain size distribution (sieve analyses), and plasticity (Atterberg limits) tests. The tests were conducted using test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures.

Soil Classifications

All soil samples obtained from the explorations were visually classified in the field and/or in our laboratory using a system based on the Unified Soil Classification System (USCS) and ASTM classification methods. ASTM Test Method D 2488 was used to visually classify the soil samples, while ASTM D 2487 was used to classify the soils based on laboratory test results. These classification procedures are incorporated in the exploration logs presented as Figures A-2 through A-8 in Appendix A.

Moisture Content Tests

Moisture contents were measured using the ASTM D 2216 test method for several samples obtained from the explorations. The results of these tests are presented on the exploration logs (Appendix A) at the respective sample depths.

Percent Fines Tests

Tests to evaluate the percent fines (particles passing the No. 200 sieve) were completed on several soil samples using ASTM D 1140. The wet sieve method was used to determine the percentage of soil particles by weight larger than the U.S. No. 200 sieve opening. The results of the percent fines tests are presented on the exploration logs (Figures A-2 through A-8) at the depths at which the samples were obtained.

Sieve Analysis

Sieve analyses were performed on two samples obtained from the borings. The analyses were conducted using the ASTM D 6913 test method. The wet sieve analysis method was used to determine the percentage of soil particles by weight larger than the U.S. No. 200 mesh sieve. The results of the sieve analyses were plotted, classified using the USCS, and presented on Figure B-1.

Plasticity Characteristics

Plasticity characteristics of four soil samples were evaluated by conducting Atterberg limits tests using the ASTM D 4318 test method. This test method evaluates the liquid limit, plastic limit and plasticity index of the portion of the sample finer than the No. 40 sieve. The results of the Atterberg limits tests are presented in Figure B-2.
The grain size analysis results were obtained in general accordance with ASTM D 6913.

Note: This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc. Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations, or generated by separate operations or processes.
The liquid limit and plasticity index were obtained in general accordance with ASTM D 4318.
APPENDIX C
Previous Explorations
APPENDIX C
PREVIOUS EXPLORATIONS

Appendix C presents the logs of previous explorations by others along or near the project alignment, including three borings (B-87, B-89, and B-93) completed by Converse Consultants NW in 1987 for the Redmond Connection sewer force main project.
SUMMARY: BORING NO. B-87  

DATE DRILLED: 11/23/87  
ELEVATION: Approx. 130'

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

<table>
<thead>
<tr>
<th>DEPTH IN FEET</th>
<th>SAMPLE NO.</th>
<th>BLOWS</th>
<th>OTHER TESTS</th>
<th>FIELD MOISTURE</th>
<th>DRY DENSITY</th>
<th>PCF</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
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<td>15</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**DESCRIPTION**

**SILTY SAND W/GRAVEL (Fill); brown mottled rust, fine to medium sand, fine to coarse gravel, low plasticity**  
(SM) slightly moist; dense  
- encountered obstruction at 3'  
- increasing gravel  
moist; loose

**PEAT; brown, abundant wood fibers and fine roots, interbedded with silty sand, occasional laminations of organic silt**  
(Pt) wet; soft  
(Continued)

**SAND; description on following page**  
(SP) m.dense

---

* A. 2" split-spoon sampler  
B. 3" O.D. thin-wall sampler  
C. 3-1/4" O.D. x 2-1/2" liner  
D. 3-1/2" O.D. split barrel sampler  
X. Sample not recovered  
G. Grain size, T - triaxial, P - permeability  
W. Water level  
I. Impermeable seal  
P. Piezometer tip  

**PROJECT No.**  
Redmond Connection Project  
for HNTB  
86-35289-04  

**Drawing No.**  
A-40  

Converse Consultants  
Geotechnical Engineering  
and Applied Sciences
### Summary: Boring No. 8-78

**Elevation:**

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

#### Description

<table>
<thead>
<tr>
<th>Depth in Feet</th>
<th>Sample No.</th>
<th>Field Moisture</th>
<th>Dry Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>7A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>23</td>
<td>13</td>
<td>SP</td>
<td>SM</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>wet</td>
<td>medium</td>
</tr>
<tr>
<td>25</td>
<td>8A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>23</td>
<td>7</td>
<td>SP</td>
<td>SM</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>wet</td>
<td>medium</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>9A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>9</td>
<td>SP</td>
<td>SM</td>
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<tr>
<td></td>
<td>10</td>
<td>wet</td>
<td>medium</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td></td>
<td></td>
</tr>
<tr>
<td>35</td>
<td>10A</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17</td>
<td>16</td>
<td>SP</td>
<td>SM</td>
</tr>
<tr>
<td></td>
<td>17</td>
<td>wet</td>
<td>medium</td>
</tr>
<tr>
<td>40</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Description:**

- SAND; light gray-brown, fine to medium, few silt, occasional laminations of brown fine sandy silt
- 6" to 8" log at depth 23'
- Occasionally stained rust-brown
- Grades to trace silt

---

Redmond Connection Project

Project No.

86-35289-04

Converse Consultants
Geotechnical Engineering and Applied Sciences

A-41
<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample</th>
<th>Blows/6</th>
<th>Field Moisture</th>
<th>Dry Density</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>11A</td>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>45</td>
<td>12C</td>
<td>40</td>
<td></td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>13A</td>
<td>18</td>
<td></td>
<td></td>
</tr>
<tr>
<td>55</td>
<td>14A</td>
<td>27</td>
<td></td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>15A</td>
<td>10</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**SUMMARY: BORING NO. B-87**

*Elevation: 50 ft*

---

**DESCRIPTION**

- **SAND (Continued)**
  - Large log at depth 42.5'
  - Sandy gravel; brown mottled orange-brown, fine to coarse, medium to coarse sand, few silt, little silt in indistinct beds
  - Grades to gray-brown mottled orange-brown, little silt, medium plasticity

- **SILT**; gray, little fine sand, occasional indistinct beds of gravelly silt with sand

---

**MOISTURE**

- Wet
- Moist

**CONSISTENCY**

- Medium dense
- Very dense

---

**NOTES**

- Total depth approximately 60.0'
- 3/4" diameter PVC piezometer installed to 58.5' with 2' screen interval at bottom; backfilled with pea gravel; bentonite seal placed at surface.

---

**PROJECT NO.** 86-35289-04

---

**Converse Consultants**

Geotechnical Engineering and Applied Sciences
### SUMMARY: BORING NO. B-89

**ELEVATION:** Approx. 133'

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

<table>
<thead>
<tr>
<th>Depth in Feet</th>
<th>Sample No.</th>
<th>Borehole</th>
<th>Other Tests</th>
<th>Field Moisture</th>
<th>Consistency</th>
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<tbody>
<tr>
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</tr>
<tr>
<td>1A</td>
<td>4</td>
<td>6</td>
<td>18</td>
<td>SP - SM</td>
<td>very loose</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>moist</td>
<td></td>
</tr>
<tr>
<td>1A</td>
<td>6</td>
<td>6</td>
<td>18</td>
<td>SP - SM</td>
<td>medium size</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>dense</td>
<td></td>
</tr>
<tr>
<td>2C</td>
<td>10</td>
<td>12</td>
<td>9</td>
<td>SP</td>
<td>wet</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>moist</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>very moist</td>
<td></td>
</tr>
<tr>
<td>3A</td>
<td>6</td>
<td>6</td>
<td>18</td>
<td>SP</td>
<td>wet</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>very moist</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>moist</td>
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<td>very moist</td>
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<td>moisture</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>increasing silt</td>
<td></td>
</tr>
</tbody>
</table>

**Description:**
- SAND (Fill); rust-brown, fine to medium, few to little silt
- SAND; light brown, fine to medium, trace silt
- Increasing silt
- SILT; light brown, occasional rust stained, little fine sand, few coarse sand and fine gravel, low plasticity

---

**Footnotes:**
- A. 2” split-spoon sampler
- B. 3” O.D. thin-wall sampler
- C. 3-1/4” O.D. x 2-1/2” liner
- D. 3-1/2” O.D. split barrel sampler
- X. sample not recovered
- G. grain size
- T. triaxial
- P. permeability
- A. Atterberg
- DS. direct shear

---

**Project No:** 86-35289-04

**Drawing No:** A-43
<table>
<thead>
<tr>
<th>Depth in Feet</th>
<th>Sample No.</th>
<th>Blows/6 in</th>
<th>Field Moisture</th>
<th>Dry Density</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>5A</td>
<td>5</td>
<td></td>
<td></td>
<td>SILT (Cont.); occasional indistinct beds of silty gravel with sand ML</td>
</tr>
<tr>
<td>25</td>
<td>6A</td>
<td>11</td>
<td></td>
<td></td>
<td>SILT; dark gray-brown, little fine sand, occasional medium beds of silty fine sand, occasional thin laminations of peat and decaying wood ML</td>
</tr>
<tr>
<td>30</td>
<td>7C</td>
<td>22</td>
<td></td>
<td></td>
<td>SANDY GRAVEL; orange-brown, fine to coarse, medium to coarse sand, few silt, occasional beds with little silt G</td>
</tr>
</tbody>
</table>

Total depth approximately 31.5 feet. 3/4" diameter PVC piezometer installed to 30' with 2' screen interval at bottom; backfilled with pea gravel; bentonite seal placed at surface.

---

* A. 2" split-spoon sampler
B. 3" O.D. thin-wall sampler
C. 3-1/4" O.D. x 2-1/2" liner
D. 3-1/2" O.D. split barrel sampler
X. sample not recovered
**A - Atterberg, C - consolidation, DS - direct shear, G - grain size, T - triaxial, P - permeability

REDMOND CONNECTION PROJECT
for HNTB

Converse Consultants
Geotechnical Engineering and Applied Sciences
**DATE DRILLED:** 12/1/87  
**SUMMARY:** BORING NO. B-93 570

**ELEVATION:** Approx. 242'

<table>
<thead>
<tr>
<th>DEPTH</th>
<th>SAMPLE NO.</th>
<th>BLOWS</th>
<th>OTHER TESTS</th>
<th>FIELD MOISTURE</th>
<th>DRY DENSITY</th>
<th>SYMBOL</th>
<th>MOISTURE</th>
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<td></td>
</tr>
<tr>
<td>1</td>
<td>1A</td>
<td>3</td>
<td>3</td>
<td>4</td>
<td>23</td>
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<td>5</td>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
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<td>7</td>
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<td>13</td>
<td>20</td>
<td>24</td>
<td>28 100</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>4A</td>
<td>7</td>
<td>13</td>
<td>19</td>
<td>30</td>
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</tr>
<tr>
<td>25</td>
<td>5A</td>
<td>7</td>
<td>11</td>
<td>18</td>
<td>27</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**DESCRIPTION**

**SAND (Fill):** brown, fine to medium, few silt, probably utility backfill

**SILT:** light brown with dark rust-brown clasts, little fine sand, low plasticity, occasional beds of sandy gravel

**SILTY SAND W/GRAVEL:** light brown, occasional rust staining, fine sand, fine to coarse gravel, low plasticity, occasional beds of sandy gravel

**TOTAL DEPTH:** approximately 20.0'

3/4" diameter PVC piezometer installed to 19.5' with 2' screen interval at bottom; backfilled with pea gravel; bentonite seal placed at surface.

**Project No.:** 86-35289-04

**Converse Consultants**

*Geotechnical Engineering and Applied Sciences*
APPENDIX D
REPORT LIMITATIONS AND GUIDELINES FOR USE

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

Read These Provisions Closely

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

Geotechnical Services are Performed for Specific Purposes, Persons and Projects

This report has been prepared for the City of Kirkland, COWI North America, Inc., and other members of the design team for the project specifically identified in the report. The information contained herein is not applicable to other sites or projects.

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

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

This report has been prepared for the Totem Lake Connector project in Kirkland, Washington. GeoEngineers considered several 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.

---

Developed based on material provided by GBA, GeoProfessional Business Association; www.geoprofessional.org.
For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structures;
- elevation, configuration, location, orientation or weight of the proposed structures;
- composition of the design team; or
- project ownership.

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

Environmental Concerns Are Not Covered

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

Subsurface Conditions Can Change

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

Geotechnical and Geologic Findings Are Professional Opinions

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

Geotechnical Engineering Report Recommendations Are Not Final

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

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

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

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

Do Not Redraw the Exploration Logs

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

Give Contractors a Complete Report and Guidance

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

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

Contractors Are Responsible for Site Safety on Their Own Construction Projects

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

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

If Client desires these specialized services, they should be obtained from a consultant who offers such services.