FINAL GEOTECHNICAL REPORT
McDonald Creek Bridge Replacement Project
Clallam County, Washington

PROJECT NO. 15-130.911
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Prepared for:

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PanGEO

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APPENDIX A: FIELD EXPLORATIONS AND LOGS OF TEST BORINGS

APPENDIX B: LABORATORY TEST RESULTS
PROJECT DESCRIPTION

The Clallam County Public Works Department plans to replace the existing bridge crossing of McDonald Creek on the Old Olympic Highway in the eastern portion of Clallam County, Washington. The existing three-span structure is proposed to be replaced with a single span configuration approximately 156 feet in length from back of pavement seat to back of pavement seat. The end slopes extending down to McDonald Creek would be flattened and the floodplain broadened for improved hydraulics. The channelization would also be widened with a new structure out-to-out width of approximately 44 feet.

The project site is located in the south ½ of Section 8, Township 30N, Range 4W. The Old Olympic Highway alignment runs east-west through the middle of the SE¼ and SW¼ of Section 8, with the existing bridge site located on the quarter-section line between the SE¼ and SW¼, as shown on Figure 1, Vicinity Map.

In addition to the bridge replacement, new retaining walls will be required to contain the widened channelization in the approaches to the new bridge abutments. These walls are planned as pre-approved, proprietary structural earth walls (SEW). Stormwater management facilities may also be sited to the west of the bridge location. These project elements are addressed in this report.

SITE DESCRIPTION

This site description is based in part on a visual site reconnaissance conducted on June 11, 2015. The original wooden trestle bridge (circa 1938-39) constructed at this location was part of a larger effort to re-channelize McDonald Creek. A similar re-channelization was constructed when McDonald Creek was bridged upstream at the SR 101 alignment (also circa 1938-39). The extent to which additional re-channelization occurred between the two locations, or downstream of the Old Olympic Highway, is uncertain. For the most part, the channel of McDonald Creek as it appears today bears little resemblance to the original natural channel which was a deeply incised, heavily meandering feature as opposed to the relatively linear, north-south flow channel that now exists.

According to drawings dated 1956, the timber trestle was replaced with the current existing structure which is a 121½-foot long, cast-in-place, voided slab reinforced concrete superstructure with an asphalt overlay wearing surface. The superstructure is supported on 13-inch driven, cast-in-place concrete piles in a pile-bent pier configuration.
The project site is located on the western fringe of the broad Dungeness River delta landform. Topographic relief in the project area is dominated by the re-channelized incision of McDonald Creek, with surrounding uplands flanking the channel at approximately El. +175 feet and sloping gently northward toward the Strait of Juan de Fuca. The incision of McDonald Creek is about 45 feet deep with steep sidewalls. The east approach to the existing bridge is a relatively long and narrow “causeway” fill that replaced about 250 of the eastern end of the original timber trestle structure. This embankment forces the flow of McDonald Creek to veer through a near-right angle turn to the west and parallel the upstream toe of the embankment before turning south again at near-right angle to flow through the opening provided by the existing bridge. The toe of the embankment is therefore vulnerable to erosion attack by McDonald Creek and has been well-armored with rip rap material.

Land use on properties adjacent to the Old Olympic Highway in the local vicinity of the project limits is a combination of agriculture and rural residential.

**GEOLOGY**

The project site is located on the western flank of the broad alluvial fan and delta of the Dungeness River, and more locally of McDonald Creek itself. According to the Geologic Map of the Carlsborg 7.5-Minute Quadrangle (Schasse & Wegmann, 2000), the fan consists of fine to medium sand and interbedded silt and clayey silt with lenses of sand and gravel. This alluvium is underlain by Vashon glacial drift and older undifferentiated glacial and non-glacial deposits in locations such as McDonald Creek, where the creek has incised itself through the fan. During the site reconnaissance on June 11, 2015, near vertical exposures of both the alluvial fan deposits and the underlying glacially overridden soils were observed in the channel of McDonald Creek just upstream of the bridge crossing location.

**FIELD EXPLORATIONS**

The subsurface exploration program was completed in two phases of field work. The first phase consisted of drilling three test borings along the project alignment, two located near the proposed abutment locations for the new bridge and one additional boring located to the east in the earth embankment “causeway” approach to the existing structure as shown in Figure 2, Site and Exploration Plan. The first phase of the field exploration took place on March 7 and 8, 2016. The borings were drilled by Holocene Drilling of Edgewood, Washington, under subcontract to PanGEO, Inc., and were drilled using hollow stem auger drilling technology. The borings were designated from west to east as TH-1 through TH-3 (see Figure 2), and were advanced to depths of 86.5, 100.8 and 71.3 feet, respectively. Standard Penetration Test (SPT) samples were taken
at 5-foot intervals. All borings were backfilled and closed in accordance with requirements of the Washington State Department of Ecology at the end of drilling each test boring.

A second phase of field explorations was completed on October 12, 2016 and consisted of advancing four hand auger borings, designated HB-1 through HB-4 to depths ranging from 2.5 to 5 feet at the location of proposed compost amended vegetative filter strips (CAVFS). These four locations were in the existing roadside swales to the west of the McDonald Creek crossing, as shown on Figure 2, Site and Exploration Plan.

An engineer or engineering geologist from PanGEO was on site to coordinate drilling activities and log the test borings. The locations of borings TH-1 through TH-3 as shown on Figure 2 of this report are based on survey provided by Clallam County. The locations of hand borings HB-1 through HB-4 are based on the relative location of the exploration points referenced from known site features with elevations obtained from contours from the base map for the project. The locations and elevations provided on the logs of the hand borings should therefore be considered approximate. Appendix A contains summary logs of boreholes and describes the field exploration methodology in greater detail.

**SUBSURFACE CONDITIONS**

The three machine test borings were drilled from the existing road grade of the Old Olympic Highway. Pavement surfacing at all three boring locations consisted of 8 inches of asphaltic concrete (AC). No discernable base coarse aggregate was apparent beneath the AC surfacing.

**SOILS**

The site soils, as encountered in the test borings at the bridge crossing, consisted of a relatively consistent sequence of fill overlying native alluvial soils in turn overlying glacially overridden deposits, while the four hand borings encountered only alluvial fan deposits as delineated on the geologic map of the area (Schasse & Wegmann, 2000). A more detailed description of the soil units encountered is as follows:

**Fill** - All three test borings drilled for the bridge and approach alignment encountered fill below the structural pavement section consisting of silty sand with gravel and clayey silt. As encountered in the test borings, the clayey silt becomes the dominant component of the fill toward the east in the approach “causeway”. The fill materials were loose to medium dense or medium stiff, indicating that only minimal compactive effort was used during placement of the fill.

**Alluvium** - Directly underlying the fill soils, the three machine test borings encountered alluvial soils consisting of interbeds of fine sand, fine to coarse sand, clayey silt and sandy gravel. The
alluvium was loose to dense or stiff. These materials are interpreted as natural stream deposits by McDonald Creek.

**Alluvial Fan Deposits** - These soils were only encountered in the hand borings drilled west of the McDonald Creek crossing and consisted generally of sandy silt overlying fine to medium silty sand with trace amounts of gravel, except at the location of HB-4, which met with practical refusal at a depth of 2.5 feet in abundant gravels.

**Glacial Drift** - Beneath the alluvium and alluvial fan deposits lies undifferentiated glacial deposits. These were encountered in all three borings drilled along the bridge and approach fill alignment to the maximum depths explored for this study. These soils are mostly fine to coarse silty sands with interbeds of silt and gravel in a dense to very dense condition due to glacial overriding.

**GROUNDWATER**

Groundwater was observed in test borings TH-1 and TH-3 at the top of the glacial drift layer. In test boring TH-2, which is more proximal to McDonald Creek the water level was observed near the top of the alluvium layer at an elevation similar to the flow in the creek. The lower groundwater levels observed in the borings further from the creek appear to indicate that McDonald Creek is a losing stream through this reach and the regional groundwater level may be lower than the ordinary flow level in McDonald Creek except at locations near the active stream channel where groundwater levels are likely to be at or near the flow level. Groundwater was not encountered in the hand borings drilled west of the McDonald Creek crossing.

**LABORATORY TESTING**

Laboratory testing of soil materials included determination of fines content, moisture content and grain size distribution on select samples recovered during the drilling and sampling program. Testing was in accordance with appropriate AASHTO and/or ASTM standards. The test results and a discussion of laboratory test methodology are presented in Appendix B. Where appropriate, test results are displayed on the summary boring logs in Appendix A.
SEISMIC CONSIDERATIONS

SITE SEISMICITY
The Strait of Juan de Fuca is a tectonically active area. As a result of northward movement of the Washington block of the North American Plate against Vancouver Island (Mosher and Johnson, 2002), Eocene sedimentary and volcanic rocks have uplifted on the north flank of the Olympic Mountains through a series of east-west trending thrust faults. The trace of one of these faults is mapped by Schasse (2003) to the southwest of the project site, and is not currently thought to be active in the Holocene. At least two of these faults, the Little River Fault, along the south side of the Port Angeles drift plain area and the River Road scarp near Sequim, have identified fault scarps that suggest Holocene movement. In addition, the eastern Strait of Juan de Fuca is an area of crustal boundary between pre-Tertiary Cascade province rocks to the east and the Coast Range Eocene rocks on the Olympics (Mosher and Johnson, 2002). The eastern Strait is underlain by several shallow, active fault structures, including the South Whidbey Fault Zone, the Devils Mountain Fault and the Leech River Fault (on Vancouver Island). Based on the current understanding of fault location and activity, the hazard potential for fault rupture at the McDonald Creek Bridge site is thought to be low.

SEISMIC DESIGN PARAMETERS
For seismic design, an acceleration coefficient of 0.375g is recommended per the current acceleration map in AASHTO (2014). The recommended acceleration coefficient is based on expected ground motion at the project site that has a 7 percent probability of exceedance in a 75-year period (approximately 1000-year return period).

Design response spectra presented in AASHTO (2014) are considered appropriate for seismic design of the bridge. A horizontal response spectral acceleration coefficient at a period of 0.2 seconds (S2) is 0.83 and the horizontal response spectral acceleration coefficient at a period of 1.0 seconds (S1) is 0.32.

Based on the test borings drilled at the site, the soils at the site are Site Class D, with associated site factors Fpga, Fa and Fv of 1.13, 1.17 and 1.77, respectively. From which values for As, SDS and SD1 of 0.44, 0.97 and 0.56, respectively, are obtained. The site is therefore in Seismic Performance Zone 4.

LIQUEFACTION POTENTIAL
Liquefaction of saturated sands occurs when the sands are subject to cyclic loading. The cyclic loading causes the water pressure to increase in the sand, reducing the inter-granular stresses. As the inter-granular stresses are reduced, the shearing resistance of the sand decreases. If pore
pressures develop to the point where the effective stresses acting between the grains become zero, the soil will behave like a viscous fluid. Under this condition soil flow is possible. The effect of liquefaction can range from reduced shear strength to viscous fluid behavior. The liquefaction potential of saturated sands is evaluated mainly on soil gradation, relative density, and the depth of the deposit. The potential for liquefaction is highest for saturated loose, fine to medium grained sands and silty sands above a depth of about 70 feet.

The liquefaction potential of the soils at the bridge site was evaluated using the procedure originally developed by Seed and modified in the 1996 and 1998 NCEER/NSF workshops (Youd et al., 2001). The liquefaction analyses were conducted using a Magnitude 7.5 event with peak ground acceleration of 0.375g, which are consistent with the AASHTO (2014) design criteria. Settlement estimates were made using the procedures of Tokimatsu and Seed (1987) or Ishihara and Y oshimine (1992) as recommended in the Geotechnical Design Manual (2015).

Our analysis indicated there is only potential for liquefaction during design earthquake at the location of the boring drilled for in the approach “causeway” from a depth of approximately 34 to 43 feet in the alluvium below the embankment fill. Neither of the borings drilled near the proposed abutments of the new bridge indicated potential for liquefaction to develop. No special design considerations for the effects of liquefaction are therefore considered necessary for the bridge itself. A discussion of risks and effects of potential liquefaction below the existing “causeway” embankment is provided below.

CONCLUSIONS AND RECOMMENDATIONS

STORMWATER MANAGEMENT CONSIDERATIONS

An assessment of stormwater infiltration potential was made for the proposed locations of compost amended vegetative filter strips (CAVFS) on the west side of the McDonald Creek crossing. This assessment was based on soil gradation data and visual-manual soil descriptions in the depth range from the ground surface to 5 feet below the existing ground surface at the locations of hand borings HB-1 through HB-4 (Figure 2, Site and Exploration Plan).

Using the procedure provided in the Stormwater Management Manual for Western Washington (WSDOE, 2012), long term design infiltration rates were estimated using soil gradation data from samples collected from hand borings HB-1 through HB-4. Based on the information from the hand borings, the upper materials, from the bottom of the existing swale to about 3.5 to 4.5 feet in depth are relatively impermeable with estimated long term (design) infiltration rates of only 0.1 to 0.2 inches per hour. Below these depths more permeable soils were encountered,
with rates ranging from 0.5 to 2 inches per hour. If the upper, less permeable soils are removed and replaced with a more permeable material such as Backfill for Sand Drains or Sand Drainage Blanket (WSDOT Standard Specifications, 2016) an average design infiltration rate of 1.0 inch per hour may be used for infiltration analysis of the proposed CAVFS. The above long-term, design infiltration rates include correction factors of $C_F = 0.33$, $C_T = 0.4$ and $C_M = 0.9$ in accordance with the design guidance provided in the above referenced manual.

Based on the groundwater level measurements observed in the test borings drilled for this project, depth to groundwater will not be a design consideration at the proposed locations of the CAVFS.

**BRIDGE APPROACHES**

Grades at the approaches to the new overcrossing are expected to be raised approximately 3 feet above existing grades to improve the vertical curve and sight distance through the project area. In order to accommodate the widened replacement structure, the approach on the east side of the new bridge will need retaining walls on both sides of the “causeway” area of the alignment using back-to-back structural earth walls (SEWs).

As discussed above under Liquefaction Potential, the native alluvium soils beneath the existing “causeway” embankment on the east side of the existing bridge are expected to liquefy during or shortly after a design-level earthquake. However, pseudo-static global stability analyses indicate that the liquefiable layer is too deep to adversely affect the stability of the approach fill retaining walls. Liquefaction effects are therefore expected to be limited to post-liquefaction settlement in the range of 2 to 4 inches. This amount of settlement is not expected to affect the serviceability of the structural earth walls or approach embankment.

**STRUCTURAL EARTH WALLS**

As stated above, the estimated total and differential post-liquefaction settlement is within tolerable limits for precast panel-faced structural earth wall systems that have minimum joint widths of at least ¾-inch. Static settlement is expected to be one inch or less. All static settlement that may occur is expected to occur rapidly, as the walls and backfill are constructed.

The overall stability of the approach fill retaining walls was analyzed using limit equilibrium methods (Spencer’s method) and the computer program SLIDE v. 6.0, developed by Roc Science. Both circular and non-circular failure surfaces were included in the analyses. The critical wall sections for the stability analyses were established based on wall height, subsurface soil and groundwater conditions, and the proposed surface grades in front of the wall.
strength parameters were assigned based on soil and groundwater conditions as encountered in the test borings. The analyses incorporated the design recommendations presented below. The seismic stability was analyzed using pseudo-static procedures, where the effect of earthquake ground shaking is represented by the use of a “seismic coefficient” in the stability calculations. One-half of the design peak ground acceleration was used for the seismic coefficient in our pseudo-static stability analysis.

Based on our analyses, provided the minimum recommended reinforcement lengths and wall embedment depths recommended below are used for wall design, minimum static and pseudo-static seismic factors of safety for the critical wall sections were found to be 1.3 and 1.1 for the service limit state and the extreme event limit state, respectively.

In order to be consistent with the Standard Specifications and the general special provisions that accompany them, the walls should be designed using LRFD methods, and the general special provision (GSP) fill-ins presented below in Table 1 are in the currently recommended LRFD format. The following recommendations should be followed to provide external stability of the proposed structural earth walls. Structural earth walls should be constructed in accordance with Section 6-13 of the Standard Specifications (WSDOT, 2016), with the information provided in Table 1 included in the general special provisions.

1. The wall may be constructed near vertical, without a specified batter.

2. The wall should be placed on a level foundation in the horizontal direction perpendicular to the wall face.

3. The reinforcing length should not be less than 100 percent of the total (exposed height plus embedment depth) wall height. These recommended minimum reinforcing lengths are needed to maintain adequate external stability. Greater reinforcing lengths may be needed to provide adequate internal stability.

4. The minimum embedment of the walls should be 4 feet below adjacent finish grade.

5. The uppermost reinforcing layer should be placed no lower than 2 feet below the top of wall. Welded wire faced systems should include a top mat at the top of the wall.

6. Since the wall will be constructed above existing grades and in a back-to-back configuration with impervious coverage above, there is limited potential for water to reach or build up in the reinforced zone. Special drainage elements, such as heel drains are therefore not required.
Table 1
Design Parameters for Pre-approved, Proprietary Structural Earth Walls

<table>
<thead>
<tr>
<th>Soil Properties</th>
<th>Wall Backfill$^1$</th>
<th>Retained Soil$^2$</th>
<th>Foundation Soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight (pcf)</td>
<td>135</td>
<td>130</td>
<td>120</td>
</tr>
<tr>
<td>Friction Angle (deg)</td>
<td>38</td>
<td>32</td>
<td>32</td>
</tr>
<tr>
<td>Cohesion (psf)</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

For the Service Limit State, the wall shall be designed to accommodate a differential settlement of 1 inch per 100 feet of wall length.

For the Extreme Event I Limit State, the wall shall be designed for a horizontal seismic acceleration coefficient $K_h$ of 0.19g and a vertical seismic acceleration coefficient $k_v$ of 0.0g.

Notes:  
1 – Reinforced zone backfill should be Gravel Borrow or Gravel Backfill for Walls (WSDOT, 2016).  
2 – Retained soil behind the reinforced zone should be Select or Gravel Borrow (WSDOT, 2016).

LATERAL EARTH PRESSURES ON ABUTMENT WALL

If a joint is provided at the abutment so that the abutment wall is free to deflect slightly, active pressures can be used in design. An equivalent fluid pressure of 35 pounds per cubic foot (pcf) may be used to calculate lateral earth pressures on the abutments. This equivalent fluid pressure does not include live load surcharge. A lateral earth pressure coefficient, $K_a$, of 0.3 may be used to calculate the lateral load due to surcharge.

If abutment walls are fixed against lateral deflection, at-rest pressures will be appropriate for design. An equivalent, at-rest fluid pressure of 55 pcf may be used to calculate at-rest earth pressures on the abutments. This equivalent fluid pressure does not include live load surcharge. An at-rest lateral earth pressure coefficient, $K_o$, of 0.45 may be used to calculate the lateral load due to surcharge.

The seismic earth pressure is computed according to the Mononobe-Okabe method described in the LRFD Bridge Design Specifications (AASHTO, 2014). The walls are assumed free to move and to develop the active earth pressure conditions during a seismic event. For this project we recommend that the seismic earth pressure increment be taken as 15H psf, where H is the height of the soil behind the abutment. The seismic earth pressure increment is in addition to the active static earth pressure, and is in a trapezoidal distribution, applied at 0.6H from the bottom of the pressure distribution.
NEW BRIDGE FOUNDATIONS

Due to the relatively poor soil bearing resistance and settlement potential of the near surface soils at the site, shallow foundations are not recommended for support of the new bridge. Drilled shafts are preferred over driven pile groups as a deep foundation system for the new bridge considering the density of the glacially overridden soils present at elevations of approximately +115 to +120 feet which would result in very hard driving conditions for large displacement piles such as WSDOT cast-in-place concrete piles or prestressed concrete piles. Small displacement piles such as open-end steel or H-pile sections are considered feasible, but may require a larger pile cap relative to drilled shafts due to their lower individual moment capacity as an earthquake resisting system.

Shaft Axial Resistance

Figures 4 through 9 present axial resistances for 3-, 4- and 6-foot diameter drilled shaft options at Piers 1 and 2 (west and east abutments), respectively. Nominal (ultimate) resistances, factored resistances at the strength limit state and settlement-limited resistances at the service limit state are all plotted together on Figures 4 through 9. Service limit state resistance was computed for tolerable settlement of ½-inch under service loading conditions. Extreme event axial loads may be evaluated using the nominal (ultimate) resistance and a resistance factor of 1.0.

Please note that the resistances plotted on Figures 4 through 9 are net at the top of shaft and that the self-weight of the shaft has not been subtracted from these resistances. The self-weight of the shaft should be considered as part of the dead load of components (DC) and included on the load side of AASHTO (2014) equation 1.3.2.1-1.

Lateral Foundation Resistance & Group Reduction Factors

If shaft foundations are used in groups, the group reduction factors for lateral analysis presented in Table 2 should be used. The factors account for shaft interaction effects due to proximity and are a function of shaft center-to-center spacing based on shaft diameter, D, and the direction of loading. The factors are outside the range provided in AASHTO (2014) and were based on the research by Brown, et al. (1987, 1988) and McVay, et al. (1995). Group effects for axial loads will not be significant so long as shafts are spaced at least 2.5D per Table 10.8.3.6.3-1 for “shaft group cap in intimate contact with ground” or 3D for any other condition.
### Table 2
**Group Reduction Factors for Lateral Analysis**

<table>
<thead>
<tr>
<th>Shaft Spacing(^{(1)})</th>
<th>Reduction Factor for Load Applied Parallel to Pier Cap</th>
<th>Reduction Factor for Load Applied Perpendicular to Pier Cap(^{(2)})</th>
</tr>
</thead>
<tbody>
<tr>
<td>5D</td>
<td>0.8</td>
<td>1.0</td>
</tr>
<tr>
<td>4D</td>
<td>0.65</td>
<td>0.9</td>
</tr>
<tr>
<td>3D</td>
<td>0.5</td>
<td>0.8</td>
</tr>
<tr>
<td>2.5D</td>
<td>0.4</td>
<td>0.7</td>
</tr>
</tbody>
</table>

\(^{(1)}\) As a function of shaft diameter, D.

\(^{(2)}\) For a single row of shafts; if two or more rows of shafts are used, the reduction factors for load applied parallel to the pier cap should be used.

Recommended parameters for analysis of lateral shaft resistance using the program LPILE™ or COM 624 are presented in Tables 3 and 4, below. Note that the soil layers are referenced to an estimated top of shaft elevation at the location of the reference borings. If the top of shaft elevation is different, the thickness of Layer 1 should be adjusted accordingly.

### Table 3 Recommended p-y Curve Parameters: West Abutment (Pier 1)

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Bottom of Layer Elevation</th>
<th>Soil Type</th>
<th>Soil Type (K SOIL)</th>
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<tbody>
<tr>
<td></td>
<td>(ft)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>+130</td>
<td>Sand</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>+120</td>
<td>Sand</td>
<td>4</td>
</tr>
<tr>
<td>3</td>
<td>+70</td>
<td>Sand</td>
<td>4</td>
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**STATIC ANALYSIS**

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Bottom of Layer Elevation</th>
<th>Soil Type</th>
<th>Soil Type (K SOIL)</th>
<th>Effective Unit Weight of Soil(^{(2)})</th>
<th>Cohesion</th>
<th>Axial Strain (\varepsilon_{50})</th>
<th>Friction Angle (\phi)</th>
<th>Modulus of Subgrade Reaction(^{(2)})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(ft)</td>
<td></td>
<td></td>
<td>(pci)</td>
<td>(pcf)</td>
<td>(psi)</td>
<td>(deg)</td>
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</tr>
<tr>
<td>1</td>
<td>+130</td>
<td>Sand</td>
<td>4</td>
<td>0.067</td>
<td>115</td>
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<td>3</td>
<td>+70</td>
<td>Sand</td>
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<td>0.042</td>
<td>73</td>
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<td>41</td>
</tr>
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</table>

Notes: 1 – Adjust as necessary for top of shaft elevation, as appropriate.

### Table 4 Recommended p-y Curve Parameters: East Abutment (Pier 2)

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Bottom of Layer Elevation</th>
<th>Soil Type</th>
<th>Soil Type (K SOIL)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(ft)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>+132</td>
<td>Sand</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>+116</td>
<td>Sand</td>
<td>4</td>
</tr>
<tr>
<td>3</td>
<td>+60</td>
<td>Sand</td>
<td>4</td>
</tr>
</tbody>
</table>

**STATIC ANALYSIS**

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Bottom of Layer Elevation</th>
<th>Soil Type</th>
<th>Soil Type (K SOIL)</th>
<th>Effective Unit Weight of Soil(^{(2)})</th>
<th>Cohesion</th>
<th>Axial Strain (\varepsilon_{50})</th>
<th>Friction Angle (\phi)</th>
<th>Modulus of Subgrade Reaction(^{(2)})</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(ft)</td>
<td></td>
<td></td>
<td>(pci)</td>
<td>(pcf)</td>
<td>(psi)</td>
<td>(deg)</td>
<td>(pci)</td>
</tr>
<tr>
<td>1</td>
<td>+132</td>
<td>Sand</td>
<td>4</td>
<td>0.067</td>
<td>115</td>
<td>--</td>
<td>--</td>
<td>31</td>
</tr>
<tr>
<td>2</td>
<td>+116</td>
<td>Sand</td>
<td>4</td>
<td>0.036</td>
<td>63</td>
<td>--</td>
<td>--</td>
<td>36</td>
</tr>
<tr>
<td>3</td>
<td>+60</td>
<td>Sand</td>
<td>4</td>
<td>0.042</td>
<td>73</td>
<td>--</td>
<td>--</td>
<td>41</td>
</tr>
</tbody>
</table>

Notes: 1 – Adjust as necessary for top of shaft elevation, as appropriate.
CONSTRUCTION CONSIDERATIONS
The following items should be considered during the final design of the project and development of the contract specifications.

1. Temporary shoring and/or open pits may be required during construction of the new foundations. The design and construction of temporary shoring/slopes should be the responsibility of the contractor. Due to the elevation of the abutments above the flow level of McDonald Creek, groundwater inflow is unlikely in foundation excavations except to the extent that inclement weather or construction during the wet season results in perched water inflow into the excavation. Such groundwater inflow should be controllable with best practices for controlling sheet flow and runoff, and/or sumps and pumps.

2. Shaft construction will likely extend below the groundwater table as excavation proceeds to depth. Positive control of groundwater pressures using counter-balancing slurries (including water) will be required to maintain stability of shaft excavations. Sidewall caving in both the existing fill materials and underlying alluvial deposits is expected, therefore temporary casing is recommended within these soils overlying the glacial drift bearing layer. Below the surface casing, excavation may be possible using open-hole methods provided drilling slurry is used to control groundwater pressures. Open-hole construction has been successfully employed on drilled shafts in similar materials on other recent bridge projects on the north Olympic Peninsula. Advancement of temporary casing by vibratory hammer or top-drive methods may be possible through the near surface fill and alluvium, but may meet with refusal above design shaft tip elevations, particularly in the dense to very dense glacial drift deposits. Relieving the excavation by removal of materials inside the casing may be needed to advance temporary casings. Relieving of material should not extend beyond the concurrent tip elevation of temporary casing at the time of removal of soils from the shaft excavation.

3. Obstructions, including boulders, are known to exist in the glacial drift soil bearing layer. Although not encountered in the test borings drilled for this project, the potential also exists for remnants of the original timber trestle crossing at this site to be present in the approach fill “causeway”. A nominal force account quantity for obstruction removal is therefore recommended for use in conjunction with the Shafts specification (Article 6-19) of the Standard Specifications for Road, Bridge and Municipal Construction (WSDOT, 2016).
4. The shaft axial side resistances presented in Figures 4 through 9 assume concrete cast against soil. As such, temporary casings should be removed during / following concrete placement and should not remain as part of permanent shaft construction.

5. Excavation for the placement of structural earth wall (SEW) reinforcement behind the east abutment of the new bridge is expected to remove a significant amount of the existing “causeway” fill. Although not encountered in the test borings drilled for this project, the potential also exists for remnants of the original timber trestle crossing at this site to be present in these excavations. A contingency for handling and disposal of such materials may be prudent in the construction contract. The existing “causeway” fill is also largely comprised of fine-grained silt and clayey silt. This material is not suitable for use as backfill within the reinforced zone of structural earth walls.

6. If the excavate-and-replace option is selected for the proposed locations of the CAVFS, the sand backfill material should be lightly compacted in accordance Method A requirements (WSDOT, 2016).

**ADDITIONAL SERVICES**

PanGEO should review the final project plans and specifications to confirm that our geotechnical recommendations were properly incorporated into the contract documents. Construction support services, including review of structural earth wall and drilled shaft submittals and field observation of construction, are beyond the scope of geotechnical design services under which this report was prepared. A supplemental scope and budget would be required for PanGEO to provide construction support services and is recommended in order to confirm that construction is consistent with the design and construction recommendations provided herein.
LIMITATIONS AND UNIFORMITY OF CONDITIONS

PanGEO, Inc. (PanGEO) prepared this report for BergerABAM and the Clallam County Public Works for the Old Olympic Highway bridge replacement overcrossing McDonald Creek. The recommendations contained in this report are based on a site reconnaissance, a subsurface exploration program, review of pertinent subsurface information, and our understanding of the project.

Variations in soil conditions may exist between the locations of the explorations and the actual conditions underlying the site. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at the site that are different from those described in this report, PanGEO should be immediately notified to review the applicability of the recommendations presented herein. Additionally, PanGEO should also be notified to review the applicability of these recommendations if there are any changes in the project scope.

This report has been prepared for planning and design purposes for specific application to the proposed bridge replacement project in accordance with the generally accepted standards of local practice at the time this report was written. This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or other factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 36 months from its issuance. PanGEO should be notified if the project is delayed by more than 36 months from the date of this report so that the applicability of the conclusions and recommendations presented herein may be evaluated considering the time lapse.

Within the limitations of scope, schedule and budget, PanGEO engages in the practice of geotechnical engineering and endeavors to perform its services in accordance with generally accepted professional principles and practices at the time this report and/or its contents was prepared. No warranty, express or implied, is made. The scope of PanGEO’s work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water or groundwater at this site. PanGEO does not practice or consult in the field of safety engineering. PanGEO does not direct the contractor’s operations, and cannot be held responsible for the safety of personnel other than our own on the site; the safety of others is the responsibility of the contractor.

It is the client’s responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes shall be at the contractor’s sole option and risk. Any party other than the client who wishes to use this report shall notify PanGEO of
such intended use and for permission to copy this report. Based on the intended use of the report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use this report.

CLOSURE

PanGEO is pleased to support the BergerABAM design team and Clallam County with geotechnical engineering recommendations. If you have any questions regarding this report, please call (206) 262-0370.

Robert E. Kimmerling, P.E.
Principal Geotechnical Engineer
REFERENCES


WSDOT, 2016. Standard Specifications for Road, Bridge and Municipal Construction, M 41-10, Washington State Department of Transportation

FIGURES
**LEGEND**

- **TH-1** Soil Boring by PanGEO, Inc. (2016 - Appendix A)
- **HB-1** Hand Boring by PanGEO, Inc. (2016 - Appendix A)

**NOTES**

1. Basemap provided by BergerABAM on October 31, 2016.
2. Map features are provided for relative information only and are not a substitution for field survey.
3. Location of hand borings are approximate and based on the relative locations of known site features.

McDonald Creek Bridge Replacement Project
Clallam County, WA

**SITE AND EXPLORATION PLAN**

`PROJECT NO. 15-130.911`  `FRAME NO. 2`
Notes:
1) Axial resistance values are for a 3-foot diameter un-cased shaft.
2) Axial resistance values are gross values at the top of the shaft (i.e. the self-weight of the shaft has not been subtracted from the resistance values shown in these plots).
3) Factored strength limit state resistance includes $\phi_s = 0.55$ & $\phi_p = 0.50$ (Sand); $\phi_s = 0.60$ & $\phi_p = 0.55$ (IGM).
4) Service limit state resistance was developed to limit settlement to less than 1/2 inch.
Notes:
1) Axial resistance values are for a 3-foot diameter un-cased shaft.
2) Axial resistance values are gross values at the top of the shaft (i.e. the self-weight of the shaft has not been subtracted from the resistance values shown in these plots).
3) Factored strength limit state resistance includes $\varphi_s = 0.55$ & $\varphi_p = 0.50$ (Sand).
4) Service limit state resistance was developed to limit settlement to less than 1/2 inch.
Notes:
1) Axial resistance values are for a 4-foot diameter un-cased shaft.
2) Axial resistance values are gross values at the top of the shaft (i.e. the self-weight of the shaft has not been subtracted from the resistance values shown in these plots).
3) Factored strength limit state resistance includes $\phi_s = 0.55$ & $\phi_p = 0.50$ (Sand); $\phi_s = 0.60$ & $\phi_p = 0.55$ (IGM).
4) Service limit state resistance was developed to limit settlement to less than 1/2 inch.
Notes:
1) Axial resistance values are for a 4-foot diameter un-cased shaft.
2) Axial resistance values are gross values at the top of the shaft (i.e. the self-weight of the shaft has not been subtracted from the resistance values shown in these plots).
3) Factored strength limit state resistance includes $q_s = 0.55$ & $q_p = 0.50$ (Sand).
4) Service limit state resistance was developed to limit settlement to less than 1/2 inch.
Notes:
1) Axial resistance values are for a 6-foot diameter un-cased shaft.
2) Axial resistance values are gross values at the top of the shaft (i.e. the self-weight of the shaft has not been subtracted from the resistance values shown in these plots).
3) Factored strength limit state resistance includes $s = 0.55$ & $p = 0.50$ (Sand); $s = 0.60$ & $p = 0.55$ (IGM).
4) Service limit state resistance was developed to limit settlement to less than 1/2 inch.

McDonald Creek Bridge Replacement Project
Clallam County, Washington

AXIAL COMPRESSIVE RESISTANCE
6-FT DIAMETER SHAFTS
WEST ABUTMENT (PIER 1)
Notes:
1) Axial resistance values are for a 6-foot diameter un-cased shaft.
2) Axial resistance values are gross values at the top of the shaft (i.e. the self-weight of the shaft has not been subtracted from the resistance values shown in these plots).
3) Factored strength limit state resistance includes $q_s = 0.55$ & $q_p = 0.50$ (Sand).
4) Service limit state resistance was developed to limit settlement to less than 1/2 inch.
APPENDIX A

FIELD EXPLORATIONS
AND
LOGS OF TEST BORINGS
APPENDIX A: FIELD EXPLORATIONS AND LOGS OF TEST BORINGS

Appendix A contains written and graphical borehole logs presenting the factual and interpretive results of our exploratory drilling program on the subject site. The descriptions of the materials encountered in the subsurface explorations are based on the samples extracted from the borings. The sample descriptions are augmented by observation of the drilling action and drill cuttings brought to the surface during field operations. The paragraphs below describe the field operations and sampling procedures used during the geotechnical field explorations.

FIELD EXPLORATIONS

The subsurface exploration program for the project was conducted in two phases. The first phase consisted of drilling three test borings along the proposed bridge and approach alignment from March 7 to 8, 2016. Two of the borings were drilled near the proposed location of the abutments for the proposed widened overcrossing of McDonald Creek. The third boring was drilled to the east of the existing bridge in the approach fill “causeway” to provide a basis for recommendations for the proposed new approach fill retaining walls on this side of the new structure. All borings drilled during this phase were completed using conventional hollow stem auger drilling technology.

The boring locations are shown on Figure 2, Site and Exploration Plan. A representative of PanGEO logged the test borings. Soil samples were collected from selected intervals in the borings. The borings were drilled using a Mobile B-61 drill rig supplied with hollow stem augers and an automatic trip safety hammer sampling system provided by Holocene Drilling of Puyallup, Washington. The locations of the borings were surveyed by Clallam County and were located on the base map provided by BergerABAM that serves as the basis for Figure 2, Site and Exploration Plan.

The second phase of field exploration was completed on October 12, 2016 and consisted of advancing four hand auger borings at the locations of proposed compost amended vegetative filter strips (CAVFS) stormwater treatment areas in the road side swale west of the McDonald Creek crossing. The locations of the hand borings were referenced to existing site features, are considered approximate, and are shown on Figure 2, Site and Exploration Plan.

SAMPLING METHODS

Standard penetration tests were taken at 5-foot intervals, starting at 5 feet below ground surface and continuing to the bottom of each boring. The number of blows to drive the sampler each 6 inches over an 18-inch interval was recorded and indicated on the boring log. The number of blows to drive the sampler the final 12 inches is termed the SPT resistance, or N-value, and is
used to evaluate the strength and consistency/relative density of the soil. The hammer used to perform SPT sampling was an automatic trip-release mechanism, which generally delivers a higher energy than a “standard” hammer equipped with a rope and cathead mechanism. The efficiency of the hammer mechanism is considered when evaluating the liquefaction potential of a soil. The SPT N-values reported on the borehole logs are field values, and are therefore not corrected for hammer efficiency, overburden stress or rod lengths.

Grab (bulk) samples were collected from the hand auger borings for laboratory testing of grain size analysis.

Soils samples were identified and described in general accordance with the guidelines shown on Figure A-1. Summary boring logs are included as Figures A-2 to A-8. The stratigraphic contacts shown on the summary logs and subsurface profile (Figure 3, main text) represent the approximate boundaries between soil types; actual stratigraphic contacts encountered at other locations in the field may differ from the contact elevations shown on the logs, and may be gradual rather than abrupt. The soil and groundwater conditions depicted are only for the specific date and locations reported, and therefore, are not necessarily representative of other locations and times.
MOISTURE CONTENT
2-inch OD Split Spoon, SPT(140-lb. hammer, 30" drop)
3.25-inch OD Spilt Spoon(300-lb hammer, 30" drop)
Non-standard penetration test (see boring log for details)
Thin wall (Shelby) tube
Grab
Rock core
Vane shear

Terms and Symbols for Boring and Test Pit Logs

Figure A-1
Asphaltic concrete 8 inches.

Medium dense to loose, brown, silty SAND (SM) with gravel: moist, slightly plastic fines, gravel sub-rounded and blocky, mixed and massive. (Fill).

Silty fine SAND with gravel, trace of coarse sand.

Silty, fine to coarse SAND with gravel, non-plastic fines.

Silty SAND with gravel, brown with rusty weathering, moderately weathered.

Gravelly, silty SAND, gravelly drilling.

Medium stiff to stiff, brown with rusty mottles, clayey SILT with sand: moist, slightly plastic, homogeneous, massive to laminated. (Fill/Floodplain Deposits?).

Medium dense, brown, silty, fine to medium SAND (SM): very moist, interbeds of clayey silt, non-plastic to slightly plastic, rapid dilatancy, fine bedded (1" to 3") and laminated, rusty pockets and mottles. (Alluvium).

Medium dense, brown gray with rusty mottles and seams, interbedded silty, fine SAND and clayey SILT: very moist to wet, non-plastic and low plastic, rapid dilatancy, fine bedded (1" to 2") and laminated. (Alluvium).

Remarks: STA 169+23, 5.5' Rt. Groundwater level estimated from soil moisture and water observed on sampling rods.

The stratification lines represent approximate boundaries. The transition may be gradual.
Very dense, brown gray, silty, fine to medium SAND: wet, homogeneous, beds grading finer / coarser, occasional 2 inch clayey silt interbed, generally non-plastic fines with rapid dilatancy, laminated. (Drift). (Continued)

Very dense, brown gray, silty, fine to medium SAND (SM): wet, non-plastic fines, homogeneous, laminated with occasional clayey silt interlaminae, one dark gray laminae, possibly organic, occasional gravel, finer / coarser beds. (Drift).

Fines non-plastic, rapid dilatancy, massive to indistinctly laminated.

Very dense, brown gray, interbedded SILT, silty fine SAND and fine to coarse SAND with silt: wet, no-plastic fines, laminated, trace weathering. (Drift).

Very dense, brown gray, fine to medium SAND: very moist, some non-plastic silt, rapid dilatancy, homogeneous, trace coarse sand, massive. (Drift).

Grading wet.

Very dense, brown gray, interbedded SILT and fine SAND with silt and fine to medium SAND: wet, non-plastic, rapid dilatancy, laminated and fine bedded (1" - 3" beds), rare sub-angular coarse sand grain. (Drift).

Very dense, brown gray, fine to medium SAND: wet some beds with non-plastic silt, homogeneous, massive to laminated. (Drift).

Remarks: STA 169+23, 5.5’ Rt. Groundwater level estimated from soil moisture and water observed on sampling rods.
Very dense, brown gray, fine to medium SAND: wet some beds with non-plastic silt, homogenous, massive to laminated.  (Drift).

Interbedded fine to medium and fine to coarse SAND, laminated, some silt laminae, gradational contacts.

Dense, green gray SILT: wet, non-plastic, rapid dilatancy, homogeneous, laminated.  (Drift).

Bottom of Boring.

Remarks: STA 169+23, 5.5' Rt.  Groundwater level estimated from soil moisture and water observed on sampling rods.
### MATERIAL DESCRIPTION

**Asphaltic concrete, 8 inches.**
- Medium dense to loose, brown, silty SAND with gravel: moist, non-plastic fines, gravel sub-rounded and blocky, massive, mixed. (Fill).

**Medium stiff, brown, clayey SILT with gravel: moist to very moist, slightly to low plastic, trace to some sand, silt lumps, mixed and massive. (Fill).**

**Medium stiff, green gray, silty SAND (SM) with gravel: very moist, some clayey silt beds, some organics, slightly to low plastic or non-plastic, mixed, massive. (Fill).**

**Becoming stiff, SILT with sand and clay, some gravel, mixed occasional gray pockets.**

**Green gray SILT with fine sand, slightly plastic, sandy partings, rusty mottles, mixed.**

**Medium dense, dark gray GRAVEL with silt: moist, non-plastic fines, sub-rounded, fine to coarse, blocky, some granitic gravels, massive. (Alluvium).**

**Medium dense brown, sandy GRAVEL: wet, trace silt, sub-rounded and blocky, massive. (Alluvium).**

---

**Remarks:** STA 170+95, 6.5' Rt. Groundwater level measured in open augers on morning of 3/8/16 after drilling to 60 feet the previous day. Boring open overnight.
Medium dense, brown gray, silty, fine to medium SAND: wet, non-plastic fines, silt interlaminae, fine and coarser beds with both sharp and gradational contacts, laminated. (Alluvium). (Continued)

Very dense, brown gray, interbedded fine sandy SILT and silty, fine to coarse SAND: wet, non-plastic, rapid dilatancy, medium bedded (6 to 8+ inches), laminated, some sharp bedding contacts. (Drift).

Very dense, brown gray, silty, fine SAND: wet, brown clayey silt interlaminae, non-plastic and slightly plastic fines, rapid dilatancy, sub-horizontal laminae, beds to 6 inches. (Drift).

Silty, fine SAND, one brown, 2 inch, clayey SILT interbed, disturbed bedding.

Dense to very dense, brown gray, fine to medium SAND (SP-SM) with silt: wet, poorly graded, homogeneous, massive. (Drift).

Grading dark brown gray, fine to medium SAND, trace silt, occasional gravel, slightly weathered.

Dark, slightly reddish gray, fine to coarse SAND, fine / coarser beds, occasional fine gravel, laminated, gradational contacts.

Slightly reddish gray, fine to medium SAND, laminated, one dark seam (organic?).

Remarks: STA 170+95, 6.5' Rt. Groundwater level measured in open augers on morning of 3/8/16 after drilling to 60 feet the previous day. Boring open overnight.

The stratification lines represent approximate boundaries. The transition may be gradual.
**MATERIAL DESCRIPTION**

<table>
<thead>
<tr>
<th>Depth, (ft)</th>
<th>Sample No.</th>
<th>Sample Type</th>
<th>Other Tests</th>
<th>Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>80</td>
<td>S-16</td>
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<td></td>
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<td>85</td>
<td>S-17</td>
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<td>90</td>
<td>S-18</td>
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<td></td>
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<tr>
<td>95</td>
<td>S-19</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>S-20</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- **Sample No.**
- **Sample Type**
- **Other Tests**
- **Symbol**

**Remarks:** STA 170+95, 6.5’ Rt. Groundwater level measured in open augers on morning of 3/8/16 after drilling to 60 feet the previous day. Boring open overnight.

---

**LOG OF TEST BORING TH-2**

**Figure A-3**

The stratification lines represent approximate boundaries. The transition may be gradual.
Asphaltic concrete, 8 inches.
Medium dense to loose, brown, silty SAND and gravel. (Fill).

Medium stiff, brown with gray pockets, sandy SILT (ML) with clay: moist, slightly plastic, occasional organics, sandy lenses, broken textures, mixed, massive. (Fill).

Grading to green-brown and gray, clayey SILT, some fine sand, slightly plastic to low plastic, layered, occasional organics, rare gravel.

Mixed brown and gray, clayey SILT, some fine sandy bands, occasional organics, layered.

Thin, black silty, oily gravel bed at top (oiled gravel road bed) underlain by clayey SILT.

Medium dense, green-gray, silty, fine SAND: moist, non-plastic fines, pistachio green mottles, occasional gravel, 1 inch blacky oily, silty, sub-angular gravel band near tip. (Fill).

Dense, gray, SAND and GRAVEL: very moist. (Alluvium).

Medium dense, green-gray, interbedded silty, fine SAND (SM) and fine to coarse SAND with gravel: wet, non-plastic fines, rapid dilatancy, bedded and laminated, one organic laminae. (Alluvium).

Remarks: STA 171+60.5, 7.5' Rt. Groundwater level estimated from soil moisture and water observed on sampling rods.
<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample No.</th>
<th>Sample Type</th>
<th>Blows / 6 in.</th>
<th>Other Tests</th>
<th>Symbol</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>S-8</td>
<td>GS</td>
<td>1 2 &amp; 6</td>
<td></td>
<td></td>
<td>Loose, green-gray, silty, fine SAND (SM): wet, occasional gravel, non-plastic fines, wood debris. (Alluvium). (Continued)</td>
</tr>
<tr>
<td>45</td>
<td>S-9</td>
<td></td>
<td>13 19 21</td>
<td></td>
<td></td>
<td>Very dense, brown with gray lenses, fine to medium SAND with silt: wet, homogeneous, non-plastic fines with rapid dilatancy, trace coarse sand, laminated. (Drift).</td>
</tr>
<tr>
<td>50</td>
<td>S-10</td>
<td></td>
<td>16 26 23</td>
<td></td>
<td></td>
<td>Very dense, brown, silty, fine SAND: wet, homogeneous, non-plastic with rapid dilatancy, massive to laminated, clayey silt interlaminae in tip. (Drift).</td>
</tr>
<tr>
<td>55</td>
<td>S-11</td>
<td></td>
<td>14 23 28</td>
<td></td>
<td></td>
<td>Very dense, brown and brown gray, interbedded SILT and silt, fine SAND: wet, non-plastic to slightly plastic with rapid dilatancy, beds 6+ inches, laminated. (Drift).</td>
</tr>
<tr>
<td>60</td>
<td>S-12</td>
<td></td>
<td>10 17 28</td>
<td></td>
<td></td>
<td>Dense to very dense, brown gray, silty, fine to medium SAND: wet, homogenous, non-plastic with rapid dilatancy, 4 inch clayey silt interbed at top, hard, indistinctly laminated, small organic pocket. (Drift).</td>
</tr>
<tr>
<td>65</td>
<td>S-13</td>
<td></td>
<td>16 39 47</td>
<td></td>
<td></td>
<td>Grading to fine to medium SAND with silt, laminated, rare gravel.</td>
</tr>
<tr>
<td>70</td>
<td>S-14</td>
<td></td>
<td>16 31 50/3</td>
<td></td>
<td></td>
<td>Very dense, brown gray, interbedded, fine to coarse and fine to medium SAND: wet, trace to some silt, laminated, gradational contacts, homogeneous. (Drift).</td>
</tr>
</tbody>
</table>

Completion Depth: 71.3 ft
Date Borehole Started: 3/8/16
Date Borehole Completed: 3/8/16
Logged By: S. Evans
Drilling Company: Holocene Drilling

Remarks: STA 171+60.5, 7.5' Rt. Groundwater level estimated from soil moisture and water observed on sampling rods.
**LOG OF TEST BORING HB-1**

- **Project:** McDonald Creek Bridge Replacement
- **Job Number:** 15-130.911
- **Location:** Old Olympic Highway, Clallam Co., WA
- **Coordinates:** Northing: , Easting:

### MATERIAL DESCRIPTION

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Sample No.</th>
<th>Sample Type</th>
<th>Blows / 6 in.</th>
<th>Other Tests</th>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-2</td>
<td>S-1</td>
<td>GS</td>
<td></td>
<td></td>
<td></td>
<td>Light brown clayey, sandy SILT (ML), moist, with trace gravel (Alluvial Fan Deposits).</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Increase in clay fraction.</td>
</tr>
<tr>
<td>4-6</td>
<td>S-2</td>
<td>GS</td>
<td></td>
<td></td>
<td></td>
<td>Brown, silty, fine to medium SAND (SM) with gravel, moist. Bottom of hand boring.</td>
</tr>
</tbody>
</table>

**Remarks:** STA 166+15, 25' Rt.

- **Completion Depth:** 5.0 ft
- **Date Borehole Started:** 10/12/16
- **Date Borehole Completed:** 10/12/16
- **Logged By:** R. Kimmerling
- **Drilling Company:** PanGEO INCORPORATED

**Surface Elevation:** 167'

**Top of Casing Elev.:**

**Drilling Method:** Hand Augers

**Sampling Method:** Grab

The stratification lines represent approximate boundaries. The transition may be gradual.
**Material Description**

- **Light tan-brown clayey SILT (ML) with sand and trace gravel, moist** (Alluvial Fan Deposits).
- **Light tan-brown silty fine to medium SAND (SM) with trace gravel, moist.**
- **Bottom of hand boring.**

**Histogram**

- **N-Value ▲**
- **PL**
- **Moisture**
- **LL**

**Formulae**

- **RQD Recovery**

**PanGEO INCORPORATED**

LOG OF TEST BORING HB-2

The stratification lines represent approximate boundaries. The transition may be gradual.
**LOG OF TEST BORING HB-3**

**Location:** Old Olympic Highway, Clallam Co., WA

- **Project:** McDonald Creek Bridge Replacement
- **Job Number:** 15-130.911
- **Drilling Method:** Hand Augers
- **Sampling Method:** Grab
- **Surface Elevation:** 171’
- **Top of Casing Elev.:**
- **Date Borehole Started:** 10/12/16
- **Date Borehole Completed:** 10/12/16
- **Logged By:** R. Kimmerling

### MATERIAL DESCRIPTION

- **S-1**
  - **Sample Type:** GS
  - **Other Tests:** Light tan-brown slightly sandy SILT (ML), moist (Alluvial Fan Deposits).

- **S-2**
  - **Sample Type:** GS
  - **Other Tests:** Light tan-brown silty SAND (SM) with gravel, moist.
  - **Remarks:** STA 163+85, 27’ Lt.

The stratification lines represent approximate boundaries. The transition may be gradual.

---

*PanGEO INCORPORATED*

**Figure A-7**

**Sheet 1 of 1**
Brown silty fine to medium SAND (SM) with some gravel, moist (Alluvial Fan Deposits).

Abundant gravel at 2.5 feet. Unable to advance hand boring further; practical refusal.

Bottom of hand boring.
APPENDIX B

LABORATORY TEST RESULTS
APPENDIX B: LABORATORY TEST RESULTS

This appendix contains descriptions of the procedures and results of physical (geotechnical) laboratory testing conducted on soil samples retained during the field explorations for the McDonald Creek Bridge Replacement Project located on Old Olympic Highway in Clallam County, Washington. The methodology of the soil sampling from the borings was described in Appendix A. The samples were tested to determine basic physical index properties of the soils for purposes of classifying the material types encountered and to measure or correlate parameters used in the geotechnical design.

Laboratory testing was performed in general accordance with the following ASTM Standard Test Methods (STM):

D 2216 STM for Laboratory Determination of Water (Moisture) Content of Soil and Rock
D 422 STM for Particle-Size Analysis of Soils

Moisture contents are shown on the logs of the test borings and hand hole in Appendix A.

Grain size results are shown on Figures B-1 through B-4.
### Grain Size Distribution

<table>
<thead>
<tr>
<th>Specimen Identification</th>
<th>Classification</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>Cc</th>
<th>Cu</th>
</tr>
</thead>
<tbody>
<tr>
<td>TH-1 @ 16.0 ft.</td>
<td>Silty SAND with Gravel (SM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TH-1 @ 31.0 ft.</td>
<td>Silty SAND (SM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TH-1 @ 46.0 ft.</td>
<td>Silty SAND (SM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TH-2 @ 16.0 ft.</td>
<td>Silty SAND with Gravel (SM)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>TH-2 @ 61.0 ft.</td>
<td>Poorly graded SAND with Silt (SP-SM)</td>
<td>1.63</td>
<td>3.52</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

### Specimen Identification

<table>
<thead>
<tr>
<th>Specimen Identification</th>
<th>D100</th>
<th>D60</th>
<th>D30</th>
<th>D10</th>
<th>%Gravel</th>
<th>%Sand</th>
<th>%Silt</th>
<th>%Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>TH-1</td>
<td>16.0</td>
<td>38.1</td>
<td>1.68</td>
<td>0.198</td>
<td>31.4</td>
<td>50.9</td>
<td>17.7</td>
<td></td>
</tr>
<tr>
<td>TH-1</td>
<td>31.0</td>
<td>9.525</td>
<td>0.148</td>
<td>0.6</td>
<td>0.6</td>
<td>56.0</td>
<td>43.4</td>
<td></td>
</tr>
<tr>
<td>TH-1</td>
<td>46.0</td>
<td>9.525</td>
<td>0.278</td>
<td>0.106</td>
<td>1.6</td>
<td>75.5</td>
<td>22.9</td>
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</tr>
<tr>
<td>TH-2</td>
<td>16.0</td>
<td>19.05</td>
<td>0.246</td>
<td>0.118</td>
<td>19.7</td>
<td>40.9</td>
<td>39.4</td>
<td></td>
</tr>
<tr>
<td>TH-2</td>
<td>61.0</td>
<td>12.7</td>
<td>0.416</td>
<td>0.283</td>
<td>1.9</td>
<td>90.4</td>
<td>7.7</td>
<td></td>
</tr>
</tbody>
</table>

**Figure**: B-1

**Project**: McDonald Creek Bridge Replacement

**Job Number**: 15-130.911

**Location**: Old Olympic Highway, Clallam Co., WA
**Specimen Identification**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Identification</th>
<th>Classification</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>Cc</th>
<th>Cu</th>
</tr>
</thead>
<tbody>
<tr>
<td>HB-1</td>
<td>@ 2.5 ft.</td>
<td>Sandy Silt (ML)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HB-1</td>
<td>@ 5.0 ft.</td>
<td>Silty Sand with Gravel (SM)</td>
<td>0.76</td>
<td>29.87</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HB-2</td>
<td>@ 2.5 ft.</td>
<td>Silty Silt (ML)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HB-2</td>
<td>@ 5.0 ft.</td>
<td>Silty Silt (SM)</td>
<td>2.30</td>
<td>100.65</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

**Specimen Identification**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Identification</th>
<th>D100</th>
<th>D60</th>
<th>D30</th>
<th>D10</th>
<th>%Gravel</th>
<th>%Sand</th>
<th>%Silt</th>
<th>%Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>HB-1</td>
<td>@ 2.5 ft.</td>
<td>2.5</td>
<td>9.5</td>
<td>0.057</td>
<td>0.014</td>
<td>0.9</td>
<td>34.4</td>
<td>46.5</td>
<td>18.2</td>
</tr>
<tr>
<td>HB-1</td>
<td>@ 5.0 ft.</td>
<td>5.0</td>
<td>38.1</td>
<td>1.227</td>
<td>0.196</td>
<td>0.041</td>
<td>30.1</td>
<td>57.0</td>
<td>9.6</td>
</tr>
<tr>
<td>HB-2</td>
<td>@ 2.5 ft.</td>
<td>2.5</td>
<td>9.5</td>
<td>0.019</td>
<td>0.004</td>
<td>0.4</td>
<td>15.6</td>
<td>52.2</td>
<td>31.8</td>
</tr>
<tr>
<td>HB-2</td>
<td>@ 5.0 ft.</td>
<td>5.0</td>
<td>19.05</td>
<td>0.2</td>
<td>0.03</td>
<td>0.002</td>
<td>5.6</td>
<td>55.3</td>
<td>25.2</td>
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</table>

**Gra Number:** B-3

**Project:** McDonald Creek Bridge Replacement

**Job Number:** 15-130.911

**Location:** Old Olympic Highway, Clallam Co., WA
**Specimen Identification**

<table>
<thead>
<tr>
<th>Specimen Identification</th>
<th>Classification</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>Cc</th>
<th>Cu</th>
</tr>
</thead>
<tbody>
<tr>
<td>HB-3 @ 2.5 ft.</td>
<td>SILT (ML)</td>
<td></td>
<td></td>
<td></td>
<td>1.27</td>
<td>123.30</td>
</tr>
<tr>
<td>HB-3 @ 5.0 ft.</td>
<td>Silty SAND with Gravel (SM)</td>
<td></td>
<td></td>
<td></td>
<td>1.00</td>
<td>187.73</td>
</tr>
<tr>
<td>HB-4 @ 2.5 ft.</td>
<td>Silty SAND with Gravel (SM)</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Specimen Identification**

<table>
<thead>
<tr>
<th>Specimen Identification</th>
<th>D100</th>
<th>D60</th>
<th>D30</th>
<th>D10</th>
<th>%Gravel</th>
<th>%Sand</th>
<th>%Silt</th>
<th>%Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>HB-3</td>
<td>2.5</td>
<td>9.5</td>
<td>0.021</td>
<td>0.004</td>
<td>0.9</td>
<td>12.8</td>
<td>54.8</td>
<td>31.5</td>
</tr>
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<td>HB-3</td>
<td>5.0</td>
<td>25.4</td>
<td>0.233</td>
<td>0.024</td>
<td>0.002</td>
<td>18.8</td>
<td>39.4</td>
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<tr>
<td>HB-4</td>
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<td>19.05</td>
<td>0.269</td>
<td>0.02</td>
<td>0.001</td>
<td>19.2</td>
<td>41.1</td>
<td>22.2</td>
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</tbody>
</table>

**GRAIN SIZE DISTRIBUTION**

- **Project:** McDonald Creek Bridge Replacement
- **Job Number:** 15-130.911
- **Location:** Old Olympic Highway, Clallam Co., WA

Figure B-4