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**Report of Geotechnical Services**
Noble Creek Tidelands Project
Coos County, Oregon
Project #014-21-006-02

## 1.0 INTRODUCTION AND SCOPE OF SERVICES

Pali Consulting, Inc., is pleased to submit this report of geotechnical services for the Noble Creek Tidelands project (Project) in Coos County, Oregon. The location of the Project site is shown on Figure 1. Our work was completed in general accordance with our Services Agreement with Waterways Consulting, Inc. (Waterways) dated April 11, 2023.

The project includes replacing the existing culverts at the intersection of Green Acres and Upper Loop Lanes. Based on site conditions and discussions with the project team, a pile supported bridge with pile-supported side hinge tide gates is proposed to replace the existing culverts. Additionally, two single barrel culverts with tide gates are planned, one through the levee south of Noble Creek, and the other at Bramble Lane. Finally, the existing levee on the south side of Noble Creek adjacent to an existing residence may be raised up to 2 feet. The locations of these site features are shown on Figure 2.

New approaches are anticipated to be at similar grade to the current roadway, but with slightly widened shoulders to accommodate guardrails, resulting in wedge fills on each side of the abutments. A temporary bypass road may be constructed to accommodate traffic during construction. Details of the bypass, such as fill thickness, width and alignment, have not yet been determined. Wing walls will extend beyond the bridge to support the approach fills. The bridge is being designed in accordance with Chapter 12 of the American Association of State Highway Transportation Officials (AASHTO) Load Resistance Factor Design Bridge Design Specifications (AASHTO BDS) (AASHTO 2020), the Oregon Department of Transportation (ODOT) Geotechnical Design Manual (ODOT GDM) (ODOT 2023), and the ODOT Bridge Design Manual (ODOT BDM) (ODOT 2022). The following report provides geotechnical design recommendations for the bridge in accordance with the above guidelines.

The purpose of our work was to evaluate subsurface conditions at the site and to provide geotechnical recommendations for design of the project. Our complete scope of work was described in our agreement with Waterways, and generally included the following:

- Reviewed information from our preliminary report for the project and additional information as available.
- Completed subsurface explorations, including drilling five mud rotary borings, maintaining logs of the soil and groundwater conditions encountered, and collecting soil samples for laboratory testing.
- Performed laboratory tests on select samples to evaluate engineering characteristics.
- Completed an evaluation of our findings, including the suitability of the site to support the project.
- Prepared this report summarizing our findings, conclusions, and recommendations, including interpreted subsurface conditions as they affect the Project, seismic performance of site soils, and geotechnical recommendations for design and construction of project elements.

# 2.0 BACKGROUND REVIEW

## 2.1 PREVIOUS REVIEW

We reviewed site geology, soils, and aerial photographs of the site in preparing our preliminary report. Our preliminary report is included as Appendix A, so information from it is not repeated in this report.

## 2.2 GEOLOGIC HAZARDS

Geologic hazards were not reviewed in preparing our preliminary report. Geologic hazards were reviewed in preparing this report using DOGAMI's Statewide Geohazards Viewer (HAZVU). Geologic hazards mapped at the site include landslides, shaking, liquefaction, flooding, and tsunami inundation. Mapped landslide hazard is low to moderate at the site, with moderate risk mapped at the Green Acres Lane culvert replacement site and sporadically along the alignment of the existing levee, and low risk elsewhere within the project area. The landslide hazards are from localized steep riverbank and levee slopes. Very strong to violent earthquake shaking from Cascadia and local earthquakes and moderate to high liquefaction hazard are mapped at the site, as well. The nearest mapped active fault is located approximately 2 miles northwest of the site. The site is within the 100-year floodplain so may intermittently be affected by flooding. The site is also partially located below the statutory tsunami inundation line, meaning it may be impacted by tsunamis caused by Cascadia and local earthquakes. The earthquake shaking, liquefaction and other seismic hazards are explained in further detail in *Section 4.1*.

# 3.0 SITE CONDITIONS

## 3.1 SURFACE CONDITIONS

Surface conditions were described in our preliminary report at the Green Acres/Upper Loop bridge location (referred to as "Tidegate" location in the preliminary report), along the Green Acres Lane road embankment, and at the Existing Levee. The Green Acres Lane road embankment is no longer included in our scope of work. Conditions at the Green Acres/Upper Loop bridge replacement and Existing Levee locations are described in the preliminary report, included in Appendix A, so are not repeated here.

During our subsurface explorations, we observed the following additional conditions at the Bramble Lane culvert site. Bramble Lane consists of a gravel roadway which is generally in good repair. Gently sloping banks on either side of the roadway were vegetated with grass, reeds, and brush such as blackberry. At the culvert replacement location, the existing culvert consisted of a rusted metal pipe approximately 12 inches in diameter. A tidegate in generally working condition was installed on the north side of the culvert. Minor sloughing of the creek banks was observed at the site, but no evidence of widespread erosion or subsidence was present at the time of our visit.

## 3.2 Subsurface Conditions

Shallow subsurface conditions from our preliminary report are included in Appendix A. Explorations completed for this geotechnical design report, and the findings from those explorations, are summarized below.

### 3.2.1 General

We completed explorations at the site on May 8th, 9th, 23rd, and 24th, 2023, by advancing five borings at the locations shown on Figure 2. The borings were completed using mud rotary methods and extended up to 66.5 feet below ground surface (bgs). A description of our subsurface explorations, logs of our borings, and the results of our laboratory testing on select soil samples are included in Appendix B.

### 3.2.2 Soil Conditions

Our subsurface explorations found the site to be underlain by three general soil units: Fill / Borrow Material placed for construction of the existing roadway and levee, Alluvium of Noble Creek, and Tyee Formation Bedrock. Descriptions of these units and summaries of laboratory test results from each are provided below. Cross sections through the bridge and along the levee are included in Figures 3A and 3B, respectively.

#### *Fill*

Our explorations encountered very soft to medium stiff, brown to yellow varicolored sandy silts and clays that we interpret as road fill in Borings B-1 and B-2, and levee fill in Borings B-4 and B-5. In Borings B-1 and B-2 at the Bridge location, the fill underlies about 8 inches of asphaltic concrete (AC) pavement and base rock. The fill extended to depths of up to 5 feet bgs. The fill was distinguished from similar native soils by color, blocky appearance, and sand content. SPT samples taken in the fill measured blow counts (N-values) of 0 to 8 blows per foot (bpf), with an average N-value of 3. Moisture contents in the samples tested were found to range from 33 to 54 percent, with an average of 43 percent. Levee fill in Borings B-4 and B-5 was found to be soft to very soft fat clay to clay, similar to the underlying native elastic silt soils. N-values ranged from 0 to 2 bpf, moisture contents on two samples measured 53 and 54 percent, and one test for dry density measured 66 pounds per cubic foot (pcf).

#### *Alluvium (Bridge Location) (Borings B-1 and B-2)*

Alluvium was encountered underlying the fill at the Bridge location (Borings B-1 and B-2) which extended to about 35 feet bgs. The Alluvium consisted of very soft to soft, moist to wet elastic silts to fat clays with lesser very loose to loose, moist to wet silty / clayey sands. Wood fragments, charcoal, and other organic materials were present in many of the samples. SPT N-values in this alluvium ranged from 0 to 4, with an average of 2. Moisture contents ranged from 33 to 83 percent, with an average of 62 percent. Moisture content generally decreased with depth and was lower in samples with a high percentage of coarse-grained material. Fines content testing found fines to range from 47 to 64 percent in two samples tested. Based on testing and our observations during drilling, we find that these soils probably transition without abrupt contacts between coarse- and fine- grained. Atterberg limits testing of the fines portion of five samples of the Bridge location measured Plasticity Indices (PI's) ranging from 26 to 71, resulting in MH to CH designations.

#### *Alluvium (Bramble Lane) (Boring B-3)*

Our exploration at the Bramble Lane culvert encountered alluvium for the entire depth of Boring B-3. The Bramble Lane Alluvium consisted of very soft to soft, moist to wet clay and very loose, moist to wet clayey sand. Wood fragments, charcoal, and other organic materials were present in all of the samples. SPT N-values in this alluvium ranged from 0 to 3, with an average of 1. Moisture contents ranged from 47 to 129 percent, with an average of 83 percent. Moisture content generally varied with organic content and was lower in samples with a high percentage of coarse-grained material. Based on our observations during drilling, we find that these soils probably transition without abrupt contacts between coarse- and fine-grained.

#### *Alluvium (Levee) (Borings B-4 and B-5)*

Alluvium was encountered underlying the levee in Borings B-4 and B-5, extending to about 55 feet bgs in B-4 and over 65 feet bgs in B-5. The Levee Alluvium consisted of very soft to medium stiff, moist to wet fat clay with minor clayey sand and organic silt. Wood fragments, charcoal, and other organic materials were present in many of the samples. SPT N-values in this alluvium ranged from 0 to 8 bpf, with an average of 1 bpf. Moisture contents ranged from 42 to 115 percent, with an average of 72 percent. Moisture content generally varied with organic content and was lower in samples with greater coarse-grained material. Fines content testing found fines to range from 53 to 75 percent in two samples tested. Based on testing and our observations during drilling, we find that these soils probably transition without abrupt contacts between coarse- and fine-grained. Atterberg limits testing of the fines portion of five samples of the Levee Alluvium measured Plasticity Indices (PI's) ranging from 88 to 98, resulting in CH designations.

#### *Tyee Formation Bedrock and Transition Zone (Borings B-1, B-2 and B-4)*

A residual soil / weathered bedrock zone (Transition Zone) was encountered underlying the Alluvium in Borings B-1, B-2, and B-4. In Borings B-1 and B-2, Tyee Formation Bedrock was encountered underneath the Transition Zone. The Transition Zone consisted of medium stiff to hard, wet, elastic silt to fat clay. Organics were minimal or not present. SPT N-values ranged from 8 to 40, with an average of 19. Moisture contents ranged from 37 to 53 percent, with an average of 44 percent. Fines content testing found 60 percent fine-grained material in one sample tested. Underlying the transition zone in Boring B-1 and B-2, the Tyee Formation Bedrock consisted of soft to medium hard dark grey siltstone to claystone with platy rock structure. SPT N-values were >100 bpf (refusal). From the SPT N-values, we interpret the bedrock as soft to medium hard (R2-R3) per Table 23 of the ODOT Soil and Rock Classification Manual (ODOT, 1987).

### *3.2.3 Groundwater*

Depths to groundwater were not determined due to the mud rotary drilling method used. However, based on saturation of the samples, the proximity of the site to Noble Creek, and the contact between the fill and native alluvium, we estimate that groundwater levels are about 5 feet bgs at the bridge and levee sites (which were drilled on top of fill embankments) and reach at or near the ground surface at the adjacent flood plain during periods of high precipitation. We note that groundwater can vary seasonally and due to other factors, such as precipitation, changes in land use and others.

# 4.0 GEOTECHNICAL ANALYSIS

## 4.1 SEISMIC ANALYSIS

The project site is in a seismically active area. The seismicity of the area is controlled largely by the Cascadia Subduction Zone (CSZ). Plate tectonics cause the oceanic Juan de Fuca Plate to subduct beneath the continental North American Plate. Three types of earthquakes are associated with subduction zones: intraslab, interface, and crustal earthquakes. Contributions from each of these sources to the total site seismic hazard was evaluated using the USGS Unified Hazard Tool website (USGS, 2014).

**Intraslab and Interface Sources**. Subduction zones are characterized by the interaction of the oceanic Juan de Fuca Plate and continental North American Plate. As the oceanic plate subducts beneath the continental plate, the two plates lock together, developing stresses in the plates. When the magnitude of the stresses becomes too large, the plates rupture causing an interface earthquake. Interface earthquakes (such as the 2011 magnitude M9.0 Tohoku earthquake in northern Japan) are some of the largest magnitude earthquakes on record.

Intraslab earthquakes originate from a deeper zone of seismicity that is associated with bending and breaking of the subducting Juan de Fuca Plate. Intraslab earthquakes (such as the 2001 magnitude M7.0 Nisqually earthquake in west central Washington) occur at depths of 40 to 70 kilometers (km) and can produce earthquakes with magnitudes up to and greater than magnitude M7.0. Our review of the interactive deaggregations indicate interface and intraslab earthquakes under the AASHTO event (AASHTO 2020) contribute approximately 86 percent of the total seismic hazard to the site.

**Crustal Sources**. Shallow crustal faults are caused by cracking of the continental crust resulting from the stress that builds as the subduction zone plates remain locked together. According to the USGS Quaternary Fault and Fold Database, the nearest mapped fault is approximately 2 miles northwest of the site. Our review of the interactive deaggregations indicate surface faults and other potentially unmapped crustal sources contribute the remaining approximately 14 percent of the total seismic hazard under the AASHTO 2020 design event to the site. Details on this event are provided in the following section.

### 4.1.1 Seismic Shaking

We evaluated potential seismic shaking at the site in accordance with the ODOT BDM (ODOT 2022, accessed at https://www.oregon.gov/odot/Bridge/Guidance/BDM-2022-10-01.pdf), which requires new bridges built west of US 97 to be designed for two levels of seismic performance criteria: Life Safety and Operational (ODOT BDM Section 1.17.2.1). The "Life Safety" criteria considers the design earthquake to be seismic shaking having a 7 percent probability of exceedance in 75 years (975-year return period); this is the standard AASHTO seismic design criteria (AASHTO, 2020). The additional "Operational" criteria requires the generation of a deterministic Design Response Spectra corresponding to a full rupture of a CSZ earthquake, as outlined in Section 4.1.3.

We evaluated potential seismic shaking at the site using data obtained from the updated ODOT Seismic Hazard Maps which are based on the USGS 2014 seismic shaking maps (ODOT 2016). The expected peak ground acceleration (PGA) at the site for the "Life Safety" criteria (975-year return period motion) is approximately 0.603g based on the ODOT (2016) maps (PGA on soft rock, $V_{s,30}$=760 m/sec). This value represents the peak acceleration on bedrock beneath the site and does not account for ground motion amplification due to site-specific effects. The site-adjusted PGA ($A_s$) is determined by applying a site class factor to the PGA noted above and is presented in Section 4.1.3. Refer to Section 4.1.2 below for a discussion of ground motion amplification.

Seismic sources contributing to the potential ground shaking above include shallow crustal faults, intraplate faults, and the CSZ megathrust interface fault. The data indicated that the "modal source" for shaking at the site under the 975-year design interval (Life Safety criteria) at all potential periods of interest (0.0 to 2.0 seconds) is a magnitude 9.1 earthquake epicentered at the CSZ approximately 27 km from the site. The modal source generally signifies the earthquake with the highest contribution to the site earthquake hazard, in this instance a rupture along the CSZ.

### 4.1.2 Seismic Site Class

Thick sequences of unconsolidated, soft sediments typically amplify the shaking of long-period ground motions, such as those associated with subduction zone earthquakes; whereas areas underlain by shallow soil profiles are not likely to amplify seismic waves.

The "Site Class" is a designation used to quantify ground motion amplification. The classification is based on the stiffness in the upper 100 feet of soil and bedrock materials at a site, as evaluated using SPT, CPT, or shear wave velocity data. Based on the two borings we completed at the crossing, the upper 100 feet of the site has average N-values of approximately 8 and 17 bpf in borings B-1 and B-2, respectively. The average N-values noted above correspond to Site Classes E and D, respectively. However, we note that these site classes do not consider potential liquefaction of site soils, as discussed in Section 4.1.4.1. If Site Class designations are needed at the other locations, we recommend Site Class E.

### 4.1.3 Design Response Spectrum

We obtained seismic design parameters for the 975-year AASHTO design event (AASHTO, 2020) and for the Operational Criteria under the CSZ event (ODOT, 2022) at Latitude 43.2561 and Longitude -124.2154. The parameters provided in Table 1 were developed using the ODOT ARS Spreadsheet (ODOT, V.2014.16), except $F_a$ for Site Class E soils (and $S_{DS}$ which is derived from $F_a$). $F_a$ for these soils was determined by interpolation from AASHTO LRFD (Edition 9, 2021), Table 3.10.3.2-2., as the ODOT GDM (ODOT 2023) does not provide $F_a$ for Site Class E with $S_s$ greater than 0.75. We developed the parameters in Table 1 for both Site Classes E and D for Borings B-1 and B-2, respectively. Site Class E parameters would apply to the other locations, if needed.

The parameters in Table 2 were developed using Portland State University's web application, Acceleration Response Spectrum for Full Rupture CSZ Earthquake (http://csz.cee.pdx.edu/). Because it requires the use of seismic velocities, rather than site classes, we developed the parameters using a seismic velocity of 200 m/sec. The USGS (2014) considers the Class D/E boundary as 180 m/sec. The PSU website does not allow velocities below 200 m/sec, so we developed the parameters using 200 m/sec, the closest to the USGS velocity of 180 m/sec for the D/E boundary allowed. It is our opinion that the parameters developed in Table 2 are applicable to both borings.

The values provided in Tables 1 and 2 are considered generally appropriate for AASHTO BDS and ODOT BDM code-based seismic design, respectively, except for liquefaction, as noted above.

Table 1 - Seismic Design Parameters for "Life Safety" Criteria

| Parameter | Value | |
|---|---|---|
| | B-1 | B-2 |
| Site Class | E | D |
| Mapped Spectral Response Acceleration (Short Period), $S_s$ | 1.2625 | 1.2625 |
| Mapped Spectral Response Acceleration (1-Second Period), $S_1$ | 0.4879 | 0.4879 |
| Peak Ground Acceleration Coefficient, $F_{pga}$ | 1.1032 | 1.1000 |
| Short Period Spectral Acceleration Coefficient, $F_a$ | 0.9 | 1.000 |
| Long Period Spectral Acceleration Coefficient, $F_v$ | 2.2242 | 1.8121 |
| $A_s$ ($F_{pga}$ x PGA) | 0.6584 | 0.6565 |
| Spectral Response Acceleration (Short Period), $S_{DS}$ | 1.1363 | 1.2625 |
| Spectral Response Acceleration (1-Second Period), $S_{D1}$ | 1.0852 | 0.8842 |
| Peak Ground Acceleration (PGA) | 0.5968 | 0.5968 |

Table 2 – Design Response Spectra for "Operational" Criteria

| Period, T (s) | Sa (g)[1] |
|---|---|
| 0.00 | 0.42378 |
| 0.05 | 0.44483 |
| 0.1 | 0.59787 |
| 0.15 | 0.73673 |
| 0.2 | 0.80782 |
| 0.25 | 0.87562 |
| 0.3 | 0.94563 |
| 0.4 | 0.99169 |
| 0.5 | 0.98742 |
| 0.6 | 0.92378 |
| 0.7 | 0.94188 |
| 0.8 | 0.93091 |
| 1 | 0.79503 |
| 1.5 | 0.57339 |
| 2 | 0.42763 |
| 2.5 | 0.33153 |
| 3 | 0.26197 |

### 4.1.4 LIQUEFACTION HAZARDS

#### *4.1.4.1 General*

When cyclic loading occurs during an earthquake, the shaking can increase the pore pressure in some soils and cause liquefaction. The rapid increase in pore water pressure reduces the effective normal stress between individual soil particles, resulting in the sudden loss of shear strength in the soil. Granular soils, which rely on interparticle friction for strength, are susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soils with low silt and clay contents are the most susceptible to liquefaction. Silty soils with low plasticity are moderately susceptible to liquefaction under relatively higher levels of ground shaking. For any soil type, the soil must be saturated for liquefaction to occur.

We reviewed our laboratory test results per ODOT's three criteria for liquefaction: stiffness, plasticity, and MC/LL ratios.  Based on these criteria both borings at the bridge have a zone of fill, elastic silt, and silty sand that may undergo liquefaction and/or cyclic softening: from depths of about 5 feet bgs (estimated depth of the water table ) to 25 feet bgs in Boring B-1 (east abutment) and to 20 feet bgs in Boring B-2 (west abutment). Soils at the Bramble Lane crossing are expected to liquefy to the depth explored as well (21.5 feet bgs). Soils beneath the levee are not expected to liquefy based on ODOT's screening criteria, although strain softening is likely.

Since liquefaction screening indicates liquefaction is likely in some of the borings, in particular at the bridge, we performed a site-specific liquefaction potential analysis based on the materials encountered in our exploration, using the software program Liquefy Pro. The analyses were conducted using field data from Boring B-1 and with the results of laboratory fines content and plasticity tests completed on samples from the site. We completed the liquefaction hazard analysis using the M-R pair from the modal earthquake based on the 975-year USGS deaggregation (USGS, 2014), which results in a magnitude 9.1 earthquake at 27 km from the site with a PGA of 0.603 on bedrock (Site Class B).

Our liquefaction analyses confirmed liquefaction is very likely to occur during a design-level earthquake at the locations noted.

#### *4.1.4.2 Seismically Induced Settlement*

Post-seismic liquefaction settlement results from densification of liquefiable coarse-grained soils and reconsolidation of finer-grained soils. We note that the settlement is typically not uniform across an area and can result in significant differential settlement. Differential settlement will have the most significant impact on structures supported by shallow foundations.

Based on our analyses, liquefaction-induced settlement and post-seismic reconsolidation are likely at the site. We estimate total settlement of about 1 foot at the B-1 abutment and about one-half foot at the B-2 abutment. The corresponding differential settlement is estimated to be up to half that amount (3 to 6 inches).

#### *4.1.4.3 Lateral Spread*

Lateral spreading occurs when large blocks of ground are displaced down gentle slopes or towards a free face, such as a stream bank channel, as a result of earthquake-induced inertial forces acting on the soil mass. Initiation of lateral spreading is often made worse when the soils within and beneath the soil mass liquefy or soften as a result of the shaking. Lateral spreading deformations can be experienced relatively far from a free face.

Like lateral spread, flow failures result when large volumes of soil near the free face displace vertically and laterally during or after earthquakes. As the ground begins to shake and the shearing resistance of liquefied soils decreases, ground displacement occurs in response to mainly static shear forces present within the soil mass and to a lesser extent earthquake-induced-inertial force. Flow failures typically manifest larger deformations than lateral spreading; however, the extent of the deformations is typically localized to the area behind the free face of the channel. Both lateral spreading and flow failures occurring within zones of deep or shallow foundations are destructive and pose a significant risk to the structures the foundations support.

We completed a lateral spreading analysis using the procedures of Bartlett and Youd (2002). The analysis was completed using modal earthquake per Section 4.1.4.1. The results of the analysis indicate that the streambanks could be subject to minor lateral spreading deformations as a result of cyclic softening in the soils. Our analysis indicates that lateral spread deformations should be less than about 6 inches. Lateral spreading would include the non-liquefied roadway embankment (approximately 5 feet thick) plus liquefied soils from about 5 feet bgs (the water table) to bottom of creek elevation (approximately 11 feet thick zone between elevations 6 feet to -5 feet MSL).

### 4.1.5 FAULT SURFACE RUPTURE

As noted previously, the nearest mapped fault is approximately 2 miles northwest of the site. Therefore, we consider the hazard from ground surface rupture on mapped faults to be relatively low. Unmapped faults may still exist that could increase the risk of ground fault rupture at the site.

## 4.2 STATIC SETTLEMENT

We estimated static settlement for Fills associated with roadway construction. Our methods and estimated settlement magnitudes are provided below. Settlement related to levee modifications is addressed in Section 4.3, separately.

Structural fills at the site may include the following:

1. Permanent wedge fills along the top edges of the existing embankment to allow minor shoulder widening for guardrails. These fills are expected to be about 2 to 3 feet wide and extend along the tops of the existing embankments for some 50 feet or so in either direction.
2. Fill for a temporary bypass road. An exact alignment and configuration have not yet been determined, but settlement from a likely configuration is provided. This fill will be left in place during construction and then removed once bridge construction is complete.

Our estimates of settlement from these fill sources are provided in the following sections.

### 4.2.1 WEDGE FILLS

Wedge fills placed to widen the embankment for placing guardrails will have outside slope gradients less than the underlying embankment slopes which are generally oversteepened, so these wedge fills are expected to extend down to the native flood plain and be wider at the base than at road elevation. To estimate settlement from these fills, we modeled them strips of soil, infinitely long and with the dimensions noted above. We used the results of our consolidation testing and laboratory results to calculate the settlement based on one-dimensional consolidation theory. Based on these methods, we estimate settlements will be on the order of about 6 inches from typical wedge fills up to 3 feet wide and with 2H:1V outside slopes. The settlement will be greatest along the edges of the embankments with decreasing magnitudes at the roadway centerline and also past the bottoms of the fills.

Because of the silty and clayey nature of the soils underlying the roadway, settlements are expected to take about 6 months to reach primary consolidation with some secondary consolidation likely lasting for a year or more, but of lesser magnitude.

### 4.2.2 TEMPORARY ROADWAY FILLS

A temporary bridge may be constructed which will require temporary roadway fills. Assuming a one-way bridge/road, we estimate fills would be an average of 14 feet wide. Fill thicknesses will be minimized as practical, and the final depth of fill is not known. For fills 5 and 10 feet thick, we estimate consolidation settlement of approximately 8 inches and 17 inches, respectively with the greatest occurring near the roadway centerline, and tapering off past the edges of the temporary roadway fill. Some settlement is expected to extend laterally beyond the roadway fill for a distance approximately equal to the roadway width. Most settlement will occur within a similar time frame as described in Section 4.2.1.

## 4.3 LEVEE MODIFICATIONS

We understand that the levee may be raised up to an additional 2 feet in height. Other reconstruction of the levee, however, is not planned. Our borings found that the levee is composed of soft to very soft clay and fat clay soils over a foundation of very soft fat clay soils. Increasing the height of the levee could cause adverse geotechnical conditions, including slope instability and excessive settlement. During a design seismic event, the performance of the levee would be greatly reduced from strong shaking (instability), additional ground settlement, and lateral spreading.

Based on our experience, we recommend that if levee raising is completed, it should be raised using a configuration that results in a maximum 3 horizontal to 1 vertical (3H:1V) slope on the water side, and maximum 2H:1V slope on the ground side. The water side slope should be set back from a projected 3H:1V line from the base of the existing levee, or lowest point within the levee slope. The minimum recommended crest width by the USACE (2000) is 12 feet. We understand that a narrower crest may be acceptable, so we analyzed a 6 feet wide crest. The cross section of the existing levee is highly variable, with slopes ranging from 3H:1V to 1H:1V. Our recommended configuration for raising the levee 2 feet vertically at a typical existing levee cross section and with a 6 feet wide crest is shown on Figure 4. A 12 feet wide crest would be preferred, and have the same geometry on the water side but then extend 6 feet further to a 2H:1V slope down to the floodplain on the land side.

Our analyses for settlement and instability of the above configuration are provided in the following sections.

### 4.3.1 STATIC SETTLEMENT

We completed our evaluation of static settlement based on the slope configuration noted above, a 6 feet wide crest, and a 2 feet increase in levee height. Although a minimum crest width of 12 feet is typical (USACE 2000), a crest width of only 6 feet was used since vehicles are not anticipated on the levee surface and fill volumes and resulting costs are preferred to be minimized. We modeled the levee fill as an infinitely long rectangular load with a cross section equal to the area of the additional fill in Figure 4 (3 feet thick by 12 feet wide).

Using one-dimensional consolidation methods and the results of our consolidation test from a sample collected from beneath the levee, we estimate the following magnitudes of settlement from the analyzed cross section:

- Approximately 4 inches of primary consolidation settlement. This will occur over an estimated period of about 3 to 12 months.
- Approximately 2 inches of secondary consolidation settlement. This will occur over an estimated period of about 20 years.

We note that a 12 feet wide crest approximately doubles the magnitude of estimated settlement and increases earthworks volume by at least 100 percent.

### 4.3.2 EMBANKMENT STABILITY

Due to the very soft soil and high groundwater conditions, it is difficult to complete meaningful slope stability analyses under existing and modified conditions. Reasonable soil strengths for existing levee soils show that it is marginally stable under static conditions. So, adding additional height to the marginally stable levee will further decrease stability. Seismic and rapid drawdown conditions further reduce the stability. We understand, however, that the existing levee is not required to meet specific design FS since it is a private levee and constructed primarily for habitat purposes and to protect some pasture land. One residence may be protected by the levee, and it belongs to the owners of the levee, so they are aware of and accept the marginal stability of the levee.

In addition to accepting the marginal stability of the existing levee, we understand that the owner wishes to move forward with raising the levee while minimizing the risk of failure, as practicable (i.e., keeping the costs to do so as low as possible). To analyze the relative stability of the levee under existing and proposed configurations, we completed stability analyses using the two-dimensional commercial software Slide2. We used a rotational failure model and homogenous soil conditions throughout the embankment and subgrade. Graphic results of our analyses are provided in Appendix C. Our comparative analyses are summarized in Table 3. These results are for 6- and 12- feet crest widths and using estimated soil properties.

Table 3 – Comparative Stability Results (FS)

| Case | Existing | Proposed | |
|---|---|---|---|
| | | 6 feet Crest | 12 feet Crest |
| Static | 1.76 | 1.51 | 1.44 |
| Seismic | 0.15 | 0.17 | 0.16 |
| Rapid Drawdown | 1.62 | 1.36 | 1.29 |

Notes:
* The above FS values are rounded to the nearest 0.01 decimal point for comparative purposes.

Based on the configuration noted in Section 4.3, raising the existing levee will cause a significant reduction in stability under static and rapid drawdown cases. The stability under seismic conditions is very low and raising the levee will not meaningfully affect it. The stability of the levee only meets the USACE recommendations under the static case with, not the rapid drawdown case for both crest widths. Under existing conditions, the levee is estimated to be stable except during a significant seismic event. Under the proposed raising of the levee, it will not meet typical design FS for rapid drawdown or seismic (1.4 static/drawdown, 1.1 seismic - USACE 2000). It is noted that these are estimates of the stability of the levee. It is significant that the proposed levee raise reduces the FS by about 25 percent for the 6 feet wide crest and over 30 percent for the 12 wide crest. These are significant reductions which may result in unstable conditions.

### 4.3.3 Seepage

We completed seepage analysis of the proposed reconstructed levee and found that exit gradients are well below 0.5 feet per hour (fph), the maximum recommended by the USACE (2000). This is true under both existing and proposed conditions.

## 5.0 CONCLUSIONS

Based on our explorations, testing, and analyses, it is our opinion that the site is suitable for the project, provided the recommendations in this report are included in design and construction, and that the limitations noted are considered and accepted. We generally conclude the following regarding the site:

- The site is underlain by very soft soils to about 40 feet deep on the north side of Noble Creek (bridge location), deepening to over 60 feet on the south side (levee and culvert locations).
- Soils below the groundwater table to about 20 to 25 feet bgs are anticipated to undergo liquefaction or cyclic strain softening during the design earthquake. Liquefaction and cyclic strain softening are expected to result in moderate seismic settlements and lateral spreading as noted in this report.
- We recommend the proposed bridge be supported on driven pile foundations bearing in Tyee Formation bedrock, using the capacities presented in this report. Downdrag loads from liquefaction/strain softening should be accounted for in the extreme limit state, but not the service state.
- Portland cement concrete walls supported on bridge piling are recommended to retain approach fills.
- New fill will cause settlement of site soils, at all locations: the bridge approaches, temporary road fill, and levee alignment. Fill placement should be limited as practicable, and the magnitudes of

settlement provided in this report from the various fills at the site should be considered in design and construction. If fill configurations change materially from those described herein, we should be contacted to re-evaluate settlements once final fill configurations are known.

- Levee reconstruction will decrease the stability of the levee by 25 to over 30 percent in the static and rapid drawdown cases. Recommended design FS will not be met in the rapid drawdown and seismic cases.
- Earthwork should generally comply with USACE manual EM 1110-2-1913 (USACOE, 2000) standards. Construction materials shall meet the USACE (2000) requirements and the Oregon Standard Specifications for Construction (OSS - latest edition), as specified in this report.

The following sections of this report provide our recommendations for geotechnical design and earthworks for the project.

# 6.0 DESIGN RECOMMENDATIONS

Per Section 5.0, we recommend the bridge and at-grade walls be supported on deep foundations. We understand that H-pile sections are preferred. The tide gates can be supported on similar or other piling types, driven to similar depths. The two culverts fitted with tide gates can be supported at grade.

Our recommendations for pile foundations to support the bridge and abutment walls are provided in the following sections. Other piling types can be evaluated if requested.

## 6.1 DEEP FOUNDATIONS

The following sections provide our geotechnical recommendations for design and construction of deep foundations to support the new bridge and abutment walls. We have developed these recommendations based on our current understanding of the project, the more conservative subsurface conditions encountered in Borings B-1 and B-2, and preferred pile types of HP 12x53 or 14x89. We recommend the piles be fitted with driving shoes to accommodate the minimum embedment requirements into sedimentary bedrock. For design, we assumed the piles will be embedded a minimum of 5 feet into medium soft (R2) to medium hard (R3) bedrock, beginning at an estimated 40 feet bgs (driving tip at 45 feet bgs).

### *6.1.1 Nominal Pile Resistance*

Nominal single pile resistance was calculated using FHWA methods outlined in the ODOT GDM (ODOT 2018). Based on embedment into Bedrock as noted above, we recommend nominal resistance of 550 and 650 kips combined side- and end-bearing for HP 12x53 and 14x89, respectively, in compression under static conditions. This is the geotechnical capacity of the piles and does not consider the structural capacity of the piles, which should be determined by the structural engineer.

In tension, we calculated nominal resistance under both static and seismic conditions. Under static conditions, tension capacity will come from friction/adhesion between the piles and all soils on the sides of the piles. We calculated tension resistance for the full pile lengths (45 feet) using estimated soil strengths in the bedrock, alluvium and fill portions of the soil profile which resulted in nominal resistances of 150 and175 kips for HP 12x53 and 14x89, respectively, in tension under static conditions.

In calculating the nominal tension capacity under liquefied soil conditions, we assumed seismically induced soil strength loss in the potentially liquefied/loss of strength zone between 5 feet 25 feet bgs, using strength reductions based on Figure 6.7 of the ODOT GDM (ODOT, 2023). Using the reduced soil strengths in this interval, but static strengths above and below this zone, we estimate reduced nominal tension capacities of 110 and 140 kips for HP 12x53 and 14x89 piles, respectively, during seismic conditions.

We calculated downdrag forces on the piles during static and seismic conditions. If no new fill is placed near the bridge or enough time is allowed to pass for primary consolidation of underlying soft soils, no downdrag need be applied. If fill is placed and settlement not complete prior to driving the piles, then downdrag loads will be imposed on the piles from settlement of soils within the upper approximately 35 feet beneath site. Such downdrag loads should be applied to the pile compression capacity and equate to 60 kips and 70 kips, respectively for HP 12x53 and 14x89, respectively. Downdrag from liquefied soils is minimal, approximately 5 kips, and can be ignored in the calculations.

### *6.1.2 Pile Resistance Factors*

Resistance factors used to calculate factored resistances were determined in accordance with AASHTO Section 10.5.5 (AASHTO, 2020) and are displayed in Table 4.

**Table 4 – Resistance Factors**

| Limit State | Compression | Tension |
|---|---|---|
| Service | 1 | 1 |
| Strength | .30 | .25 |
| Extreme | 1.0 | 0.8 |

Our estimated pile resistances are based on our interpretation of soil properties deduced from our borings. Subsurface conditions below this depth are based on extrapolation of the material encountered at the bottom of the borings, which is a highly likely condition.

The structural engineer should verify that the pile is designed to structurally support the anticipated design loads. Driving stresses may control and pile sizes should be considered in structural design. Pile group effects should be accounted for in accordance with Sections 10.7.3.9 and 10.7.3.11 of the AASHTO BDS (2020) or similar guidance.

Pile installation recommendations are provided in Section 6.1.5.

### *6.1.3 Lateral Pile Resistance*

Lateral resistances and deflections of vertical pile foundations are often governed by the lateral resistance of near-surface soils and the strength of the pile itself. The design lateral resistance of the vertical piles will depend, to a large extent, on the allowable lateral deflections of the piles. Note that lateral resistance of the soils will be reduced during seismic and post-seismic loading due to soil liquefaction or cyclic strength loss. The reduced lateral resistance of the alluvium in the liquefiable zone was modeled based on recommendations for Soft Clay Criteria in the ODOT GDM.

Lateral resistance of the piles should be in accordance with Section 10.7.3.12 of AASHTO (2020). We recommend that P-y curves be generated using the software program LPile developed by Ensoft, Inc., using the model parameters for the soils outlined in Table 5. Assume that the design groundwater depth is approximately 5 feet bgs from the road surface (or at surface level adjacent the roadway embankments). The parameters are given from Boring B-1, which is the worst-case condition. Conditions at Boring B-2 include stiffer soils, so will result in greater lateral resistance and reduced deflection compared to Boring B-1.

**Table 5 – LPile Parameters**

| Soil | Depths of Occurrence (feet bgs @ B-1) | Total Unit Weight (pcf) | Effective Unit Weight (pcf) | P-Y Curve Model | Soil Modulus (pci) | Friction Angle (degrees) | Undrained Shear Strength (psf) | Strain (e50) |
|---|---|---|---|---|---|---|---|---|
| Service Limit State Parameters (Static) | | | | | | | | |
| Fill | 0 to -5 | 125 | 63 | Clay (Matlock) | NA | NA | 1000 | 0.010 |
| Alluvium | -5 to -35 | 85 | 23 | Clay (Matlock) | NA | NA | 250 | 0.020 |
| Residual Soil | -35 to -40 | 105 | 43 | Clay (Matlock) | 500 | NA | 1000 | 0.007 |
| Tyee Wx & Bedrock | -40 to >-51.5 | 115 | 53 | Clay (Matlock) | 2000 | NA | 5,000 | 0.004 |
| Extreme Limit State Parameters (Seismic) | | | | | | | | |
| Fill | 0 to -5 | 125 | 63 | Clay (Matlock) | NA | NA | 1000 | 0.010 |
| Alluvium | -7 to -35 | 85 | 23 | Clay (Matlock) | NA | NA | 25 | 0.020 |
| Residual Soil | -35 to -40 | 105 | 43 | Clay (Matlock) | 200 | NA | 1000 | 0.010 |
| Tyee Wx & Bedrock | -100 to >-51.5 | 115 | 53 | Clay (Matlock) | 800 | NA | 5,000 | 0.004 |

Notes:
a. pci = pounds per cubic inch
b. Reduced strengths estimated per Method 1 in the ODOT GDM (2018), Section 6.5.5.2.

Note that the parameters provided herein assume that conditions near the surface are not significantly changed from their current conditions by construction. The foundation designer should account for any differences in elevations and surface conditions between those at our explorations and the final pile locations.

Pile group effects under lateral loading should be accounted for in accordance with Section 10.7.2.4 of the AASHTO BDS. Appropriate P-multipliers will need to be applied to P-y curves based on pile group geometry and direction of applied loads.

#### 6.1.4 Pile Settlement

We estimate pile settlement will be up to approximately 1 inch under design dead and live loads. We note that the ground surface around the bridge is expected to settle up to about 1 foot due to liquefaction, though the piles (and structure) will generally resist liquefaction-induced settlement.

#### 6.1.5 Extreme Limit State – Lateral Forces

The piles supporting the bridge will be subjected to lateral earth forces due to creek bank instability in the extreme limit state following the onset of liquefaction. Case histories of deep foundations installed in laterally spreading soils have shown that the lateral loads induced onto the foundations come from two sources: 1) the non-liquefied crust riding over the liquefied soil and 2) loads from lateral movement of the liquefied soils. Typically, the loads from the movement of the non-liquefied crust are significantly larger than those from the movement of the liquefied soil.

Per the force-based approach outlined in ODOT GDM Section 6.5.5.5, piles at the abutments should be designed to resist loads caused by seismically induced slope movement. This section states that non-liquefied crustal layers exert the full passive earth pressure on the foundation system, and liquefied soils exert a pressure equal to 30 percent of the total overburden pressure. The groundwater level at the site is expected to be relatively stable due to the fine-grained (low permeability) upper soils and proximity to the Noble Creek, so a groundwater level of 5 feet bgs is recommended. As such, we recommend using a non-buoyant passive equivalent fluid unit weight for the soils of 350 pounds per cubic foot for the upper 5 feet of fill. This load should be applied to the abutment wall from the ground surface to the bottom of the pile cap or 5 feet bgs, whichever is deeper. Below 5 feet bgs to the bottom of the creek (about 16 feet bgs) the soils would be liquefied, so 30 percent of the total overburden pressure should be applied (an earth pressure coefficient of 0.3 applied to the total vertical stress). Liquefiable soils are found below the creek channel, but are confined across the creek channel, so do not transmit lateral forces.

#### 6.1.6 Pile Driving

Axial pile resistance should be verified at end of driving conditions. Resistances should be verified per Section 8.12.2 in the ODOT GDM. Pile resistances should be calculated using the FHWA Gates Equation in accordance with the ODOT GDM and OSS Section 00520.42. Due to the downdrag loads affecting the piles, the driving resistances for pile installation will be greater than the capacity of the piles (for design, the downdrag zone is ignored for capacity, but during installation a non-liquefied skin friction resists the pile driving). We should be consulted once pile lengths and capacities are determined in order to verify the actual driving resistances required.

#### 6.1.7 Corrosivity

We recommend the site be considered a "Coastal Area", per Section 1.2.4.2 of the ODOT Bridge Design Manual, and appropriate corrosion protection utilized.

## 6.2 Abutment Retaining Wall Recommendations

PCC retaining walls supported on the bridge piling are proposed to retain approach fills. The following report sections provide our recommendations for PCC wall design.

### 6.2.1 General

We recommend that the bridge abutments be retained by PCC walls supported by the bridge piling. Resistance factors should be per Table 6, below.

**Table 6 - Resistance Factors for Shallow Foundations**

| Limit State | Sliding | Passive Earth Pressure of Sliding Resistance |
|---|---|---|
| Service | 1.00 | 1.00 |
| Strength | 0.80 | 0.50 |
| Extreme | 0.90 | 1.00 |

The following sections provide geotechnical parameters for design of such walls.

### 6.2.2 Earth Pressures

PCC concrete walls should be designed to resist the earth pressures shown in Table 7. The designer should also include hydrostatic forces per Note C, as appropriate.

**Table 7 – Abutment Wall Earth Pressures**

| Material | Condition | Horizontal Earth Pressure Coefficient | Equivalent Fluid Pressure |
|---|---|---|---|
| Wall Backfill (Static Loading) | Active | $K_a = 0.26$ | 15 pcf below gwt[c] |
| | | | 30 pcf above gwt |
| | At-Rest | $K_o = 0.44$ | 26 pcf below gwt[c] |
| | | | 52 pcf above gwt |
| Wall Backfill Seismic Surcharge | Active | | 6H psf[d] |

Notes:

a. Wall backfill should meet the specifications provided in OSSC 00510.12 – Granular Wall Backfill. For wall backfill a compacted unit weight of 125 pcf and a friction angle of 34 degrees were assumed. Drainage of soils above the water table was assumed and should be provided.

b. gwt = groundwater table.

c. For materials located below the groundwater table, an additional hydrostatic pressure of 62.4 pcf should be added. Hydrostatic forces should also be applied as appropriate.
d. For the seismic conditions, the nominal seismic lateral earth pressure should be applied as a uniform rectangular distribution and should only extend to the base of abutment walls (AASHTO 2020).

The earth pressures in Table 7 assume level, granular backfill behind the walls. For walls that can move at least 0.1 percent of their height (i.e., yielding wall), the design should use active pressures. Where walls are restrained from moving this distance, they are considered "non-yielding" and should be designed with at-rest pressures.

Where traffic loads are located within a horizontal distance from the top of the wall equal to one-half the wall height, the lateral earth pressure shall be increased by a surcharge load equal to 2 feet of soil (assuming a soil density of 125 pcf).

The walls should be designed to resist surcharge loads from adjacent footings, equipment, material storage, and vehicular loads placed within a 1 horizontal to 1 vertical (1H:1V) projection from the base of wall.

The wall designer should apply appropriate load factors to the earth pressures provided in Table 7. If other surcharge loads (e.g., footings, stored construction goods, etc.) will be placed within 10 feet of the backs of the walls, then we should be contacted for additional information regarding how such loads are applied to the retaining walls.

#### 6.2.3 Lateral Resistance

We assume that most lateral resistance against earth pressures on the walls will come from the piles and bridge structure. However, where the walls are in contact with soil, we recommend using an unfactored sliding coefficient of 0.65 for concrete against soil. The sliding coefficient of friction was calculated based on AASHTO LRFD 10.6.3.4, which is tan(φ). The passive earth pressure coefficient (Kp) to be used for lateral resistance of shallow foundations is 3.55. If soft or loose soil is observed directly below the bottom of foundations during construction, the soft or loose soil should be excavated and replaced with compacted structural fill.

We recommend a nominal lateral resistance from passive earth pressures using an equivalent fluid pressure of 445 pcf.

Resistance factors should be per Table 6 and AASHTO requirements.

### 6.3 Culvert Recommendations

Culverts with tide-gate systems supported on the culverts can be installed at grade as recommended below. We recommend bearing pressures not exceed 1,500 pounds per square foot (psf) locally and 500 psf average across the structure footprint. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and may be increased by one-third for short-term loads.

We recommend a layer of rock be placed over the subgrade of the tide gate portion of the excavation to limit disturbance prior to placing the tide gate structure on grade. If placed in the dry, the rock should consist of a 12-inch layer of pit or quarry run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSS 00330.14 – *Selected Granular Backfill* or OSS 00330.15 – *Selected Stone Backfill*. The imported granular material should also be angular, fairly well graded between coarse and fine material, have less than 5 percent by dry weight passing the U.S. Standard No.

200 Sieve, and have at least two mechanically fractured faces. If placed in the wet, a 12 to 18-inch layer of imported granular material should be placed as a subgrade stabilization layer. The stabilization layer should meet the specifications of OSS 00330.14 Selected Granular Backfill, or OSS 0330.15 Selected Stone Backfill. The rock should be compacted to a dense well-keyed condition with appropriate equipment, but not to cause damage or softening of the subgrade.

A layer of rock should not be placed along the full length of the culvert, as it could facilitate seepage and piping through the culvert backfill. However, soft spots can be excavated and backfilled with stabilization material, as noted above, to support the culvert locally.

A construction geosynthetic should be placed on the undisturbed subgrade at the tidegate and areas of overexcavation prior to placing stabilization, to prevent migration of fines into the void spaces of the material. The geosynthetic should meet the specifications of Table 02320-4 of OSS 02320 for soil stabilization and installed in conformance with the specifications provided in OSS 00350 – Geosynthetic Installation.

Provided the structure is placed as recommended it should experience “static” settlement of less than 2 inches, with differential settlement of less than 1 inch over the culvert’s span. This assumes the roadway/levee grades at new culvert/tide gate structures will not be raised.

Lateral loads can be resisted per Section 6.2.3.

### 6.4 Embankment Design

The USACE (2000) recommends a 12’ minimum crest width, however, we primarily evaluated a 6’ wide crest to minimize earthwork volumes. Our design otherwise follows typical stable gradients for dry and submerged slopes as noted in Section 4.3. The levee raise should follow the recommendations in Section 7.7, including stripping, benching & keying, and compaction. Although our design was based on a 6 feet wide crest, a 12 feet wide crest is recommended by USACE (2000) and would also be our recommended width, if the levee is raised.

The USACE (2000) recommends a 6-feet-deep foundation trench excavated along the center of a new levee for its complete length to confirm that shallow subsurface conditions are suitable for construction of the levee and remove shallow deleterious materials. Excavation of a key per section 7.7 can be used to achieve a similar purpose and need not be carried to 6 feet deep. Since the project includes raising the existing levee, it is assumed that the existing levee is situated on a suitable mineral soil surface and organics and deleterious soils were removed from the subgrade at the time it was constructed.

## 7.0 EARTHWORK RECOMMENDATIONS

All earthwork should be conducted in accordance with the Oregon Standard Specifications for Construction (OSS) (ODOT 2021). Earthworks related to raising the levee should be conducted per the recommendations of the USACE (2000). Specific recommendations for earthworks are provided in the following sections.

### 7.1 Demolition

Demolition should include complete removal of existing site improvements within the work area. Materials generated during demolition of existing improvements should be transported off site for disposal or stockpiled in areas designated by the owner. In general, these materials will not be suitable for reuse as engineered fill. However, asphalt, concrete, and base rock materials may be crushed and recycled for use as general fill. Such recycled materials should meet the specifications for imported granular material, as described in Section 7.5.

## 7.2 EXCAVATION

### 7.2.1 Mass Excavation

Site soils within expected excavation depths generally consist of silt and clay with lesser sand, and road gravel. It is our opinion that conventional earthmoving equipment in proper working condition should be capable of making necessary general excavations for the project.

The earthwork contractor should be responsible for providing equipment and following procedures as needed to excavate the site soils, as described in this report, while protecting the subgrade.

### 7.2.2 Temporary Open Cuts

Temporary soil cuts for site excavations that are more than 4 feet deep should be adequately sloped back to prevent sloughing and collapse, in accordance with Occupational Safety and Health Administration (OSHA) guidelines.

The stability and safety of cut slopes depend on a number of factors, including:

- Type and density of the soil;
- Presence and amount of groundwater seepage;
- Depth of cut;
- Proximity and magnitude of the cut to any surcharge loads, such as stockpiled material, traffic loads, or structures;
- Duration of the open excavation; and
- Care and methods used by the contractor.

Because of the variables involved, actual slope angles required for stability in temporary cut areas can only be estimated before construction. It is the responsibility of the contractor to ensure that the excavation is properly sloped or braced for worker protection, in accordance with OSHA guidelines. Most of the site soils anticipated within excavations should generally consist of soft/loose silt, clay, and silty sand. We recommend all soils be classified as OSHA Class C for excavation purposes.

If site constraints do not allow proper excavation slopes for trenching, shoring may be used to support trench excavations. Shoring selection and design should be the responsibility of the contractor. If shored, excavations are left open for extended periods of time, caving of the sidewalls may occur between the cut and shoring if voids between the shoring and cut are not filled. The presence of caved material will limit the ability to properly backfill cuts. The voids between box shoring and the sidewalls of cuts should be properly filled with sand or gravel before caving occurs. It should be the contractor's responsibility to employ trenching, excavation, and shoring methods that ensure proper compaction will be achieved and protect adjacent facilities.

### 7.2.3 Permanent Slopes

Permanent cut and fill slopes should not exceed grades of 2 horizontal to 1 vertical (2H:1V) where not submerged. Slopes subject to intermittent or permanent submersion should not exceed 3H:1V. Where rip rap is placed at the abutments for erosion protection, we anticipate slopes of 2H:1V will be stable, however, this should be verified during construction when the slopes are opened up and soil conditions can be confirmed locally.

#### 7.2.4 Riprap Slope Protection

Riprap slope protection should be placed to protect the abutment walls, approach fills, and culvert inlets and outfalls. The riprap should be placed on slopes per Section 7.2.3 above, that have been prepared per Section 7.4 of this report, and in accordance with OSS 00390 - Riprap Protection. The toe of the riprap should extend below scour depth as determined by others. Riprap protection should include a geotextile fabric, overlain by filter material, overlain by the riprap per the OSS and Sections 7.5.5 and 7.5.6 of this report.

### 7.3 Wet Soil Construction and Dewatering

Site soils will be extremely sensitive to moisture and susceptible to disturbance during periods of wet weather. We recommend earthworks be completed during the dry season/dry conditions. If construction occurs during the wet season, wet soil construction practices should be implemented. Wet soil construction practices include using equipment, such as smooth excavator buckets and tracked equipment, to limit subgrade disturbance. Some operations may be next to impossible to perform during the wet season and/or during wet conditions.

We assume Noble Creek will be controlled upstream of the bridge by a temporary tidegate or bypass the construction site, which will reduce maximum water levels/flooding. However, groundwater will likely still be present at or near working depths during construction. Dewatering excavations are the responsibility of the contractor. The contractor may be able to control groundwater using sumps and pumps due to the mostly fine-grained nature of site soils. Excessive groundwater inflows could occur, however, if sand facies extend into the excavations and due to the proximity of the excavations to the creek. If inflow is severe, it will likely not be controlled with sumps and pumps but will require other measures.

If foundation excavations cannot be adequately dewatered resulting in subgrade disturbance and softening, a layer of stabilization rock may be required. Stabilization rock should meet the specifications of Section 7.5. The stabilization layer is placed by over-excavating below subgrade elevation about 2 feet (or as needed) and backfilling with the stabilization rock. The rock is compacted by tamping with an excavator bucket until well-keyed. We recommend a minimum 3-inch layer of compacted aggregate base backfill, conforming to the specifications of Section 7.5, be placed over the stabilization layer to act as a leveling pad.

### 7.4 Subgrade Preparation

Excavations for roadways, structural fill and structures supported at grade should expose undisturbed native soils free from organics and debris. Once excavations are completed to grade, the subgrade should be probed or proofrolled, depending on access and soil moisture, to determine that suitable soils are present at subgrade elevation. If soft/loose soils are present at subgrade elevation, they should be overexcavated to the depth they occur and replaced with crushed rock structural fill per Section 7.5 or as directed by Pali Consulting. If water infiltrates and pools in excavations, the water should be removed before placing structural fill or elements supported at grade.

We recommend that Pali Consulting observe all subgrades before placement of structural fill or elements supported at grade to determine whether bearing surfaces have been adequately prepared and the soil conditions are consistent with those observed during our explorations.

## 7.5 Structural Fill and Backfill

Structural fill should include fills intended to support structures and roadways (including embankment slopes) and fill within the influence zone of structures. Fill should only be placed over a subgrade that has been prepared in conformance with the prior sections of this report. A variety of material may be used as structural fill. However, all material used as structural fill should be free of debris, clay balls, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 4 inches, and other deleterious materials and should meet the appropriate specification provided in OSS 00330.12 – Borrow Material, 00330.13 – Selected General Backfill, or 00330.14 – Selected Granular Backfill.

Fill and backfill materials should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities as recommended in Tables 7 and 8 that follow.

See Section 7.7 for Levee Earthworks Recommendations.

### *7.5.1 On-Site Soils*

We expect that minimal to no borrow of on-site soils will occur for bridge and culvert construction. We note that most soils within likely excavations depths are clays and silts with PI's greater than 20, so not suitable for use as structural fill. Levee fill is discussed separately in Section 7.7. We recommend only the base rock/processed AC portions of the embankments be reused as structural fill, provided it meets the other recommendations in this report.

### *7.5.2 Imported Select Structural Fill*

Imported granular material used as structural fill or during periods of wet weather should be pit or quarry run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSS 00330.14 – Selected Granular Backfill, 00330.15 – Selected Stone Backfill, or 00330.16 – Selected Stone Embankment. The imported granular material should also be angular, fairly well graded between coarse and fine material, have less than 5 percent by dry weight passing the US Standard No. 200 Sieve, and have at least two mechanically fractured faces. During dry weather, the fill may have up to 12 percent fines content.

### *7.5.3 Aggregate Base/Base Rock*

Imported granular material used as aggregate base (base rock) beneath pavements or structures should be clean, crushed rock or crushed gravel and sand that is fairly well graded between coarse and fine. The base aggregate should meet the specifications provided in OSS 02630.10 – Dense Graded Base Aggregate, depending upon application, with the exception that the aggregate has less than 5 percent by dry weight passing a US Standard No. 200 Sieve and at least two mechanically fractured faces. The aggregate base should have a maximum particle size of 1.5 inches.

### *7.5.4 Stabilization Material*

We recommend that imported granular material used to create haul roads for construction traffic or to stabilize culvert or other subgrades consist of pit or quarry run rock, or crushed rock. The material should generally be sized between 2 and 6 inches, have less than 5 percent by dry weight passing the US Standard No. 4 Sieve, and have at least two mechanically fractured faces. Material meeting the specifications of OSS 00330.15 – Stone Backfill Material is acceptable for use, excepting recycled glass shall not be used.

Stabilization material should be placed in lifts 12 inches thick and be compacted to a well-keyed condition with appropriate compaction equipment without using vibratory action.

#### *7.5.5 Rip Rap and Filter Blanket*

Riprap for erosion protection should meet the specifications provided in OSS 00390 – Riprap Protection. The rock shall consist of imported rock meeting the specifications of OSS 00390.11 – Riprap Requirements and the gradation requirements of OSS 0039.11(c) based on flow velocities in Noble Creek, as determined by others. The riprap shall be placed on filter blanket material meeting the specifications of OSS 003913 – Filter Blanket. A geotextile fabric should be placed over the subgrade prior to placing the rock per the OSS and Section 7.5.6 of this report.

#### *7.5.6 Geotextile Material*

A geotextile fabric should be used beneath stabilization rock and in conjunction with streambank riprap protection. Geotextile materials may be used elsewhere if soft subgrade conditions are encountered. Geotextiles should meet the requirements of Tables 2320-2 (Type 2) or 2320-4, depending on application Geotextiles shall be installed per OSS Section 00350.

### 7.6 FILL PLACEMENT AND COMPACTION

Structural fill should be placed and compacted in accordance with *OSS 00330.43 – Earthwork Compaction* requirements and the following guidelines.

- Place fill and backfill on a prepared subgrade that consists of firm, inorganic native soils or approved structural fill.
- Place fill or backfill in uniform horizontal lifts with a thickness appropriate for the material type and compaction equipment. Table 8 provides general guidance for uncompacted lift thicknesses.

**Table 8 – Guidelines for Uncompacted Lift Thickness**

| Compaction Equipment | Guidelines for Uncompacted Lift Thickness (inches) | | |
|---|---|---|---|
| | On-site Soils | Granular and Crushed Rock Maximum Particle Size $\leq$ 1½ inch | Crushed Rock Maximum Particle Size > 1½ inch |
| Plate Compactors and Jumping Jacks | 4 – 8 | 4 – 8 | Not Recommended |
| Rubber-Tire Equipment | 6 – 8 | 8 – 12 | 6 – 8 |
| Light Roller | 8 – 10 | 8 – 12 | 8 – 10 |
| Heavy Roller | 10 – 12 | 12 – 18 | 12 – 16 |
| Hoe Pack Equipment | 12 – 16 | 18 – 24 | 12 – 16 |

Note:

The above table is based on our experience and is intended to serve as a guideline. The information provided in this table should not be included in the project specifications.

- Do not place fill and backfill until the required tests and evaluation of the underlying materials have been made and the appropriate approvals have been obtained.
- Limit the maximum particle size within the fill to two-thirds of the loose lift thickness.
- Control the moisture content of the fill to within 3 percent of the optimum moisture content based on laboratory Proctor tests. The optimum moisture content corresponds to the maximum attainable Proctor dry density.
- Complete a sufficient number of in-place density tests to verify that the specified degree of compaction is being achieved.
- Do not damage or displace underground utilities or adjacent facilities during backfilling and compaction.
- Grade the surface of the fill at the end of each working shift so that surface water can drain readily.
- Compact fill soils to the percentages of maximum dry density as shown in Table 9.

**Table 9. Fill Compaction Criteria**

| Fill Type | Percent of Maximum Dry Density Determined in Accordance with ASTM D1557 | | |
|---|---|---|---|
| | 0 – 2 Feet Below Subgrade | >2 Feet Below Subgrade | Pipe Bedding and Pipe Zone |
| Mass Fill (imported granular materials) | 95 | 92 | ------ |
| Aggregate Bases | 95 | 95 | ------ |
| Trench Backfill | 95 | 92 | 90 |
| Nonstructural Trench Backfill | 88 | 88 | ------ |
| Nonstructural Zones | 88 | 88 | 90 |

Notes:

1. Structural fill with more than 30 percent retained on the ¾-inch sieve should be compacted to a well-keyed dense state within 3 percent of optimum moisture content. Compaction should be verified by Pali Consulting through performance testing, such as a proofroll.
2. Within 3 feet of the back of retaining walls, compact to a lower density of 92 percent to limit potential wall damage from high horizontal stresses.

During structural fill placement and compaction, a sufficient number of in-place density tests should be completed to verify that the specified compaction is being achieved.

## 7.7 Earthworks Recommendations for Levee Raising

### *7.7.1 Subgrade Preparation*

The subgrade within the footprint of all fill placed to raise the levee should be prepared by clearing, grubbing, and stripping; by excavation of benches on surfaces to receive fill with a gradient greater than 5H:1V; and by excavation of a toe keyway.

Clearing, grubbing and stripping should consist of the following:

- The removal of all objectional matter above the ground surface, including logs, fallen trees, and brush.
- The removal of stumps, shrubs, and their roots, including roots and organics up to 1.5 inches in diameter within 3 feet of the subgrade.
- The removal of non-native subsurface materials such as drainage tile and/or piping
- Stripping of low-growing grasses and other organic matter to mineral soils. We anticipate a typical stripping depth of about 6 to 12 inches bgs. Stripping should be evaluated by Pali Consulting at the time of construction, and strippings will not be suitable for structural fill but should be wasted in non-structural areas or hauled off site.

Once all organics and deleterious materials are removed, sloped surfaces to receive fills that are at or steeper than 5H:1V should be benched to provide flat surfaces on which to place and key in new fill. The benches should preferably the width of the equipment excavating them and not less than 4 feet wide. They should be level or slope downward to drain at up to 5 percent gradient.

A key should be cut into the toe of the new fill on the dry side of the levee. The key should be at least 2 feet deep and 4 feet wide, or the minimum width needed to accept suitable excavation and compaction equipment to construct levee fill materials, whichever is wider.

After preparation is complete, embankment subgrades should be evaluated by Pali Consulting to identify areas of soft or loose subgrade soils, and the toe key should be evaluated to confirm no deleterious materials are present. The subgrade evaluation should be completed prior to placement of fill. If soft or loose zones are identified during subgrade evaluation, these areas should be excavated and replaced with structural fill meeting the requirements for structural levee fill material. Significant voids due to subgrade preparation should be graded back and backfilled with levee fill material and compacted per Table 10.

### 7.7.2 Levee Fill Material

Material used for raising the levee shall be fine grained material placed and compacted per these specifications. The material shall consist of silt or clay with lesser sand or gravel. The materials shall meet the specifications provided in OSS 00330.12 – Borrow Material and OSS 00330.42 – Embankment, Fills, and Backfills, and be classified as ML, CL, or CL-ML per the Unified Soil Classification System (USCS), and have a plasticity index (PI) of less than 20. We note that these soils will be very sensitive to moisture and may require moisture conditioning before they can be used. They should be placed and compacted at a moisture content between approximately +5% to -3% of optimum and compacted per the recommendations in following sections of this report. This material should not be placed, compacted, or worked during wet weather.

### 7.7.3 Levee Fill Placement and Compaction

Structural fill should be placed and compacted in accordance with OSS (latest edition) and USACE (2000) requirements. We recommend that compaction of all levee fill be completed as a Category II material per the USACE (2000). Placement and compaction should meet the recommendations provided in Section 7.6 except as modified by Table 10.

Table 10 – Guidelines for Category II Fill Construction

| Uncompacted Lift Thickness (inches) | Percent maximum dry density per AASHTO T-180 / ASTM D 1557 |
|---|---|
| 8 - 12 | 88 |

Notes:
1. The above table is based on the recommendation by the USACOE (2000) and our experience and is intended to serve as a guideline. The information provided in this table should not be included in the project specifications.

During placement and compaction of levee fill, a sufficient number of in-place density tests should be completed by Pali Consulting to verify that the specified degree of compaction is being achieved.

## 8.0 CONSTRUCTION OBSERVATIONS

Satisfactory foundation and earthwork performance depend to a large degree on quality of construction. Sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during subsurface explorations. Recognition of changed conditions often requires experience; therefore, Pali Consulting or their representative should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

We recommend that Pali Consulting be retained to monitor construction at the site to confirm that subsurface conditions are consistent with the site explorations, and the intent of project plans and specifications relating to earthwork and foundation construction are being met. We recommend that pile installation, roadway and structural fill subgrades, structure excavations, and compaction of structural fill/backfill be observed and/or tested by Pali Consulting or their representative.

# 9.0 LIMITATIONS

We have prepared this geotechnical report for use by Waterways Consulting, Inc., Coos Soil and Water Conservation District, and their consultants for the proposed improvements at Noble Creek, in Coos County, Oregon. Our work was completed in general accordance with our Service Agreement with Waterways Consulting, Inc, dated April 11, 2023. Our report is intended to evaluate geologic hazards and provide geotechnical recommendations for design of the project. However, all geotechnical design involves risks, only part of which can be mitigated through geotechnical engineering and construction practices. Favorable performance in the near term does not imply a certainty of long-term performance, especially under conditions of adverse weather or seismic activity.

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, 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 Pali Consulting and will serve as the official document of record.

# 10.0 REFERENCES

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## 11.0 CLOSING

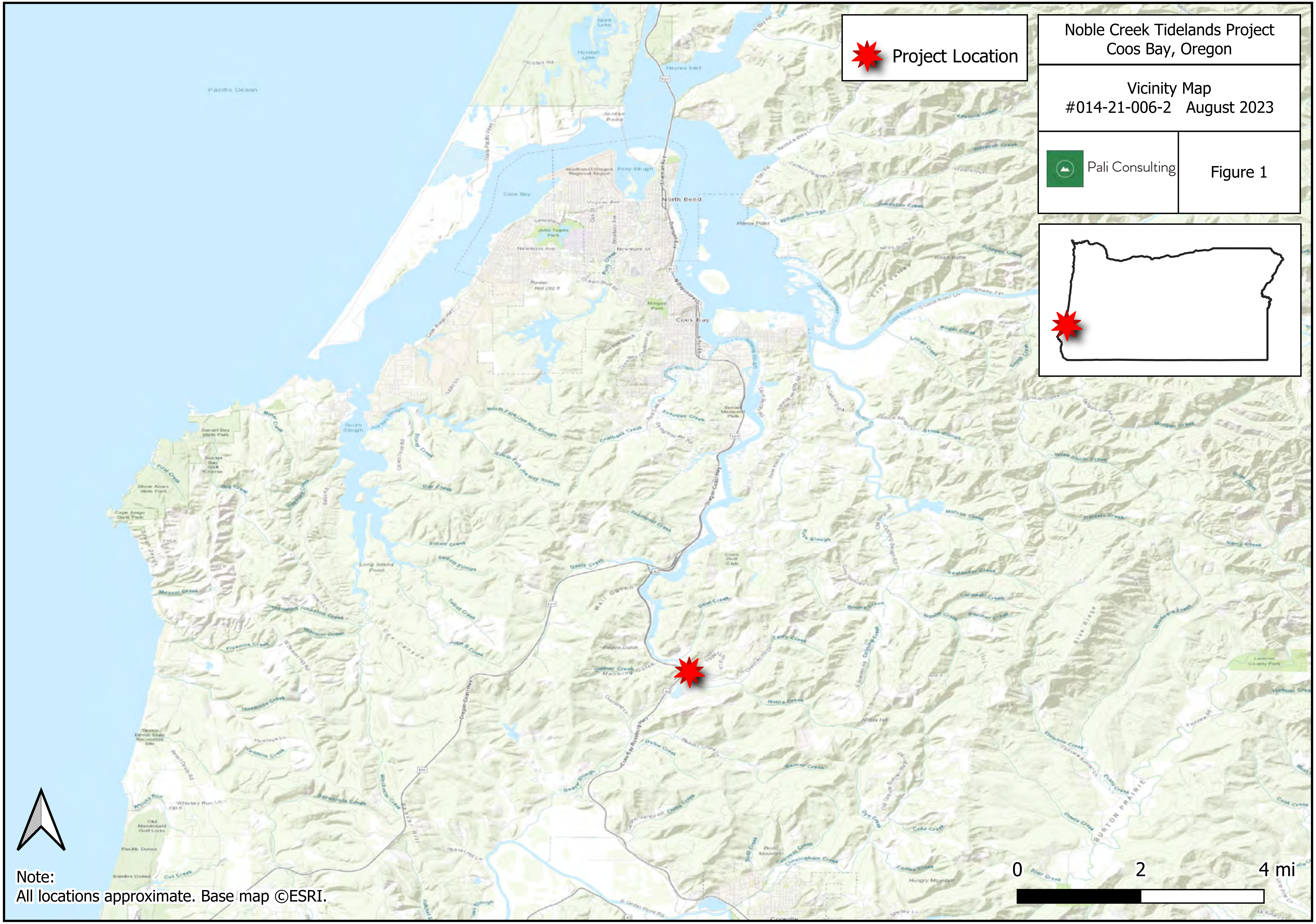
We appreciate the opportunity to submit this report for your project. Please contact us if you have any questions or need additional information.

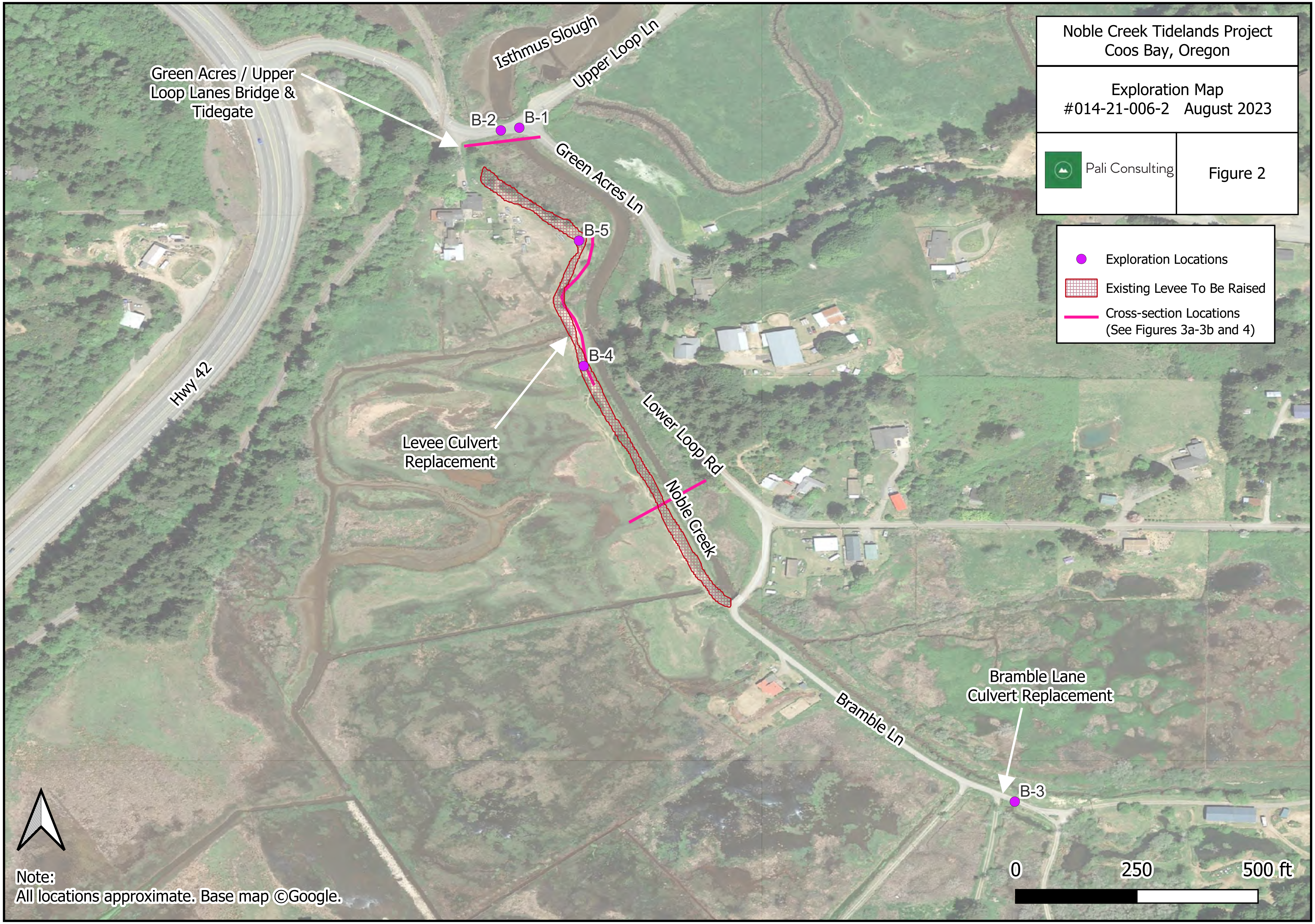
Sincerely,


Timothy W. Blackwood, PE, GE, CEG
President/Principal Engineer

Attachments
- Figures 1 – 4
- Appendices A-C

Document ID: 014-21-006-02NobleCrkGeotechRpt


Noble Creek Tidelands Project
Coos Bay, Oregon
Vicinity Map
#014-21-006-2 August 2023
Pali Consulting
Figure 1
Project Location
0
2
4 mi
Note:
All locations approximate. Base map ©ESRI.
Pacific Ocean
North Bend
Coos Bay


Noble Creek Tidelands Project
Coos Bay, Oregon
Exploration Map
#014-21-006-2 August 2023
Pali Consulting
Figure 2
Exploration Locations
Existing Levee To Be Raised
Cross-section Locations
(See Figures 3a-3b and 4)
Isthmus Slough
Upper Loop Ln
Green Acres / Upper Loop Lanes Bridge & Tidegate
B-2
B-1
Green Acres Ln
B-5
B-4
Hwy 42
Levee Culvert Replacement
Lower Loop Rd
Noble Creek
Bramble Ln
Bramble Lane Culvert Replacement
B-3
0
250
500 ft
Note:
All locations approximate. Base map ©Google.

Pali Consulting

Figure 3a

Notes: Cross sections based on LiDAR data provided by DOGAMI. All distances and elevations are approximate. Native soils are as interpreted by Pali Consulting staff, see Report and Appendix B for further description of subsurface conditions. $N_{avg}$ refers to average SPT blow counts (see Report and Appendix B).

Noble Creek Tidegate
Coos County, Oregon

## Interpretive Cross Section

#014-21-006 August 2023

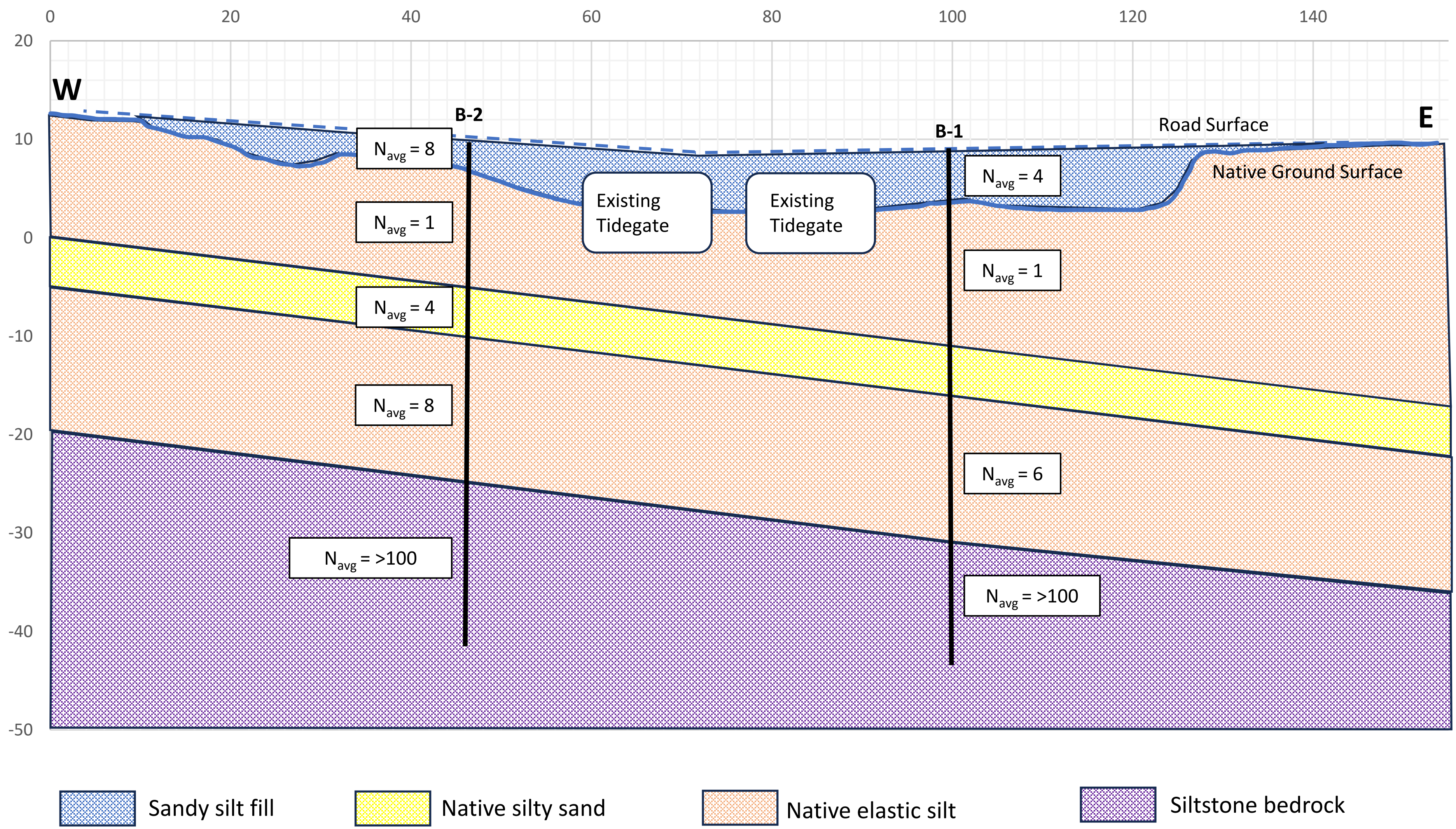


Figure 3b

Notes: Cross sections based on LiDAR data provided by DOGAMI. All distances and elevations are approximate. Native soils are as interpreted by Pali Consulting staff, see Report and Appendix B for further description of subsurface conditions. $N_{avg}$ refers to average SPT blow counts (see Report and Appendix B).

Noble Creek Levee
Coos County, Oregon

Interpretive Cross Section
#014-21-006 August 2023

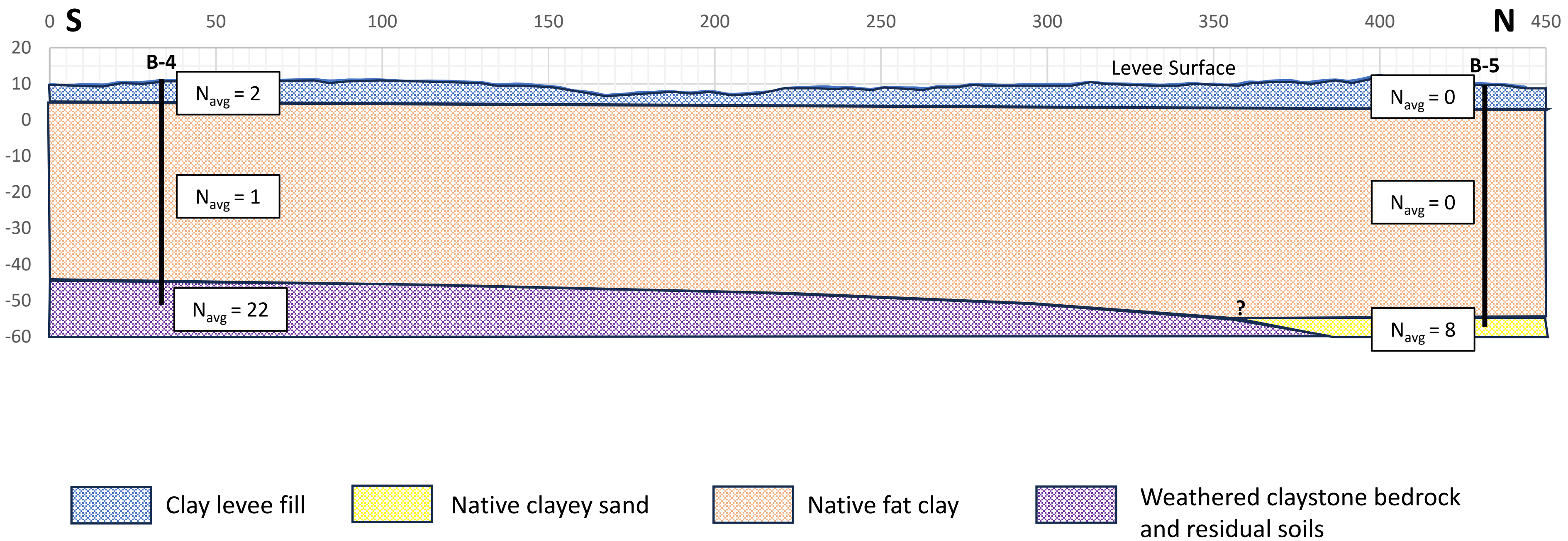
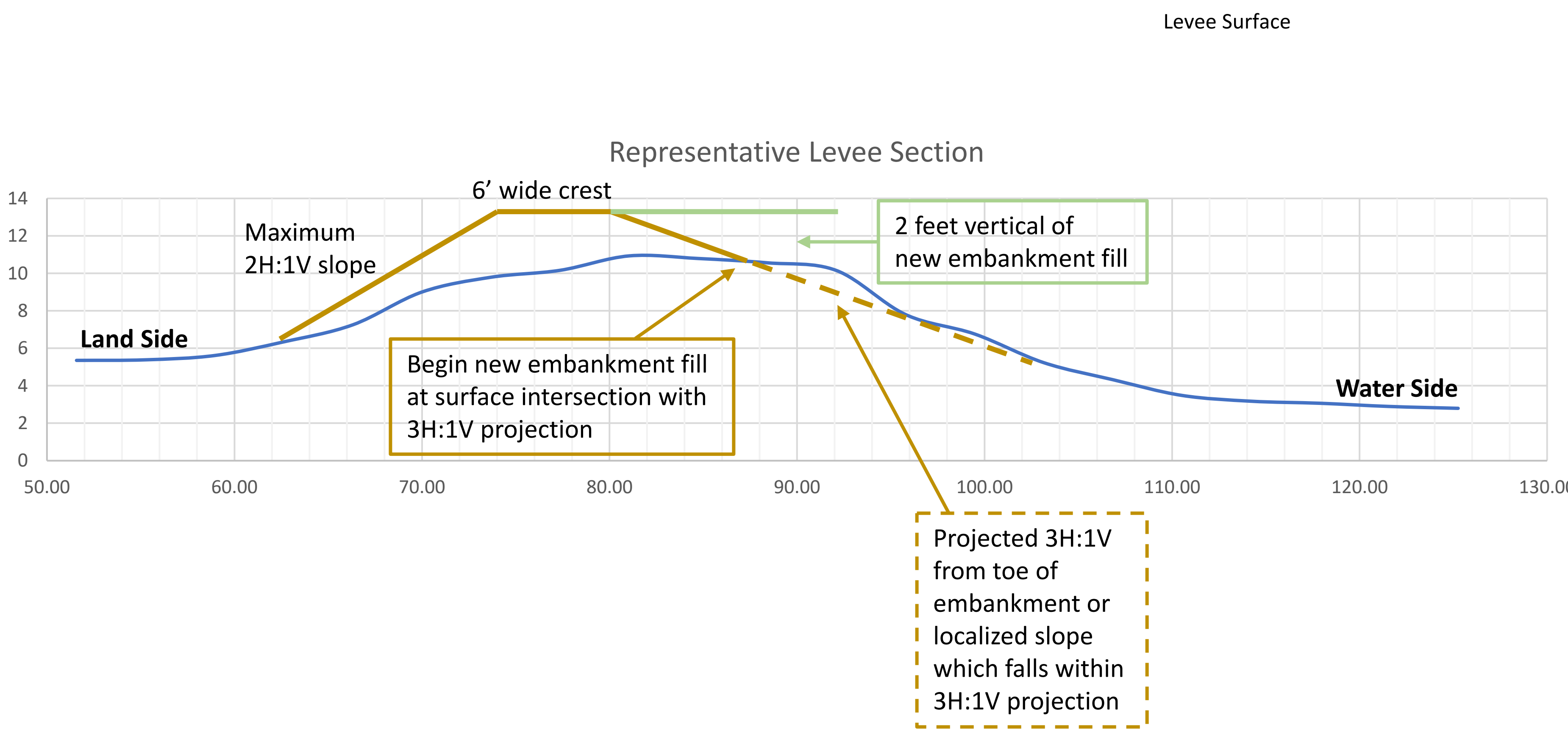


Figure 4

Notes: Cross sections based on LiDAR data provided by DOGAMI. All distances and elevations are approximate.

Noble Creek Levee
Coos County, Oregon

Representative Levee Section
#014-21-006 August 2023



# APPENDIX A -

## OCTOBER 13, 2022

## PRELIMINARY GEOTECHNICAL REPORT

## NOBLE CREEK TIDAL RESTORATION PROJECT

Pali Consulting

October 13, 2022

Waterways Consulting, Inc.
1020 SW Taylor Street, Suite 380
Portland, Oregon 97205
Attention: Mr. Daniel Malmon

REGISTERED PROFESSIONAL ENGINEER
52838
OREGON
JAN. 12, 1999
TIMOTHY W. BLACKWOOD
EXPIRES 12/31/2023

Preliminary Geotechnical Report
Noble Creek Tidal Restoration Project
Coos County, Oregon
Pali Consulting Project #014-21-006

## 1.0 INTRODUCTION

This memorandum provides Pali Consulting, Inc.'s (Pali Consulting's) preliminary geotechnical report for the Noble Creek Tidal Restoration Project (Noble Creek Project) in Coos County, Oregon. Waterways Consulting, Inc (Waterways) is designing the project for the Coos Soil and Water Conservation District (Coos SWCD) and requested that Pali Consulting provide geotechnical services for the project. Our work was completed in general accordance with our agreement with Waterways for the project dated January 5th, 2022. The location of the site is shown on Figure 1.

The project is located at Noble Creek where it parallels Green Acres Lane east of Highway 42 about 2.5 miles south of Southport, Oregon. It includes habitat improvements to include upgrading three interior tide gates, replacing the main tide gates at the north end of Green Acres Lane or removing them and replacing them with a bridge, and replacing or repairing an existing levee which protects existing structures. As an alternative, a second bridge, the Bramble Lane Bridge, may be replaced, but is not addressed in this report. A site plan with the limits of the project area and significant site features is shown on Figure 2.

Our scope of work for this first phase of the project included a review of background information, a field reconnaissance of the site, and limited subsurface explorations using hand methods. Our findings from this phase are intended to identify significant geotechnical elements affecting the various design options and bracket the likely range of design variables. The results of this phase of the project will guide Phase II work, which may involve more comprehensive subsurface explorations and final geotechnical design for the project.

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## 2.0 BACKGROUND INFORMATION

We reviewed geologic and soil mapping of the site, water well logs in the general area, and historic aerial photos of the site. Our work did not include an evaluation of geologic hazards, such as earthquake shaking, and earthquake-induced liquefaction and tsunami inundation. These hazards may be significant at this site but were not within our scope of work to evaluate. A summary of our review follows.

### 2.1 GEOLOGY

The geology of the site is mapped by the Oregon Department of Geology and Mineral Industries' (DOGAMI) Interactive Geologic Map of Oregon (https://gis.dogami.oregon.gov/maps/geologicmap/, accessed February 2022). Three geologic formations are mapped at the site: (1) Quaternary alluvial deposits consisting of mixed grained, unconsolidated sediments, which are found along Noble Creek and Isthmus Slough; (2) Eocene deltaic sandstones and slope mudstones of the Coaledo Formation, surrounding the creek channel to the north, east, and west; and (3) Eocene slope mudstones of the Bastendorff Formation which are at the southern end of the site. According to the Geologic Map of the Coos Bay Quadrangle (Black and Madin, 1995) all three of these formations are unconformably underlain by well-indurated sandstone of the Eocene Tyee Formation. Black and Madin also map numerous faults and folds at and near to the site, including an unnamed syncline which runs NE-SW through the center of the site. The nearest mapped fault is the Isthmus Slough Fault, a high-angle reverse fault which runs NE-SW about 1/4-mile northwest of the site. The site is located on the upthrown block of the fault.

### 2.2 SOILS

Soils within the area are mapped by the Natural Resources Conservation Service's Web Soil Survey (https://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx, accessed February 2022). Mapping shows two soil types in the project area: Coquille silt loam and Langlois silty clay loam. The most common soil type in the area is Coquille silt loam, which covers the area surrounding Noble Creek, except for in the northwest corner where it joins with the Isthmus Slough, and the triangular-shaped area southwest of Noble Creek. Coquille silt loam is described as typically found on floodplains and has a parent material of alluvium. The soil has a typical profile of silt loam from 0 to 14 inches and silty clay loam from 14 to 60 inches. The water table is typically at the ground surface (0 inches depth) and depths to a restrictive feature are more than 80 inches. It is described as very poorly drained with a moderately low to moderately high capacity of the most limiting layer to transmit water (0.06 to .20 in/hr). The northeast portion of Noble Creek where it joins with the Isthmus Slough is mapped as Langlois silty clay loam, which is typically found on floodplains and has a parent material of mixed alluvium. The soil has a typical profile of silty clay loam from 0 to 10 inches, silty clay from 10 to 28 inches, and clay from 28 to 60 inches. The water table is typically at the ground surface (0 inches depth) and depths to a restrictive feature are more than 80 inches. It is described as very poorly drained with a moderately low to moderately high capacity of the most limiting layer to transmit water (0.06 to .20 in/hr).

### 2.3 WATER WELL LOGS

We reviewed water well logs from the surrounding area to better understand subsurface conditions. Well logs generally reported clay to about 40 feet which was typically underlain by sandstones and siltstones. Static water levels varied widely, from 2 feet below ground surface (bgs) about 450 feet east of the Bramble Lane Bridge to 69 feet bgs about a quarter mile south of the Bramble Lane Bridge. The closest well to

Noble Creek, located about 450 feet east of the Bramble Lane Bridge, reported a static water level of 2 feet bgs, and found clays to 43 feet and siltstone from 43 to 80 feet.

### 2.4 AERIAL PHOTOGRAPHS

We reviewed aerial photographs of the area from Google Earth Pro™ to evaluate historical changes to the surface drainage system. During the photo period reviewed (1994-2019) we did not observe changes to the drainage channels, however we acknowledge that changes may have occurred which were not visible at the air photo scale. In addition, we reviewed air photo imagery for the years 1942, 1957, and 1969, accessed via USGS's Earth Explorer website. In the earliest air photo (1942) the tide gates at the end of Isthmus Slough appear to be in place, however no channels are yet cut into the surface, which still appears in a natural condition. The 1957 air photo shows significant changes to surface drainage including new canals and apparent backfilling of the southwestern portion of the floodplain. The results of these changes are apparent in the 1969 air photo, which shows the drainage system more or less as it is in the present day.

## 3.0 SITE CONDITIONS

We evaluated site conditions by conducting a geologic reconnaissance and completing hand subsurface explorations. Subsurface explorations and laboratory testing are described in Appendix A. Photographs from the site are included in Appendix B. The following sections describe our findings.

### 3.1 SURFACE CONDITIONS

The site is an irregularly shaped area which generally runs along Noble Creek and Isthmus Slough in the vicinity of Green Acres Lane. In this area, Noble Creek is bound to the east by Green Acres Lane and Lower Loop Road, and to the west by an existing levee that parallels the creek. The northern extent of the project area is defined by an existing failed tidegate at the junction between Noble Creek and Isthmus Slough. The southern extent of the project area is where Noble Creek Road crosses Noble Creek at the Bramble Lane bridge. Alterations to natural conditions at the site include the tidegate, levee, and the Green Acres Lane road prism. Elevations at the site range from about 11 feet above mean sea level (MSL) on Green Acres Lane atop the failed tidegate to 3 feet MSL along the banks of Noble Creek. The top of the levee varies from about 7 to 9 feet MSL.

Vegetation is mostly pasture grasses with some mixed conifer and deciduous trees planted along the top of the levee and some native deciduous trees, reeds, and shrubs along the creek.

We completed a detailed reconnaissance and documented physical features along the levee and Green Acres Lane. Specific locations where we collected information are noted on Figure 2, and photographs referenced below are in Appendix B.

#### 3.1.1 TIDEGATE CONDITIONS

An existing failed tidegate is present at the junction between Noble Creek and Isthmus Slough, at the north end of the site where Green Acres Lane crosses Noble Creek. The tidegate is in poor condition, with the gate askew and the wooden cribbing adjacent the gate beginning to rot (Photo B-1). A short staircase leads from the road down to a concrete walkway atop the tidegate. The walkway is likewise in poor condition, with concrete blocks ranging from loose to not present. Between the walkway and the road, the embankment consists of heavily raveled bare soil with sparse grass cover (Photo B-2). The base of the embankment is undercut up to 6 inches in places. Below the walkway, riprap has been placed on the slopes adjacent the tidegate (Photo B-3). The riprap has been undercut by up to 2 feet in areas and some has fallen to the channel below, exposing native silty sand to sandy silt.

#### 3.1.2 GREEN ACRES LANE CONDITIONS

Green Acres Lane runs along the northeast side of Noble Creek in the project area. Significant features of pavement distress were noted on the southwest side of the road closest Noble Creek including arcuate cracking, sagging, and displacement of the roadway. These features are most prominent in the vicinity of Hand Auger 3 (HA-3, see Figure 2). In this approximately 100 feet long area, two large arcuate cracks and many smaller accessory cracks define a zone of pavement sagging and displacement approximately 7 feet in width (Photo B-4). Displacement on the two largest cracks ranges from 0 to 2.5 inches in the horizontal direction and from 0 to 3 inches in the vertical direction. Asphalt patches cover parts of the cracks and may obscure areas of greater displacement. Probing with a ½" steel foundation probe indicated that the cracks extend up to 15 inches deep. Overall, about 4 inches of sagging is present between the centerline and the southwest edge of the roadway.

In the most significantly distressed area of the roadway approximately 3-4 feet of rip rap has been placed along the road prism above Noble Creek, with metal sheets to keep it in place (see Photo B-4). The rip rap has settled between 4 and 18 inches (assuming it was originally placed at roadway elevation) and there is a crack 0 to 6 inches wide running between the rip rap and the edge of the roadway (Photo B-5).

There is a second, smaller zone of cracking and settlement in the roadway centered about the location of Hand Auger 2 (HA-2, see Figure 2). In this area minor arcuate cracks with horizontal displacement of up to ½ inch and vertical displacement of up to ¼ inch are present approximately 2 to 4 feet from the southwest edge of the roadway (Photo B-6). Probing with a ½" steel foundation probe indicated that the cracks extend up to 3 inches deep. Overall, about half an inch of sagging is present between the centerline and the southwest edge of the roadway in this area.

North and south of these two areas pavement conditions appear normal, with only minor pavement distress observed that is typical of roads of similar age and use.

#### 3.1.3 LEVEE CONDITIONS

A levee runs along the southwest side of Noble Creek, protecting pasture land and a residential building to the west. The levee is primarily bare dirt on the top and west sides. Scrubby grasses, cedar, and apple trees grow along the eastern side, which slopes down towards Noble Creek at about 30 percent. Minor ground disturbance is evident on the east side of the levee including "steps" of up to 6 inches which expose bare soil (Photo B-7). The majority of these disturbances appear related to livestock traversing the levee, as was demonstrated while we were on-site (Photo B-8). Other than these ground disturbances, we found the levee to be in generally good condition.

### 3.2 SUBSURFACE CONDITIONS

We completed four hand auger borings and one drive probe sounding at the locations shown on Figure 2. Laboratory testing was conducted on representative samples collected from the borings. A description of our subsurface exploration program and laboratory testing, and logs of the boring are included in Appendix A. The soils we encountered are described in more detail below based on their area of occurrence.

#### 3.2.1 Creekside Conditions

Hand Augers 1, 2, and 3, and Drive Probe 1 were completed in native soils adjacent Noble Creek at low tide. On the north side of the tidegate, Hand Auger 1 (HA-1) encountered alternating layers of clayey sand and sandy fat clay with variable organics to about 2.67 feet deep, at which point our auger was terminated due to caving. The clayey sand/sandy clay was loose/soft, moist to wet, grey in color, and mottled at all depths. DP-1 was completed immediately adjacent HA-1 and found blow counts to range between 1 and 4 blows per 6 inches in the upper 3 feet below ground surface (bgs), which increased to 6 to 11 blows from 3-7 feet bgs, and increased again to 8 to 17 blows from 7 to 12 feet bgs. Blow counts increased significantly to between 28 and 35 blows between 12 and 13.5 feet bgs, the maximum depth of exploration.

On the south side of the tidegate, Hand Augers 2 and 3 (HA-2 and HA-3, respectively) both encountered soft, moist to wet, dark blue-grey fat clay with organics which became mottled with light grey at approximately 2 feet bgs. Both HA-1 and HA-2 were terminated between 2 and 3 feet bgs due to caving.

The fat clay was of high plasticity and generally contained much organic material. Atterberg limits testing on one sample from HA-3 found a plasticity index (PI) of 70, leading to a CH/OH classification per ASTM. The clay contained fine sand on the north side of the tidegate (within Isthmus Slough) which decreased on the south side of the tidegate (within Noble Creek). Laboratory fines content testing conducted on a sample taken in a clayey sand on the north side of the tidegate measured 41 percent passing the number 200 sieve, the division between fine-grained and coarse-grained soils. Moisture content in the clayey sand found in HA-1 was measured at 62 percent, while moisture contents in the fat clays varied from 90 to 132 percent, reflecting a high amount of organic material in the soil.

Groundwater was encountered at approximately 1 foot bgs in all three creekside borings.

#### 3.2.2 Levee Conditions

Hand Auger 4 (HA-4) was completed atop the levee southwest of Noble Creek. HA-4 encountered 4.5 feet of sandy silt fill overlying native fat clay to 6 feet bgs, at which point our auger was terminated due to caving. The sandy silt was medium stiff, moist, tan in color, and became mottled with grey and red at approximately 3 feet bgs. A laboratory sieve test measured 71 percent passing the number 200 sieve, the division between fine-grained and coarse-grained soils. Moisture contents in the sandy silt ranged from 27 percent closest to the ground surface to 71 percent at 4 feet bgs. The underlying native fat clay was soft, moist to wet, grey in color, and mottled at all depths. Moisture content in the clay was measured at 89 percent in a sample taken at 5 feet bgs.

## 4.0 CONCLUSIONS

Based on our evaluation, the following are the primary geotechnical factors that may affect the project, (excluding seismic considerations).

- The existing levee exhibits distress based on the overlying roadway damage, which is likely a combination of construction over soft soils and oversteepened slopes adjacent Noble Creek, with the latter being the more significant factor. Permanent stabilization of the slopes will be difficult as it would require stabilization of the toe (likely in-water work) or flattening of the slope (requiring roadway widening to the north if the current road width is to be maintained). Other options may also be considered.

- Shallow native soils are weak and prone to elastic and consolidation settlement but appear to stiffen at about 10 to 15 feet bgs. Depending on the vertical extent and trend of the observed increasing stiffness with depth, settlements may be limited compared to if soft soil continued to greater depth. If this is the case, minor to moderate changes to levee configuration would not induce excessive settlements, and culverts can bear on the soils without the risk of excessive settlement.
- Similar to improving consolidation characteristics with depth, bearing characteristics appear to improve at about 10 to 15 feet bgs. If bridges will be constructed, depending on footing depth and loads, it may be possible to support them on shallow foundations.
- If shallow foundations are found not to be feasible, deep foundations bearing in sedimentary bedrock (if present) at around 50 feet bgs should be suitable to support the brdige, based on nearby water well logs.
- On-site soils are not optimal for earthworks construction due to their high plasticity and organic content. These soils, are not recommended for structural fill.
- The high water table will make earthworks construction difficult, especially excavation below existing floodplain grade. Wet weather construction methods, dewatering, and shoring requirements should be contemplated in project design and budgeting.

Further discussion of our anticipated geotechnical recommendations is provided in the following sections.

# 5.0 RECOMMENDATIONS

Based on our evaluation, the following are the primary geotechnical factors that will affect the project, (excluding seismic factors).

## 5.1 Culvert and Tide Gate Support

We understand that new culverts and tide gates may be installed at the approximate location of HA-1. Based on our subsurface explorations and analyses, we recommend bearing pressures not exceed 1,500 pounds per square foot (psf) locally and 500 psf average across the structure footprint. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and may be increased by one-third for short-term loads.

We recommend a layer of rock be placed over the subgrade to limit disturbance prior to placing the culvert/tide gate structure on grade. If placed in the dry, the rock should consist of a 6-inch layer of pit or quarry run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSS 00330.14 – *Selected Granular Backfill* or OSS 00330.15 – *Selected Stone Backfill*. The imported granular material should also be angular, fairly well graded between coarse and fine material, have less than 5 percent by dry weight passing the U.S. Standard No. 200 Sieve, and have at least two mechanically fractured faces. If placed in the wet, a 12 to 18-inch thickness of imported granular material should be placed as a subgrade stabilization layer. The stabilization layer should meet the specifications of OSS 00330.14 Selected Granular Backfill, or OSS 0330.15 Selected Stone Backfill. Construction geosynthetic for soil stabilization (per Table 02320-4 of OSS 02320) may be used to prevent migration of fines into the void spaces of the coarser material. The rock should be compacted to a dense well-keyed condition with appropriate equipment, but not to cause damage or softening of the subgrade. Provided the structure is placed as recommended it should experience "static" settlement of less than 2 inches, with differential settlement of less than 1 inch over an approximate 50 feet span. Levee soils placed above the culvert/tide gate structure will cause additional settlement beyond those estimated above.

Lateral loads on footings can be resisted by friction on bearing surfaces. Friction coefficients of 0.25 or 0.40 may be used for pre-cast concrete footings placed on native silt or a minimum 6-inch-thick layer of imported granular fill, respectively. Imported granular fill should meet the specifications provided in OSS 00330.14 – *Selected Granular Backfill* or OSS 00330.15 – *Selected Stone Backfill*. The imported granular material should also be angular, fairly well graded between coarse and fine material, have less than 5 percent by dry weight passing the U.S. Standard No. 200 Sieve, and have at least two mechanically fractured faces. It should be compacted to a well-keyed state as noted above.

Additional resistance to lateral loads can be achieved by passive earth pressures on the sides of embedded elements. However, passive resistance can only be relied upon 1 foot and greater below the ground surface, or the potential depth of scour or erosion, whichever is greater. Passive earth pressures can be calculated using an equivalent fluid weight of 290 pounds per cubic foot (pcf) for footings backfilled with compacted imported structural fill. The passive earth pressure and friction components may be combined, provided that the passive component does not exceed two-thirds of the total. The above lateral resistance values do not include safety factors.

Where culvert/tidegate structures are placed, settlement caused by new levee fill could cause significant settlement of the culvert/tidegate. In this case, we would recommend placing new levee fill first, monitoring settlement of the levee and installing the culvert/tidegate once survey has confirmed that settlement is acceptably complete.

### 5.2 Bridges

Design of bridges will require geotechnical explorations to determine feasible types as well as footing capacities. It is most likely that deep foundations will be required, such as pipe piles or H-piles supporting pile caps. Piles will likely bear on sedimentary rock at depths of about 50 feet bgs based on nearby water well logs.

Retaining walls to support abutments would likely consist of sheetpile walls, but concrete cantilever walls, either supported on an overexcavated pad or on the bridge bent pilings, may also be feasible.

If soils are sufficiently stiff at shallow depths (within about 10 to 15 feet based on our limited explorations) shallow foundations may be an option for support of the bridge. If so, soft soils may need to be overexcavated and the footings supported on granular fill pads. In this case, excavation stability and dewatering may be significant factors.

### 5.3 Earthworks

Native soils to at least 3 feet bgs consist of fat clay with organics. Such soils are generally not suitable for use as structural fill. We recommend they not be included in project planning as structural fill material. If the existing roadway levee is modified or reconstructed, levee soils appear suitable for re-use as structural fill, although they will be moisture sensitive. Soil types below 3 feet bgs were not evaluated as our hand augers within native soils could not be completed below this depth due to caving.

Groundwater was encountered at about 1 feet bgs in HA-1 through HA-3, which were completed at floodplain grade. Groundwater is expected to be present at similar depths during most of the year, and likely to the ground surface, especially during the wet season. This will result in several geotechnical constraints on excavations and fill, including:

- Caving of excavations where they extend below the water table.
- Dewatering of deeper excavations. The fine-grained soils will generally have slow seepage/infiltration characteristics, so dewatering can likely use a sump pump type system.
- Stabilization of subgrade soils with stabilization rock and geotextile fabric as they will generally be saturated, very soft, and easily disturbed during most of the year.

Soils within the levee are expected to consist mostly of sandy silt with localized organics and possible deleterious materials. These sandy silt soils can be reused as structural fill provided such deleterious materials are separated from the silt soils.

### 5.4 Levee/Roadway Repair

As noted in Section 4, repair of unstable parts of the levee (mainly those exhibiting roadway distress) will be difficult. Grading only solutions may require in-water work in very soft soils or flattening the steep slopes along Noble Creek and moving the road prism to the north. This would initiate some settlement where significant new fill is placed. Staged fill construction might be required to prevent instability during construction.

Other solutions to consider include a reinforced earth fill embankment in the same location and cross section as the existing levee. The reinforced section may be all of the levee or possibly just levee slopes. Structural solutions using walls are also technically feasible.

## 6.0 CONSTRUCTION MONITORING

Satisfactory earthwork and foundation performance depend to a large degree on quality of construction. Sufficient monitoring of the contractor's activities is key to determining that the work is completed in accordance with our recommendations. Subsurface conditions observed during construction should be compared with those encountered during the subsurface exploration. Recognition of changed conditions often requires experience; therefore, Pali Consulting or their representative should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

We recommend that Pali Consulting be retained to monitor construction of geotechnical elements at the site to confirm that subsurface conditions are consistent with the site explorations and to confirm that the intent of the project plans and specifications relating to earthwork and culvert/tidegate installation are being met. In particular, we recommend that subgrade preparation and fill compaction for the culvert/tidegate be observed by Pali Consulting.

## 7.0 LIMITATIONS

This evaluation is based on a limited scope of work which was negotiated between Pali Consulting, Waterways and the Coos SWCD. The opinions and recommendations contained within this report are, therefore, based primarily upon surface observations supplemented with explorations of limited extent and depth. Our report should not be construed as a warranty or guarantee of site conditions or performance. Soil conditions can differ from those encountered during our field work, as well as during different seasons, from earth processes, from storms, or other factors that occur after our work has been completed.

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

We appreciate the opportunity to provide this information for you. Please contact us if we can be of further assistance or if you have any questions.

## 8.0 REFERENCES

Black, G.L., and Madin, I.P., 1995. Geologic Map of the Coos Bay Quadrangle, Coos County, Oregon, GMS-96. Scale 1:24,000.

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United States Department of Agriculture (USDA), Natural Resources Conservation Service, Web Soil Survey (https://websoilsurvey.nrcs.usda.gov), accessed February 2022.

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Attachments:
- Figures 1 and 2
- Appendix A – Subsurface Explorations and Laboratory Testing
- Appendix B – Site Photographs

Document ID: 014-21-006NobleCreekPreliminaryGeotechnicalReport

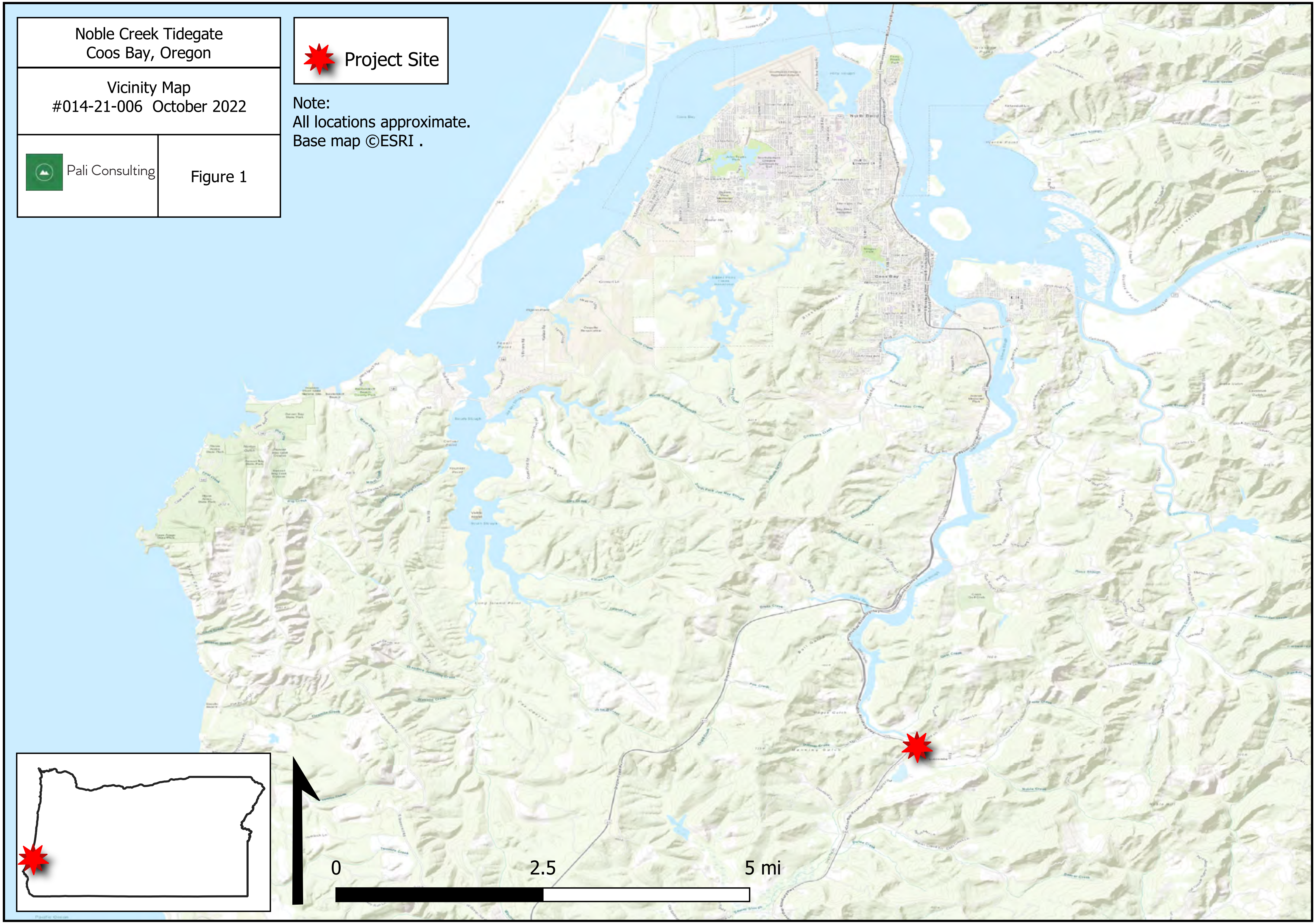
Noble Creek Tidegate
Coos Bay, Oregon

Vicinity Map
#014-21-006 October 2022

Pali Consulting

Figure 1

Note:
All locations approximate.
Base map ©ESRI .




Noble Creek Tidegate
Coos Bay, Oregon
Site Map
#014-21-006 October 2022
Pali Consulting
Figure 2
Exploration Locations
Levee
Isthmus Slough
HA-1/DP-1
Tidegate
Area of pavement cracking / slumping
HA-2
HA-3
HA-4
Green Acres Ln
Noble Creek
Bramble Lane Bridge
0
100
200 ft
Note:
All locations approximate.
Base Map ©Google

# APPENDIX A -
## FIELD EXPLORATIONS AND LABORATORY TESTING

# FIELD EXPLORATIONS

## GENERAL

We evaluated subsurface soil and groundwater conditions at the site by completing four hand auger borings and one drive probe sounding on August 26th, 2022. The hand auger borings and drive probe sounding were completed by Pali Consulting staff. The locations of the explorations are shown on Figure 2 of the report. The exploration locations were approximately located with a recreational grade GPS so should be considered accurate only to the degree implied by the locating methods used.

## SAMPLING AND LOGGING

Soil samples were collected from the borings at the intervals noted on the exploration logs in this appendix. Sampling was completed by collecting "grab" samples for further evaluation. All disturbed samples obtained were sealed in watertight containers and transported to our laboratory for subsequent classification and testing. Soil sampling intervals are shown in the exploration logs in this appendix.

The field explorations were coordinated by a geologist on our staff, who located the explorations, classified the various soil units encountered, obtained representative soil samples for geotechnical testing, observed and recorded groundwater conditions, and maintained a detailed log of each exploration. Exploration logs are included in this appendix.

# LABORATORY TESTING

## GENERAL

Soil samples obtained from the explorations were transported to our laboratory and evaluated to confirm or modify field classifications, as well as to evaluate engineering properties of the soils encountered. Representative samples were selected for laboratory testing. The tests were performed in general accordance with the test methods of the ASTM or other applicable procedures. Test results are indicated on the boring logs.

## VISUAL CLASSIFICATIONS

Soil samples obtained from the explorations were visually classified in the field and in our geotechnical laboratory based on the USCS and ASTM classification methods. ASTM Test Method D2488 was used to classify soils using visual and manual methods. ASTM Test Method D2487 was used to classify soils based on laboratory test results.

### *Moisture Content*

Moisture contents of samples were obtained in general accordance with ASTM Test Method D2216. The results of the moisture content tests completed on samples from the explorations are presented on the exploration logs included in this appendix.

### *Fines Content Analyses*

Fines content analyses were performed to determine the percent of soils finer than the U.S. No. 200 Sieve, the boundary between sand size particles and silt size particles. The tests were performed in general accordance with ASTM Test Method D 1140. The test results are indicated on the exploration logs included in this appendix.

### *Atterberg Limits*

Atterberg limits (liquid limit, plastic limit and plasticity index) of fine-grained soil samples were obtained in general accordance with ASTM Test Method D4318-02. The results of the Atterberg limits tests completed on samples from the explorations are presented on the exploration logs included in this appendix.

# KEY TO EXPLORATION LOGS

Pali Consulting

4891 Willamette Falls Drive, Suite 1
West Linn, Oregon 97068
www.pali-consulting.com

## SOILS CLASSIFICATION CHART

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

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

| SYMBOLS LETTER | DESCRIPTIONS |
|---|---|
| CC | CEMENT CONCRETE |
| AC | ASPHALT CONCRETE |
| TS | TOPSOIL/SOD FORREST DUFF |

### Stratigraphic Contact

- ____ Distinct contact between soil strata or geologic units
- ----- Gradual or approximate change between soil strata or geological units

### Moisture Modifiers

**Dry -** Absence of moisture, dusty, dry to the touch

**Moist -** Damp, but no visible water

**Wet -** Visible free water or saturated, usually soil is obtained from below the water table

### Seepage Modifiers

None

Slow - < 1 gpm

Moderate - 1- 3 gpm

Heavy - > 3 gpm

### Caving Modifiers

None

Minor - isolated

Moderate - frequent

Severe - general

### Minor Constituents

Trace: < 5% (silt/clay)

Occasional: < 15% (sand/gravel)

With: 5-15% (silt/clay) in sand or gravel

15-30% (sand/gravel) in silt or clay

### Sampler Symbol Descriptions

- Core
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- Bulk or grab

### Laboratory / Field Tests

| | |
|---|---|
| %F | Percent fines |
| AL | Atterberg Limits |
| CP | Laboratory compaction test |
| CS | Consolidation test |
| DS | Direct shear |
| HA | Hydrometer analysis |

### Laboratory / Field Tests

| | |
|---|---|
| DD | Dry density |
| OC | Organic content |
| PP | Pocket penetrometer |
| SA | Sieve analysis |
| TV | Torvane shear |
| MC | Moisture Content |

**Blowcount (N) is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted) per ASTM D-1586. See exploration log for hammer weight and drop.**

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

**(2.4-inch) sampler N approximately corrected to equivalent SPT N by 50% reduction in N - modified California.**

Note: Refer to the report text and exploration logs for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the exploration locations at the time the explorations were made. The logs are not warranted to be representative of the subsurface conditions at other locations or times.

# HA-1 / DP-1

**Surface Elevation: 4'**
**Boring Date: 8/26/22**
**Boring Location: North side of Tidegate**
**Drilling Method: Hand Auger, Williamson Drive Probe**

| Depth | Remarks | Moisture (%) | Dry Density | Blow Counts | Sample Type | Water Table | USCS | Description |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SM | Loose, moist, mottled brown and grey, clayey SAND with trace gravel and organics |
| | | | | 3 | | | CH | Grades to soft, wet, blue-grey, sandy FAT CLAY with infrequent thin lenses of light grey fine to medium sand |
| | | 90 | | 3 | | | | |
| | | | | 1 | | | | Increasing sand |
| 2 | | | | 3 | | | SM | Loose, wet, blue-grey, clayey FINE SAND |
| | %F=41 | 62 | | 4 | | | | Grades to clayey medium sand |
| | | | | 4 | | | END | HA-1 terminated at 2.67 ft bgs due to caving |
| | | | | 6 | | | | |
| 4 | | | | 7 | | | | |
| | | | | 11 | | | | |
| | | | | 9 | | | | |
| | | | | 8 | | | | |
| 6 | | | | 7 | | | | |
| | | | | 6 | | | | |
| | | | | 9 | | | | |
| | | | | 13 | | | | |
| 8 | | | | 10 | | | | |
| | | | | 12 | | | | |
| | | | | 8 | | | | |
| | | | | 14 | | | | |
| 10 | | | | 13 | | | | |
| | | | | 13 | | | | |
| | | | | 15 | | | | |
| | | | | 17 | | | | |
| 12 | | | | 28 | | | | |
| | | | | 33 | | | | |
| | | | | 35 | | | | |
| | | | | 35 | | | | |
| 14 | | | | | | | | |

File: C:\Users\Jane\Pali Consulting Dropbox\1-Projects\Active-Projects\014-Waterways\014-21-006NobleCreek\Analysis\NobleCreekLogs.log Date: 10/12/2022

## LOG OF BORING

| Pali Consulting | Noble Creek<br>Project No. 014-21-006 | A-4 |
|---|---|---|

# HA-2

**Surface Elevation: 4'**
**Boring Date: 8/26/22**
**Boring Location: Green Acres Ln Embankment**
**Drilling Method: Hand Auger**

| Depth | Remarks | Moisture (%) | Dry Density | Blow Counts | Sample Type | Water Table | USCS | Description |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | CH | Soft, moist, dark blue-grey FAT CLAY with much organics |
| | | 120 | | | | | | |
| 1 | | | | | | ▼ | | Grades to wet |
| | | | | | | | | Grades to mottled light and dark grey |
| | | 132 | | | | | | |
| 2 | | | | | | | END | HA-2 terminated at 2 ft bgs due to caving |
| 3 | | | | | | | | |
| 4 | | | | | | | | |
| 5 | | | | | | | | |
| 6 | | | | | | | | |
| 7 | | | | | | | | |

*File: C:\Users\Jane\Pali Consulting Dropbox\1-Projects\Active-Projects\014-Waterways\014-21-006NobleCreek\Analysis\NobleCreekLogs.log Date: 10/12/2022*

## LOG OF BORING

# HA-3

**Surface Elevation: 4'**
**Boring Date: 8/26/22**
**Boring Location: Green Acres Ln Embankment**
**Drilling Method: Hand Auger**

| Depth | Remarks | Moisture (%) | Dry Density | Blow Counts | Sample Type | Water Table | | |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | CH | Soft, moist, dark blue-grey FAT CLAY with much roots and organics |
| 1 | | | | | | | | Grades to wet, trace gravel |
| 2 | PI = 70 | 119 | | | | | | Grades to mottled light and dark grey with trace charcoal and gravel |
| | | | | | | | END | HA-3 terminated at 2.67 ft bgs due to caving |
| 3 | | | | | | | | |
| 4 | | | | | | | | |
| 5 | | | | | | | | |
| 6 | | | | | | | | |
| 7 | | | | | | | | |

File: C:\Users\Jane\Pali Consulting Dropbox\1-Projects\Active-Projects\014-Waterways\014-21-006NobleCreek\Analysis\NobleCreekLogs.log Date: 10/12/2022

## LOG OF BORING

Pali Consulting

**Noble Creek**
**Project No. 014-21-006**

**A-6**

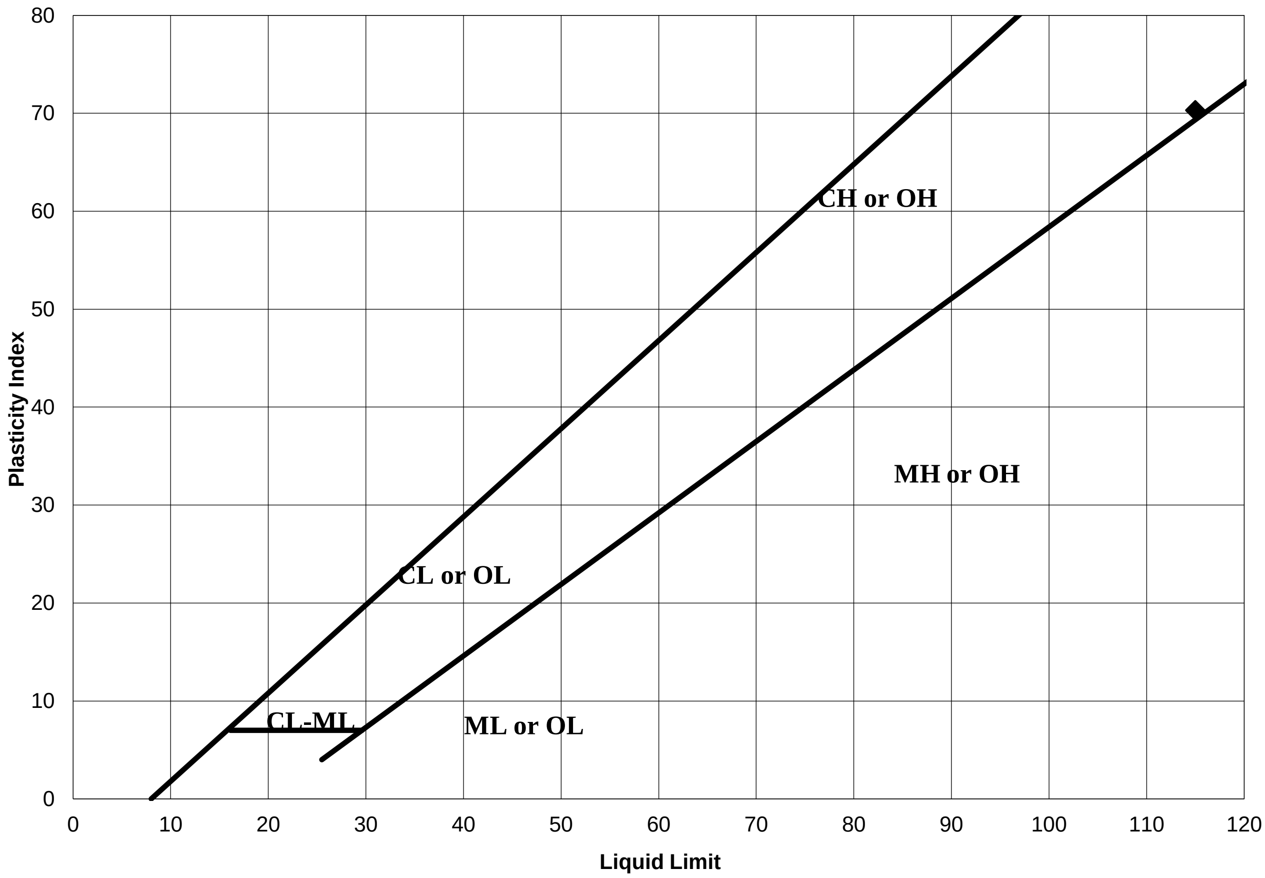
# HA-4

**Surface Elevation: 8'**
**Boring Date: 8/26/22**
**Boring Location:** **Levee on west side of Noble Creek**
**Drilling Method: Hand Auger**

| Depth | Remarks | Moisture (%) | Dry Density | Blow Counts | Sample Type | Water Table | USCS | Description |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | GWT not encountered | SM | Stiff, dry, tan, SANDY SILT with occasional small gravels, roots (levee fill) |
| 1 | %F=71 | 27 | | | Sample | | | |
| 2.5 | | | | | Sample | | | Grades to tan with minor light grey mottling |
| 3 | | | | | | | | Grades to medium stiff, tan and grey, decreasing sand |
| 3.5 | | | | | Sample | | | With minor rusty red mottles |
| 4 | | 71 | | | Sample | | | Grades to moist, decreasing sand |
| 4.5 | | | | | | | CH | Grades to soft, mottled, brown and grey, sandy FAT CLAY with charcoal (native) |
| 5 | | 89 | | | Sample | | | |
| 5.5 | | | | | Sample | | | Grades to grey with rusty red mottles, decreasing sand, increasing moisture, with much charcoal and organics |
| 6 | | | | | | | | Grades to wet, blue-grey |
| 6.25 | | | | | | | | HA-4 terminated at 6.25 ft bgs due to caving |
| 7 | | | | | | | | |

File: C:\Users\Jane\Pali Consulting Dropbox\1-Projects\Active-Projects\014-Waterways\014-21-006NobleCreek\Analysis\NobleCreekLogs.log Date: 10/12/2022

## LOG OF BORING




| ATTERBERG LIMITS DETERMINATION - ASTM D4318 | | | |
|---|---|---|---|
| **Project** | Noble Creek | **File No.** | 014-21-006 |
| **Boring No.** | HA-3 | **Sample No.** | S3 |
| **Lab ID No.** | N/A | **Depth** | 2.5 |
| **Soil Description** | CH | | |

| | | | |
|---|---|---|---|
| **Liquid Limit =** | 115 | **Date:** | 10/13/2022 |
| **Plastic Limit =** | 45 | **Tested By:** | JLE |
| **Plasticity Index =** | 70 | **Checked By:** | TWB |

Pali Consulting

# APPENDIX B –

# FIELD PHOTOGRAPHS

Photo B-1

Photo B-2

Photo B-3

Photo B-4

Photo B-5

Photo B-6

Photo B-7

Photo B-8

# APPENDIX B -
## FIELD EXPLORATIONS AND LABORATORY TESTING

## FIELD EXPLORATIONS

We evaluated subsurface conditions at the site by completing five drilled borings on May 8th, 9th, 23rd, and 24th, 2023. The borings were advanced using the mud-rotary method utilizing a truck-mounted drill rig for Borings B-1 and B-2, and a track-mounted drill rig for borings B-3 through B-5. Both drill rigs were operated by Western States Soil Conservation, Inc. The mud-rotary borings created holes approximately 6 inches in diameter. The holes were backfilled to approximately 12 inches bgs with hydrated bentonite chips and topped with gravel and cold patch asphaltic concrete where borings were completed on the roadway. Figure 2 shows the approximate location of the explorations.

The field explorations were coordinated by a geologist on our staff, who classified the various soil units encountered, obtained representative soil samples for geotechnical testing, and maintained a detailed log of each boring. Exploration logs are included in this appendix.

The exploration log within this appendix shows our interpretation of the drilling, sampling, and testing data and indicates the depth where the soils change. Note that the change may be gradual. In the field, we classified the samples taken from the exploration according to the methods presented on the *Key to Exploration Logs* in this appendix. The key also provides a legend explaining the symbols and abbreviations used in the logs.

Materials encountered in the exploration were classified in the field in general accordance with American Society for Testing and Materials (ASTM) International Standard Practice D 2488 “Standard Practice for the Classification of Soils (Visual-Manual Procedure).” Soil classifications and sampling intervals are shown in the exploration logs in this appendix.

Soil samples were obtained from the boring using the following methods.

- Sampling using a SPT sampler was completed in general conformance with ASTM Test Method D 1586 “Standard Method for Penetration Test and Split-Barrel Sampling of Soils.” The sampler was driven with a 140-pound auto-trip hammer falling 30 inches. The N-value, or number of blows required to drive the sampler 1 foot or as otherwise indicated into the soils, is shown adjacent to the sample symbols on the boring logs. Disturbed samples were obtained from the sampler for subsequent classification and testing.

- Relatively undisturbed samples were obtained using a thin-walled Shelby tube sampler in general conformance with ASTM Test Method D 1587 "Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes." The sampler is driven using the hydraulic down-pressure of the drill rig mast.

# LABORATORY TESTING

## GENERAL

Soil samples obtained from the explorations were evaluated to confirm or modify field classifications, as well as to evaluate their engineering properties. Representative samples were selected for laboratory testing. The tests were performed in general accordance with the test methods of the ASTM or other applicable procedures. Test results are indicated on the boring logs and as described below.

## SOIL CLASSIFICATIONS

Soil samples obtained from the exploration were visually classified in the field and in our geotechnical laboratory based on the USCS and ASTM classification methods. ASTM Test Method D2488 was used to classify soils using visual and manual methods. ASTM Test Method D2487 was used to classify soils based on laboratory test results.

## LABORATORY TESTING

### Moisture Content

Moisture contents of samples were obtained in general accordance with ASTM Test Method D 2216. The results of the moisture content tests are presented on the exploration log included in this Appendix.

### Fines Content Analysis

Fines content analyses were performed to determine the percent of soils finer than the U.S. No. 200 Sieve, the boundary between sand size particles and silt size particles. The tests were performed in general accordance with ASTM Test Method D 1140. The test results are indicated on the exploration log included in this Appendix.

### Atterberg Limits Testing

Atterberg Limits (liquid limit, plastic limit and plasticity index) of fine-grained soil samples were obtained from four samples in general accordance with ASTM Test Method D 4318-02. The results of the Atterberg Limits tests are presented on the exploration logs included in this Appendix.

### Consolidation Testing

Consolidation testing was completed on two samples by Northwest Testing, Inc., in general accordance with ASTM Test Method D 2435. The test results are presented in this Appendix.

### Density Testing

Density testing was performed on seven undisturbed soil samples in general accordance with ASTM Test Method D 7263-21. The results of the density tests are presented on the exploration logs included in this Appendix.

# KEY TO EXPLORATION LOGS

Pacific Geotechnical

**Pacific Geotechnical, Inc.**
**1419 Washington Street, Suite 101**
**Oregon City, Oregon 97045**

## SOIL CLASSIFICATION CHART

| MATERIAL TYPES | MAJOR DIVISIONS | | GROUP SYMBOL | SOIL GROUP NAMES & LEGEND |
|---|---|---|---|---|
| COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE | GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE | CLEAN GRAVELS <5% FINES | GW | WELL-GRADED GRAVEL |
| | | | GP | POORLY-GRADED GRAVEL |
| | | GRAVELS WITH FINES, >12% FINES | GM | SILTY GRAVEL |
| | | | GC | CLAYEY GRAVEL |
| | SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE | CLEAN SANDS <5% FINES | SW | WELL-GRADED SAND |
| | | | SP | POORLY-GRADED SAND |
| | | SANDS AND FINES >12% FINES | SM | SILTY SAND |
| | | | SC | CLAYEY SAND |
| FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE | SILTS AND CLAYS LIQUID LIMIT<50 | INORGANIC | CL | LEAN CLAY |
| | | | ML | SILT |
| | | ORGANIC | OL | ORGANIC CLAY OR SILT |
| | SILTS AND CLAYS LIQUID LIMIT>50 | INORGANIC | CH | FAT CLAY |
| | | | MH | ELASTIC SILT |
| | | ORGANIC | OH | ORGANIC CLAY OR SILT |
| HIGHLY ORGANIC SOILS | | | PT | PEAT |

Note: Multiple symbols are used to indicate borderline or dual classifications

| OTHER MATERIAL SYMBOLS |
|---|
| Concrete |
| Asphalt |
| Topsoil |

### MOISTURE MODIFIERS

Dry - Absence of moisture, dusty, dry to the touch
Moist - Damp, but no visible water
Wet - Visible free water or saturated, usually soil is obtained from below the water table

### SEEPAGE MODIFIERS

None -
Slow - < 1 gpm
Moderate - 1-3 gpm
Heavy - > 3 gpm

### CAVING MODIFIERS

None -
Minor - isolated
Moderate - frequent
Severe - general

### MINOR CONSTITUENTS

Trace - < 5% (silt/clay)
Occasional - < 15% (sand/gravel)
With - 5-15% (silt/clay) in sand or gravel
15-30% (sand/gravel) in silt or clay

### SAMPLE TYPES

Dames & Moore
Standard Penetration Test (SPT)
Shelby Tube
Bulk or Grab

### LABORATORY/ FIELD TESTS

ATT - Atterberg Limits
CP - Laboratory Compaction Test
CA - Chemical Analysis (Corrosivity)
CN - Consolidation
DD - Dry Density
DS - Direct Shear
HA - Hydrometer Analysis
OC - Organic Content
PP - Pocket Penetrometer (TSF)
P200 - Percent Passing No. 200 Sieve
SA - Sieve Analysis
SW - Swell Test
TV - Torvane Shear
UC - Unconfined Compression

### GROUNDWATER SYMBOLS

Water Level (at time of drilling)
Water Level (at end of drilling)
Water Level (after drilling)

### STRATIGRAPHIC CONTACT

Distinct contact between soil strata or geologic units
Gradual or approximate change between soil strata or geologic units

Notes:

Blowcount (N) is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted) per ASTM D-1586. See exploration log for hammer weight and drop.

(2.4-inch) sampler N approximately correlated to equivalent SPT N by 50% reduction in N - modified California.

Refer to the report text and exploration logs for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the exploration locations at the time the explorations were made. The logs are not warranted to be representative of the subsurface conditoins at other locations or times.

Pali Consulting

# Noble Creek Tidelands Project
**Coos County, OR**

**B-1**

| | |
|---|---|
| Project: Noble Creek Tidelands Project | Driller: Western States Soil Conservation, Inc |
| Proj No. 014-21-006-2 | Date: 5/9/23 |
| Drilling Method: Mud Rotary | Elevation: 11' |
| Diameter: 4" / Water Table: N/A | Logged by: JLE |

| Sample No. | Sample Type | Recovery (%) | RQD (%) | Blow Count per 6 inches | Blows/Foot (N) | Water Table | Depth (ft BGS) | Graphic Log | Materials Description | Moisture (%) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | GWT not encountered | 0 | AC | Asphalt roadway surface underlain by 6" well graded gravel (Roadway Base Rock) | | |
| | | | | | | | | ML | Soft to medium stiff, moist, brown, SANDY SILT (FILL) | | |
| S1 | | 20 | | 2-2-2 | 4 | | | | | 43 | |
| S2 | | 50 | | 0-1-1 | 2 | | 5 | CH | Soft, wet, grey, FAT CLAY with minor charcoal (NATIVE) | 66 | AL, PI=60, LL=100, PL=40 |
| | | | | | | | | | Grades to dark grey | | |
| S3 | | 75 | | | | | | | | | DD=27 pcf |
| S4 | | 100 | | 0-0-0 | 0 | | 10 | MH | Very soft, wet, dark grey ELASTIC SILT with variable grey sand | 83 | AL, PI=71, LL=120, PL=49 |
| S5 | | 100 | | | | | | | | | CS |
| S6 | | 100 | | 0-0-1 | 1 | | 15 | | With interbedded very loose, wet, dark grey SILTY SAND, with minor charcoal and wood | 83 | %F=64 |
| S7 | | 100 | | | | | 20 | SM | Very loose, wet, dark grey SILTY SAND | | |
| S8 | | 100 | | 0-1-2 | 3 | | 25 | MH | Soft, wet, dark grey with minor orange mottling, ELASTIC SILT with charcoal and wood | 39 | AL, PI=26, LL=57, PL=31 |
| S9 | | 100 | | | | | 30 | | Grades to light grey | | |

*File: C:\Users\Jane\Pali Consulting Dropbox\1-Projects\Active-Projects\014-Waterways\014-21-006NobleCreek\02-GeotechnicalDesign\Analysis\NobleCreekLogs.log   Date: 8/18/2023*

Pali Consulting

# Noble Creek Tidelands Project
**Coos County, OR**

## B-1

| | |
|---|---|
| Project: Noble Creek Tidelands Project | Driller: Western States Soil Conservation, Inc |
| Proj No. 014-21-006-2 | Date: 5/9/23 |
| Drilling Method: Mud Rotary | Elevation: 11' |
| Diameter: 4" / Water Table: N/A | Logged by: JLE |

| Sample No. | Sample Type | Recovery (%) | RQD (%) | Blow Count per 6 inches | Blows/Foot (N) | Water Table | Depth (ft BGS) | Graphic Log | | Materials Description | Moisture (%) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| S10 | | 100 | | 1-4-5 | 9 | | 35 | | CL | Grades to stiff, wet, light grey CLAY with minor fine sand, little to no charcoal or wood | 43 | |
| S11 | | 100 | | 30-50 for 4" | >100 | | 40 | | BR | Soft to medium hard (R2 - R3), grey, platy SILTSTONE (BEDROCK) | | |
| S12 | | 100 | | 22-50 for 3" | >100 | | 45 | | | Becomes less platy | | |
| S13 | | 80 | | 30-50 for 5" | >100 | | 50 | | | Grades to dark grey | | |
| | | | | | | | | | END | Boring completed at 51.5' BGS | | |
| | | | | | | | 55 | | | | | |
| | | | | | | | 60 | | | | | |
| | | | | | | | 65 | | | | | |

| Pali Consulting | **Noble Creek Tidelands Project** **Coos County, OR** | **B-2** |
|---|---|---|
| Project: Noble Creek Tidelands Project | Driller: Western States Soil Conservation, Inc | |
| Proj No. 014-21-006-2 | Date: 5/9/23 | |
| Drilling Method: Mud Rotary | Elevation: 13' | |
| Diameter: 4" — Water Table: N/A | Logged by: JLE | |

| Sample No. | Sample Type | Recovery (%) | RQD (%) | Blow Count per 6 inches | Blows/Foot (N) | Water Table | Depth (ft BGS) | Graphic Log | Materials Description | Moisture (%) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | GWT not encountered | 0 | AC | Asphalt roadway surface underlain by 6" well graded gravel (Roadway Base Rock) | | |
| S1 | | 35 | | 4-4-4 | 8 | | | MH | Medium stiff to stiff, moist, yellow to light grey, SANDY ELASTIC SILT with occasional gravel (FILL) | 33 | |
| N/A | | 0 | | 0-0-0 | 0 | | 5 | SM | Very loose to loose, moist, dark grey SILTY SAND (NATIVE) | | No recovery |
| S2 | | 100 | | | | | | MH | Very soft, moist, light grey ELASTIC SILT | 60 | AL, PI=26, LL=72, PL=46, DD=50 pcf |
| S3 | | 35 | | 2-2-1 | 3 | | 10 | | Grades to soft, with large pieces of wood in sampler | 68 | |
| S4 | | 70 | | 0-2-2 | 4 | | 15 | SM | Very loose to loose, moist, dark grey to orange SILTY SAND with zones of SILT | 33 | %F=47, Cuttings almost entirely wood from 12-15' DD=67 pcf |
| S5 | | 100 | | | | | | | | | |
| S6 | | 100 | | 1-3-5 | 8 | | 20 | MH | Medium stiff to stiff, moist, dark grey ELASTIC SILT with minor orange and red mottling | 43 | AL, PI=33, LL=66, PL=33 |
| S7 | | 100 | | | | | 25 | | Grades to mottled light grey, dark grey, and orange | | |
| S8 | | 100 | | 3-4-4 | 8 | | 30 | | Grades to light grey SANDY ELASTIC SILT | 37 | %F=60 Drillers report increased difficulty drilling at 32.5' |

File: C:\Users\Jane\Pali Consulting Dropbox\1-Projects\Active-Projects\014-Waterways\014-21-006NobleCreek\02-GeotechnicalDesign\Analysis\NobleCreekLogs.log *Date: 8/18/2023*

Pali Consulting

# Noble Creek Tidelands Project
**Coos County, OR**

## B-2

| | |
|---|---|
| Project: Noble Creek Tidelands Project | Driller: Western States Soil Conservation, Inc |
| Proj No. 014-21-006-2 | Date: 5/9/23 |
| Drilling Method: Mud Rotary | Elevation: 13' |
| Diameter: 4" / Water Table: N/A | Logged by: JLE |

| Sample No. | Sample Type | Recovery (%) | RQD (%) | Blow Count per 6 inches | Blows/Foot (N) | Water Table | Depth (ft BGS) | Graphic Log | | Materials Description | Moisture (%) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| S9 | | 100 | | 18-19-21 | 40 | | 35 | | | Grades to hard, moist, grey to dark grey to green, SILT (WEATHERED BEDROCK) | 53 | |
| S10 | | 100 | | 50 for 2" | >100 | | 40 | | **BR** | Soft to medium hard (R2 - R3) grey to dark grey SILTSTONE (BEDROCK) | | |
| S11 | | 100 | | 50 for 6" | >100 | | 45 | | | | | |
| S12 | | 100 | | 50 for 5" | >100 | | 50 | | | | | |
| | | | | | | | | | **END** | Boring completed at 51.5' BGS | | |
| | | | | | | | 55 | | | | | |
| | | | | | | | 60 | | | | | |
| | | | | | | | 65 | | | | | |

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Pali Consulting

# Noble Creek Tidelands Project
**Coos County, OR**

# B-3

| | |
|---|---|
| Project: Noble Creek Tidelands Project | Driller: Western States Soil Conservation, Inc |
| Proj No. 014-21-006-2 | Date: 5/9/23 |
| Drilling Method: Mud Rotary | Elevation: 7' |
| Diameter: 4" / Water Table: N/A | Logged by: JLE |

| Sample No. | Sample Type | Recovery (%) | RQD (%) | Blow Count per 6 inches | Blows/Foot (N) | Water Table | Depth (ft BGS) | Graphic Log | Materials Description | Moisture (%) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | GWT not encountered | 0 | CL | | | |
| S1 | | 67 | | 0-1-2 | 3 | | | | Soft, moist, grey-brown CLAY with minor orange mottles, much organic matter / charcoal (NATIVE) | 66 | |
| S2 | | 50 | | 0-0-1 | 1 | | 5 | | Grades to very soft, dark grey with minor orange mottles | | |
| S3 | | 75 | | | | | | SC | Very loose, moist to wet, dark grey CLAYEY SAND with organic matter / charcoal | 95 | |
| S4 | | 100 | | 0-0-0 | 0 | | 10 | | | 65 | |
| S5 | | 100 | | 0-0-0 | 0 | | 15 | CL | Very soft, moist to wet, grey CLAY with organic matter / charcoal. | 129 | |
| S6 | | 75 | | | | | | | | | |
| S7 | | 100 | | 0-0-0 | 0 | | 20 | CL-SC | Very loose, moist to wet, grey CLAYEY SAND with organic matter / charcoal grading to 6" of very soft, moist to wet, grey CLAY at bottom of sampler | 47/93 | MC=47% in sand portion of sample, 93% in clay portion of sample |
| | | | | | | | | END | Boring completed at 21.5' BGS | | |
| | | | | | | | 25 | | | | |
| | | | | | | | 30 | | | | |

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File: C:\Users\Jane\Pali Consulting Dropbox\1-Projects\Active-Projects\014-Waterways\014-21-006NobleCreek\02-GeotechnicalDesign\Analysis\NobleCreekLogs.log Date: 8/18/2023

Pali Consulting

# Noble Creek Tidelands Project
**Coos County, OR**

## B-4

| | |
|---|---|
| Project: Noble Creek Tidelands Project | Driller: Western States Soil Conservation, Inc |
| Proj No. 014-21-006-2 | Date: 5/9/23 |
| Drilling Method: Mud Rotary | Elevation: 10' |
| Diameter: 4" Water Table: N/A | Logged by: JLE |

| Sample No. | Sample Type | Recovery (%) | RQD (%) | Blow Count per 6 inches | Blows/Foot (N) | Water Table | Depth (ft BGS) | Graphic Log | | Materials Description | Moisture (%) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | GWT not encountered | 0 | | CH | | | |
| S1 | | 67 | | 0-1-1 | 2 | | | | | Soft, moist, varicolored (yellow/red/gray/brown) FAT CLAY with minor sand and organics (LEVEE FILL) | 54 | |
| | | | | | | | 5 | | | Grades to grey (NATIVE) | | |
| S2 | | 100 | | | | | | | | | | DD=33 pcf |
| S3 | | 50 | | 0-0-0 | 0 | | | | | Grades to very soft, moist to wet, grey with minor orange mottles. Increasing charcoal / organic material | | |
| | | | | | | | 10 | | | | | |
| S4 | | 50 | | | | | | | | | | |
| | | | | | | | 15 | | | | | |
| S5 | | 50 | | | | | | | | | | CS |
| | | | | | | | 20 | | | | | Drillers report much wood in cuttings |
| S6 | | 100 | | 0-0-0 | 0 | | | | | | 113 | AL, PI=98, LL=146, PL=48 |
| | | | | | | | 25 | | | Grades to dark grey | | |
| S7 | | 75 | | | | | | | | | | DD=22 pcf |
| | | | | | | | 30 | | OH | Very soft, moist, dark grey to brown to black, ORGANIC CLAY (much wood and organic material) | | |
| S8 | | 33 | | 0-0-1 | 1 | | | | | | | |

File: C:\Users\Jane\Pali Consulting Dropbox\1-Projects\Active-Projects\014-Waterways\014-21-006NobleCreek\02-GeotechnicalDesign\Analysis\NobleCreekLogs.log Date: 8/18/2023

Pali Consulting

# Noble Creek Tidelands Project
**Coos County, OR**

| | | |
|---|---|---|
| Project: Noble Creek Tidelands Project | Driller: Western States Soil Conservation, Inc | **B-4** |
| Proj No. 014-21-006-2 | Date: 5/9/23 | |
| Drilling Method: Mud Rotary | Elevation: 10' | |
| Diameter: 4" / Water Table: N/A | Logged by: JLE | |

| Sample No. | Sample Type | Recovery (%) | RQD (%) | Blow Count per 6 inches | Blows/Foot (N) | Water Table | Depth (ft BGS) | Graphic Log | Materials Description | Moisture (%) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|
| S9 | | 100 | | 0-0-0 | 0 | | 35 | CH | Very soft, moist to wet, grey FAT CLAY with minor charcoal, no organics | 55 | |
| S10 | | 75 | | 1-2-3 | 5 | | 40 | | Grades to medium stiff with minor rusty orange mottles | 56 | |
| S11 | | 100 | | 0-0-2 | 2 | | 45 | | Grades to soft, with minor organic material | | |
| S12 | | 50 | | 7-4-6 | 10 | | 50 | | Grades to stiff, grey-brown | 44 | |
| S13 | | 100 | | 5-9-12 | 21 | | 55 | | Grades to very stiff, dark grey, no organics, remnant platy rock structure (RESIDUAL SOIL) | | Drillers report increased difficulty drilling at 57' |
| S14 | | 100 | | 14-17-19 | 36 | | 60 | | Grades to extremely soft to very soft (R0-R1) sedimentary rock | | |
| | | | | | | | 65 | | Boring completed at 61.5' BGS | | |

Pali Consulting

# Noble Creek Tidelands Project

**Coos County, OR**

| | | | |
|---|---|---|---|
| Project: Noble Creek Tidelands Project | | Driller: Western States Soil Conservation, Inc | **B-5** |
| Proj No. 014-21-006-2 | | Date: 5/9/23 | |
| Drilling Method: Mud Rotary | | Elevation: 9' | |
| Diameter: 4" | Water Table: N/A | Logged by: JLE | |

| Sample No. | Sample Type | Recovery (%) | RQD (%) | Blow Count per 6 inches | Blows/Foot (N) | Water Table | Depth (ft BGS) | Graphic Log | | Materials Description | Moisture (%) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | 0 | | CL | | | |
| S1 | | 100 | | 0-0-0 | 0 | GWT not encountered | | | | Very soft, moist, varicolored (yellow/orange/gray/brown) SANDY CLAY with minor charcoal and organics (LEVEE FILL) Grades to brown-grey | | |
| | | | | | | | 5 | | | | | |
| S2 | | 100 | | | | | | | | | 53 | DD=66 pcf |
| S3 | | 67 | | 0-0-0 | 0 | | | | CH | Very soft, moist to wet, grey to brown to black FAT CLAY with much organic material, decreasing sand (NATIVE) | 103 | AL, PI=88, LL=140, PL=52 |
| | | | | | | | 10 | | | Grades to dark grey, moist to wet, with increasing sand | | |
| S4 | | 0 | | | | | | | | | | |
| S5 | | 100 | | | | | | | | | | |
| S6 | | 100 | | 0-0-1 | 1 | | 15 | | | | 76 | %F=53 |
| | | | | | | | 20 | | | Grades to grey. decreasing sand | | |
| S7 | | 100 | | | | | | | | | | |
| | | | | | | | 25 | | | Grades with increasing sand, wood, organics, and charcoal | | |
| S8 | | 100 | | 0-0-0 | 0 | | | | | | | |
| | | | | | | | 30 | | | Grades to brown-grey, with minor sand | | |
| S9 | | 100 | | 0-0-0 | 0 | | | | | | 115 | |

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Pali Consulting

# Noble Creek Tidelands Project
**Coos County, OR**

| | | |
|---|---|---|
| Project: Noble Creek Tidelands Project | Driller: Western States Soil Conservation, Inc | **B-5** |
| Proj No. 014-21-006-2 | Date: 5/9/23 | |
| Drilling Method: Mud Rotary | Elevation: 9' | |
| Diameter: 4" / Water Table: N/A | Logged by: JLE | |

| Sample No. | Sample Type | Recovery (%) | RQD (%) | Blow Count per 6 inches | Blows/Foot (N) | Water Table | Depth (ft BGS) | Graphic Log | | Materials Description | Moisture (%) | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| S10 | | 100 | | 0-0-0 | 0 | | 35 | | | Grades to light grey, with minor charcoal, no wood/organics or sand | | |
| | | | | | | | 40 | | | | | |
| S11 | | 100 | | | | | | | | | | DD=47 pcf |
| S12 | | 100 | | 0-0-0 | 0 | | 45 | | | Grades to grey, with increasing sand | 48 | %F=75 |
| | | | | | | | 50 | | | | | |
| S13 | | 100 | | 0-0-1 | 1 | | 55 | | | With minor sand | 42 | |
| | | | | | | | 60 | | | | | |
| S14 | | 35 | | 0-2-6 | 8 | | 65 | | **SC** | Grades to loose, moist to wet, grey CLAYEY SAND with large pieces of wood in sampler | | |
| | | | | | | | | | **END** | Boring completed at 66.5' BGS | | |

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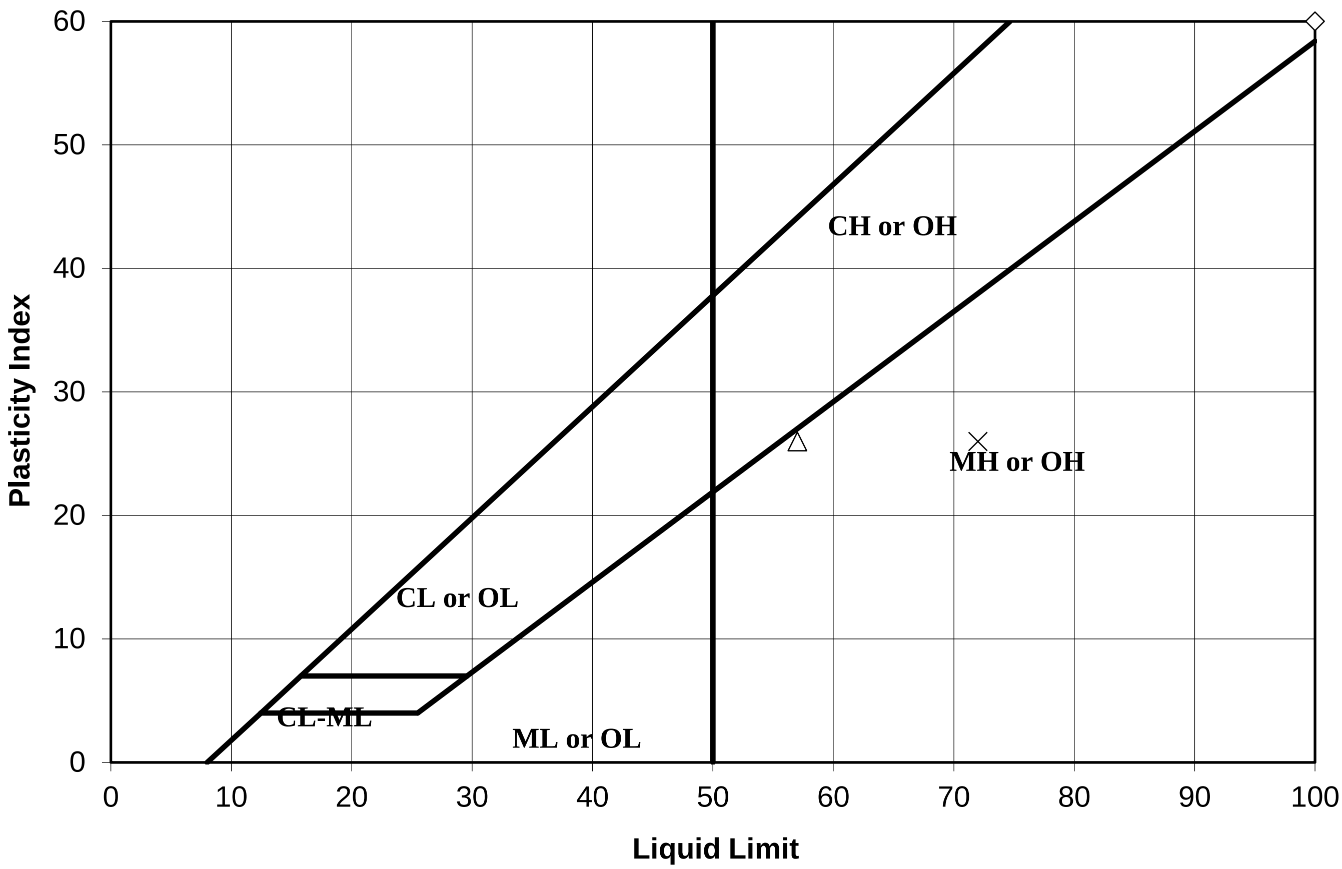



| Atterberg Limits Determination | | | | | | | |
|---|---|---|---|---|---|---|---|
| Symbol | Boring | Sample | Depth (ft) | Liquid Limit | Plastic Limit | PI | Classification |
| ◇ | B-1 | S-2 | 5 | 100 | 40 | 60 | CH |
| □ | B-1 | S-4 | 9.5 | 120 | 49 | 71 | MH |
| △ | B-1 | S-8 | 25 | 57 | 31 | 26 | MH |
| × | B-2 | S-2 | 7.5 | 72 | 46 | 26 | MH |

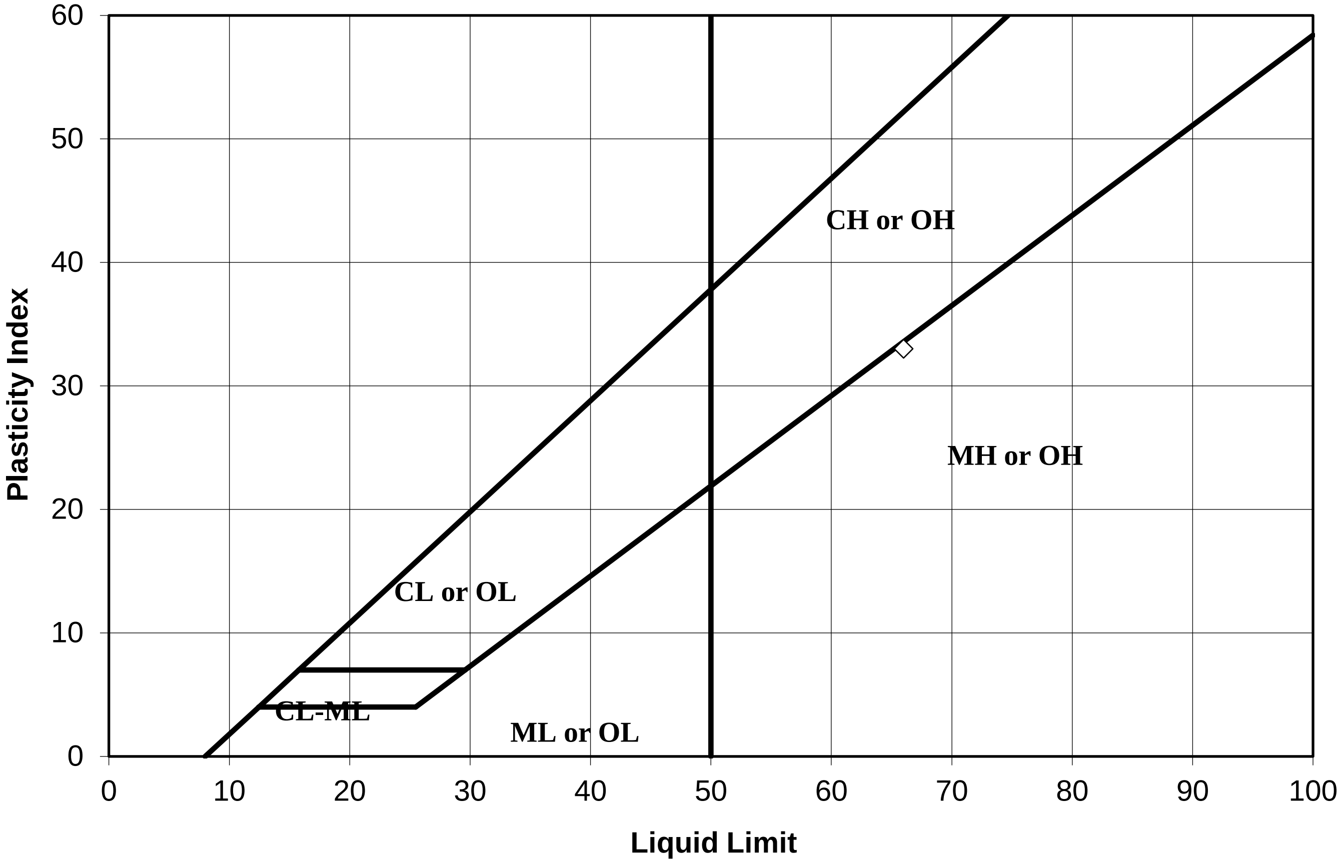


**ATTERBERG LIMITS - ASTM D4318**

**Noble Creek Tidelands Project**
**Project No.: 014-21-006-2**

Pali Consulting

**Figure B-1a**




| Atterberg Limits Determination | | | | | | | |
|---|---|---|---|---|---|---|---|
| Symbol | Boring | Sample | Depth (ft) | Liquid Limit | Plastic Limit | PI | Classification |
| ◇ | B-2 | S-6 | 20 | 66 | 33 | 33 | MH |
| □ | B-4 | S-6 | 20 | 146 | 48 | 98 | CH |
| △ | B-5 | S-3 | 7 | 140 | 52 | 88 | CH |
| ╳ | | | | | | | |



**ATTERBERG LIMITS - ASTM D4318**

**Noble Creek Tidelands Project**
**Project No.: 014-21-006-2**

Pali Consulting

**Figure B-1b**

Northwest Testing, Inc.
An RMA Company
9120 SW Pioneer Court, Suite B, Wilsonville, Oregon 97070 | ph: 503.682.1880 | fax: 503.682.2753 | www.nwgeotech.com

# TECHNICAL REPORT

| | | | |
|---|---|---|---|
| **Report To:** | Tim Blackwood, PE, GE, CEG<br>Pali Consulting, Inc.<br>1120 SW Fifth Avenue, Suite 1302<br>Portland, Oregon 97204 | **Date:**<br><br>**Lab No.:** | 6/16/2023<br><br>23-095 |
| **Project:** | Noble Creek | **Project No.:** | 00-223425-1 |

**Report of:** One-Dimensional Consolidation testing of soil.

**Sample Identification**

As requested, NTI provided one-dimensional consolidation testing of soil on a tube sample delivered to our laboratory by a Pali Consulting, Inc. representative on May 15, 2023. Testing was performed in general accordance with the standard indicated. Our laboratory test results are summarized on the following table and pages.

## Laboratory Testing

**Sample ID: B-1, S-5 @ 11.0 – 13.0 Ft.**

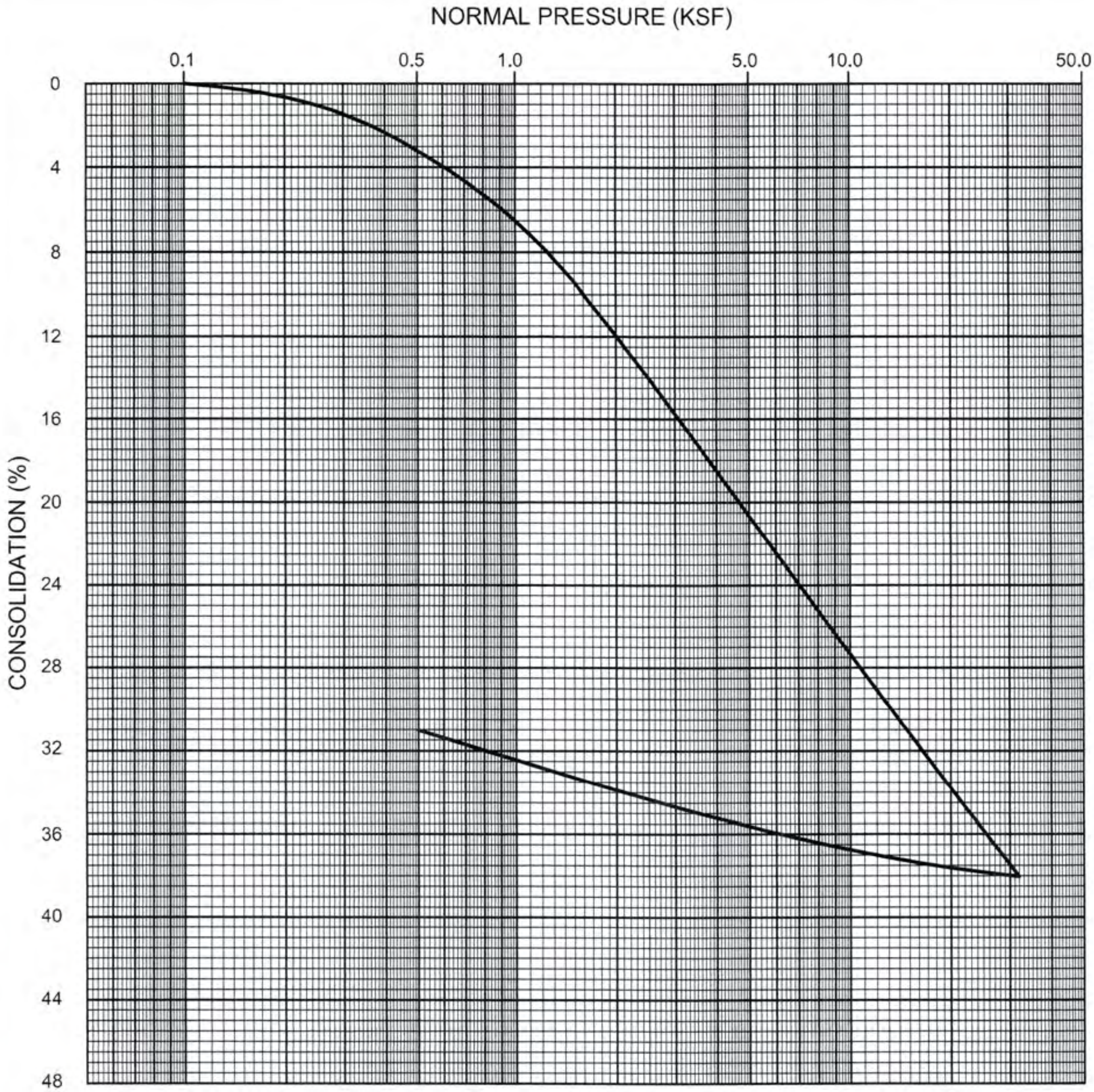
| One Dimensional Consolidation of Soil (ASTM D2435) | | | |
|---|---|---|---|
| **Test** | **Initial Conditions** | **Final Conditions** | |
| Moisture Content, (%) | 83.2 | 54.3 | |
| Dry Unit Weight, (pcf) | 51.4 | 66.2 | |
| Height of Specimen, (inches) | 1.0000 | 0.6898 | |
| **Load (ksf)** | **Dial Reading (inches)** | **Load (ksf)** | **Dial Reading (inches)** |
| Initial | 0.0000 | 8.0 | 0.2523 |
| 0.25 | 0.0102 | 16.0 | 0.3185 |
| 0.5 | 0.308 | 32.0 | 0.3802 |
| 1.0 | 0.0651 | 8.0 | 0.3639 |
| 2.0 | 0.1187 | 2.0 | 0.3389 |
| 4.0 | .01847 | 0.5 | 0.3102 |

**Attachments:** Laboratory Test Results – One-Dimensional Consolidation

**Copies:** (1) Addressee
(1) Jane Eisenberg, Pali Consulting, Inc.



REVIEWED BY: Michael A. Ginsbach MAG




| SYMBOL | SAMPLE LOCATION | NORMAL PRESSURE AT SATURATION (KSF) |
|---|---|---|
| ———— | B-1, S-5 @ 11.0 - 13.0 FT. | 0.25 |

# CONSOLIDATION TEST RESULTS - ASTM D2435

| PROJECT NO. 00-223425-1 | PALI CONSULTING, INC.<br>NOBLE CREEK | LAB NO. 23-095 |
|---|---|---|

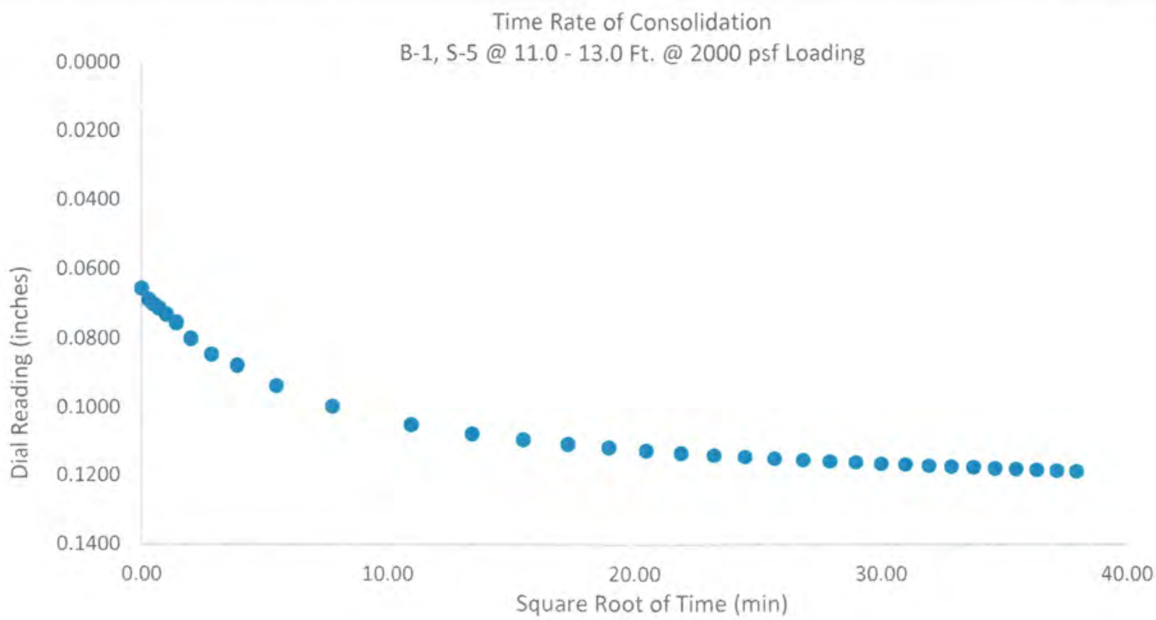


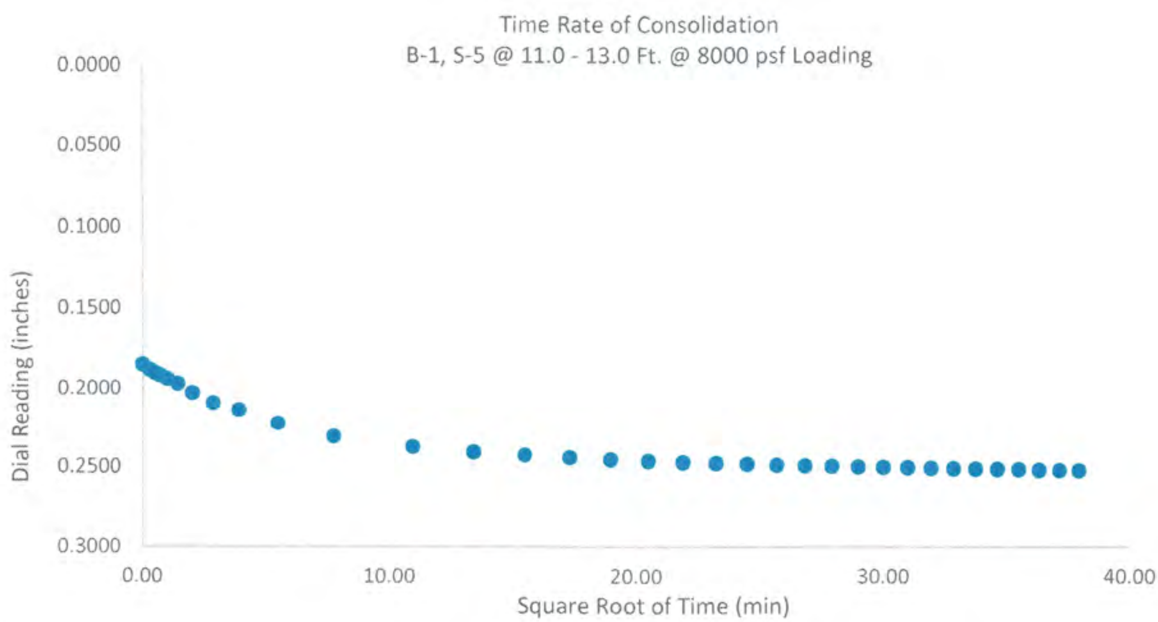
Northwest Testing, Inc.
An RMA Company

Northwest Testing, Inc.
An RMA Company
9120 SW Pioneer Court, Suite B, Wilsonville, Oregon 97070 | ph: 503.682.1880 fax: 503.682.2753 | www.nwgeotech.com

# TECHNICAL REPORT

| | | | |
|---|---|---|---|
| **Report To:** | Tim Blackwood, PE, GE, CEG<br>Pali Consulting, Inc.<br>1120 SW Fifth Avenue, Suite 1302<br>Portland, Oregon 97204 | **Date:** | 6/16/2023 |
| | | **Lab No.:** | 23-095 |
| **Project:** | Noble Creek | **Project No.:** | 00-223425-1 |





 REVIEWED BY: Michael A. Ginsbach

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# TECHNICAL REPORT

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|---|---|---|---|
| **Report To:** | Tim Blackwood, PE, GE, CEG<br>Pali Consulting, Inc.<br>1120 SW Fifth Avenue, Suite 1302<br>Portland, Oregon 97204 | **Date:**<br>**Lab No.:** | 7/10/2023<br>23-129 |
| **Project:** | Noble Creek | **Project No.:** | 00-223425-1 |
| **Report of:** | One-Dimensional Consolidation testing of soil. | | |

**Sample Identification**

As requested, NTI provided one-dimensional consolidation testing of soil on a tube sample delivered to our laboratory by a Pali Consulting, Inc. representative on June 16, 2023. Testing was performed in general accordance with the standard indicated. Our laboratory test results are summarized on the following table and pages.

## Laboratory Testing

**Sample ID: B-4, S-5 @ 15.0 – 17.0 Ft.**

| One Dimensional Consolidation of Soil (ASTM D2435) | | |
|---|---|---|
| **Test** | **Initial Conditions** | **Final Conditions** |
| Moisture Content, (%) | 82.6 | 45.6 |
| Dry Unit Weight, (pcf) | 49.6 | 75.2 |
| Height of Specimen, (inches) | 1.0000 | 0.6625 |

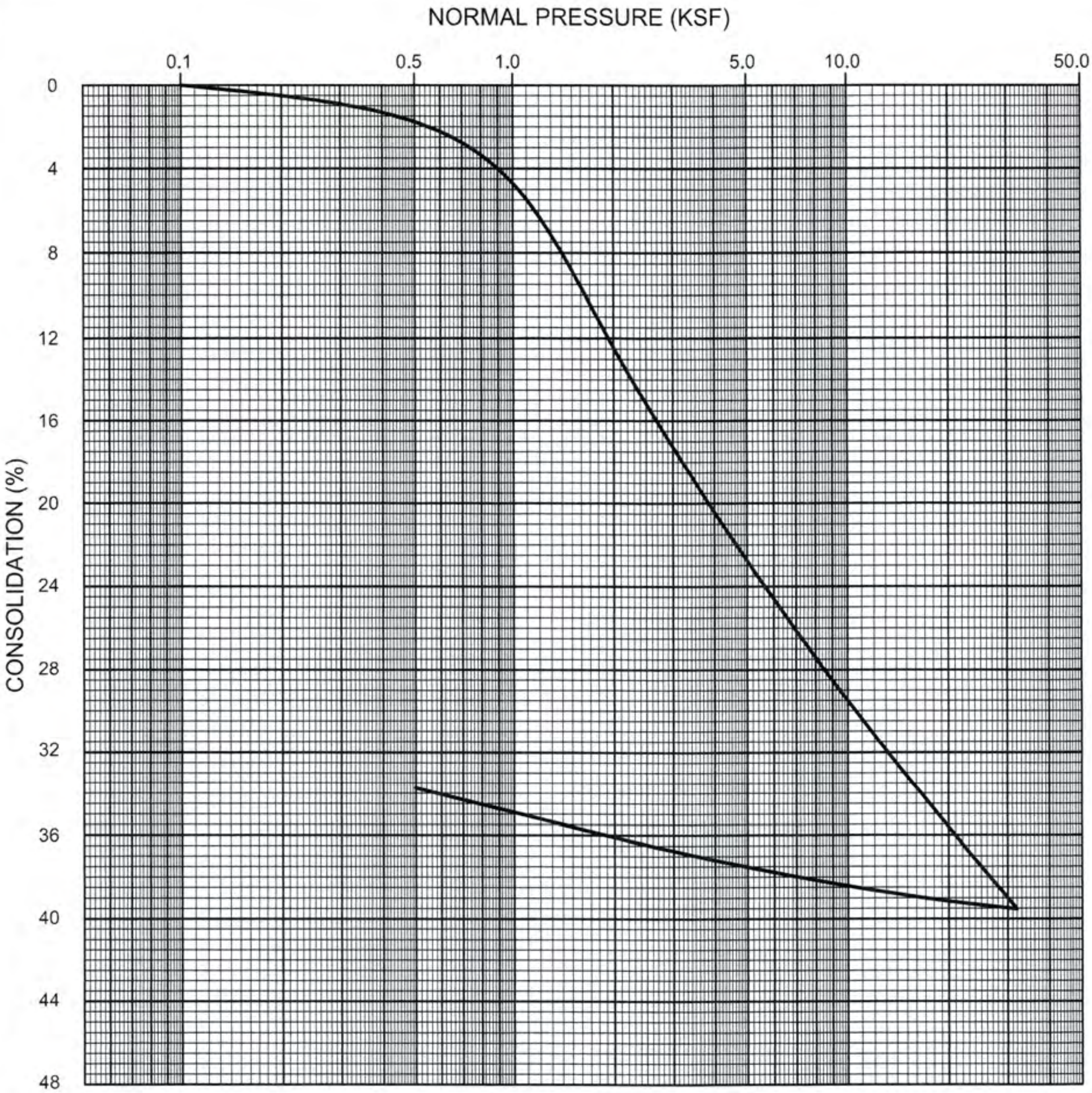
| Load (ksf) | Dial Reading (inches) | Load (ksf) | Dial Reading (inches) |
|---|---|---|---|
| Initial | 0.0000 | 8.0 | 0.2754 |
| 0.25 | 0.0062 | 16.0 | 0.3385 |
| 0.5 | 0.0179 | 32.0 | 0.3953 |
| 1.0 | 0.0465 | 8.0 | 0.3824 |
| 2.0 | 0.1272 | 2.0 | 0.3614 |
| 4.0 | 0.2056 | 0.5 | 0.3375 |

**Attachments:** Laboratory Test Results – One-Dimensional Consolidation

**Copies:** (1) Addressee
(1) Jane Eisenberg, Pali Consulting, Inc.



REVIEWED BY: Michael A. Ginsbach MG




| SYMBOL | SAMPLE LOCATION | NORMAL PRESSURE AT SATURATION (KSF) |
|---|---|---|
| ——— | B-4, S-5 @ 15.0-17.0 FT. | 0.25 |

# CONSOLIDATION TEST RESULTS - ASTM D2435

| PROJECT NO. 00-223425-0 | PALI CONSULTING, INC.<br>LABORATORY TESTING | LAB NO. 23-129 |
|---|---|---|



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# TECHNICAL REPORT

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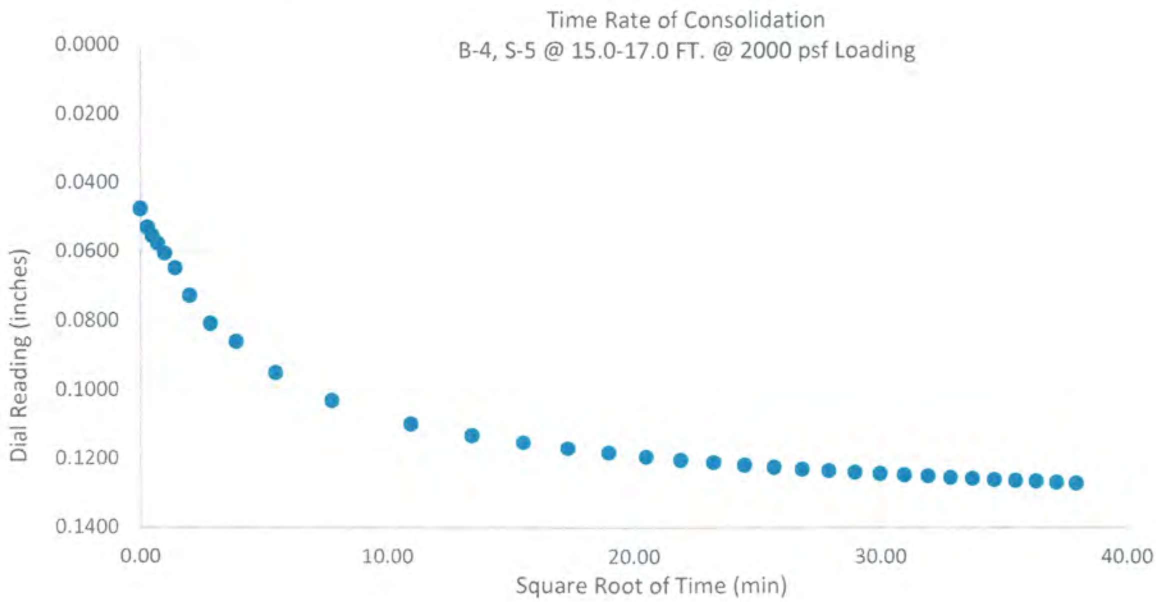
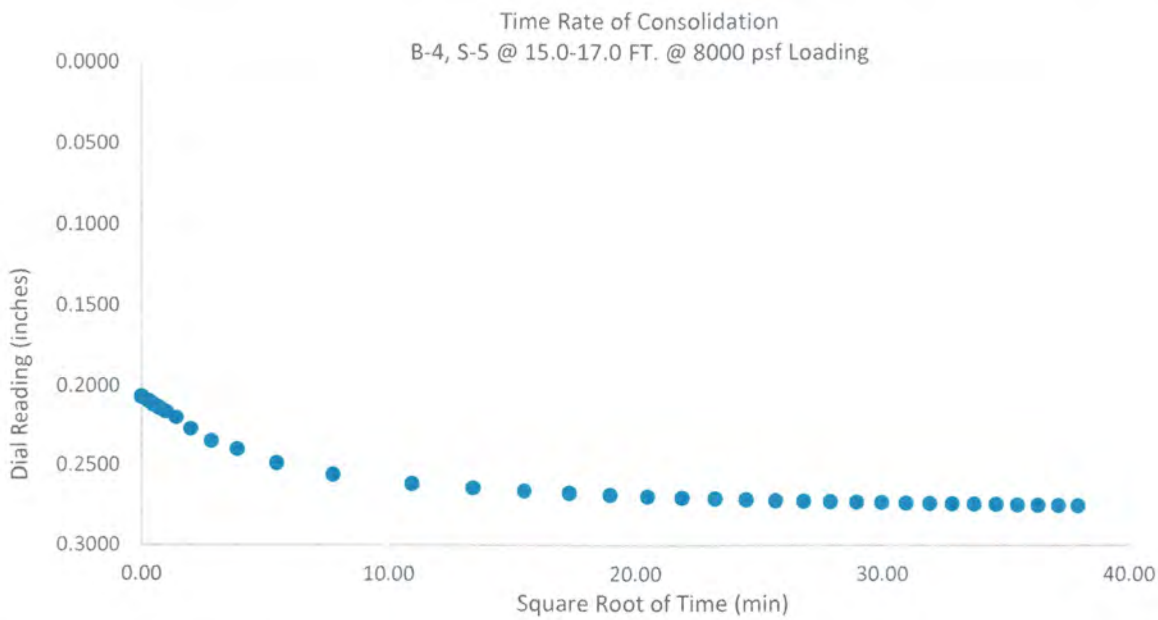
| **Report To:** | Tim Blackwood, PE, GE, CEG<br>Pali Consulting, Inc.<br>1120 SW Fifth Avenue, Suite 1302<br>Portland, Oregon 97204 | **Date:** | 7/10/2023 |
| | | **Lab No.:** | 23-129 |
| **Project:** | Noble Creek | **Project No.:** | 00-223425-1 |

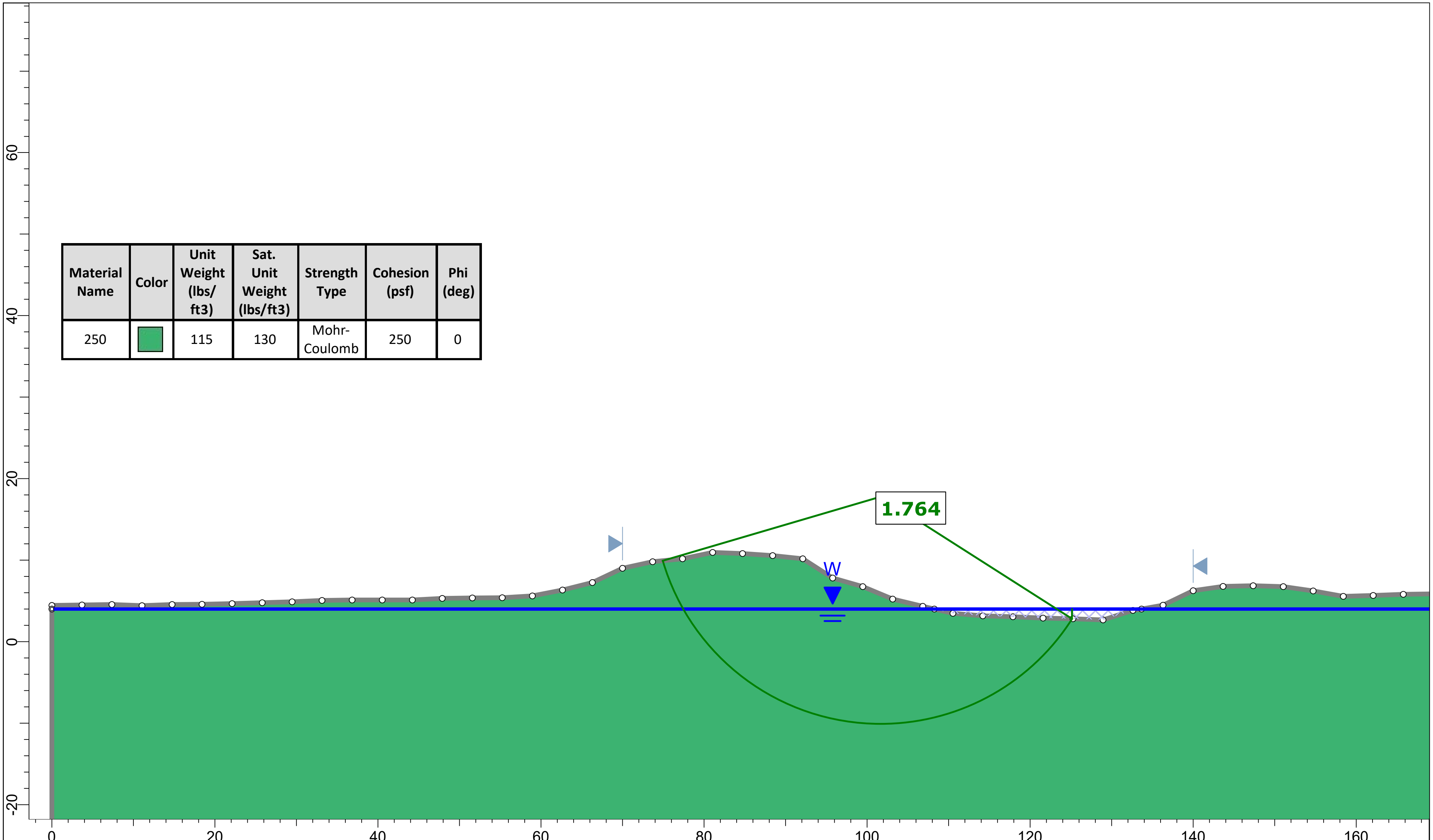




 REVIEWED BY: Michael A. Ginsbach

# APPENDIX C -
## SLOPE STABILITY ANALYSES




| Material Name | Color | Unit Weight (lbs/ ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) |
|---|---|---|---|---|---|---|
| 250 | | 115 | 130 | Mohr-Coulomb | 250 | 0 |

Noble Creek Levee
Slope Stability Analysis

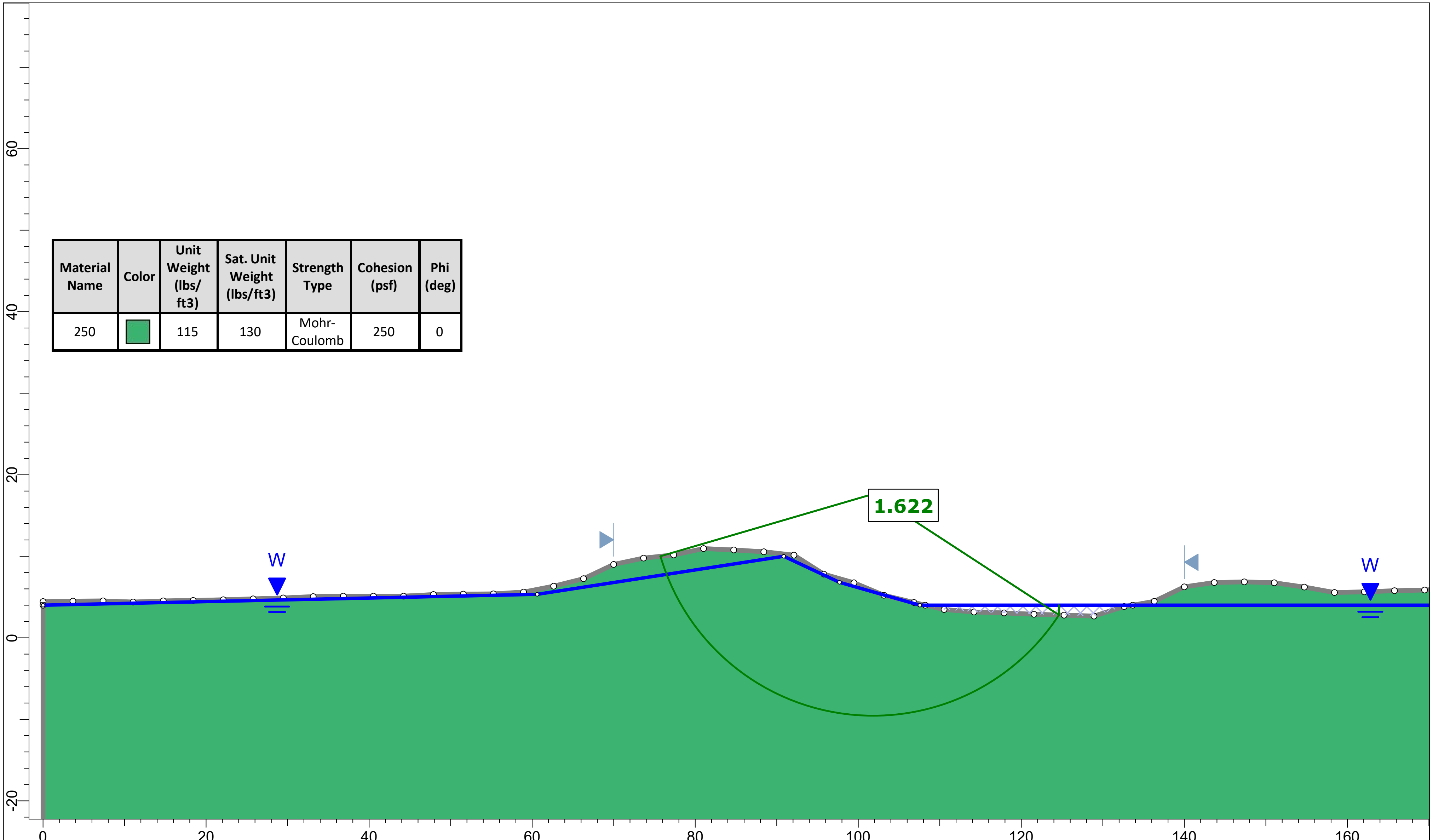
**1) Existing Conditions**

Static

Scale 1:200 | 8/20/2023

Pali Consulting

Figure
C-1C-2




| Material Name | Color | Unit Weight (lbs/ ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) |
|---|---|---|---|---|---|---|
| 250 | | 115 | 130 | Mohr-Coulomb | 250 | 0 |

Noble Creek Levee
Slope Stability Analysis

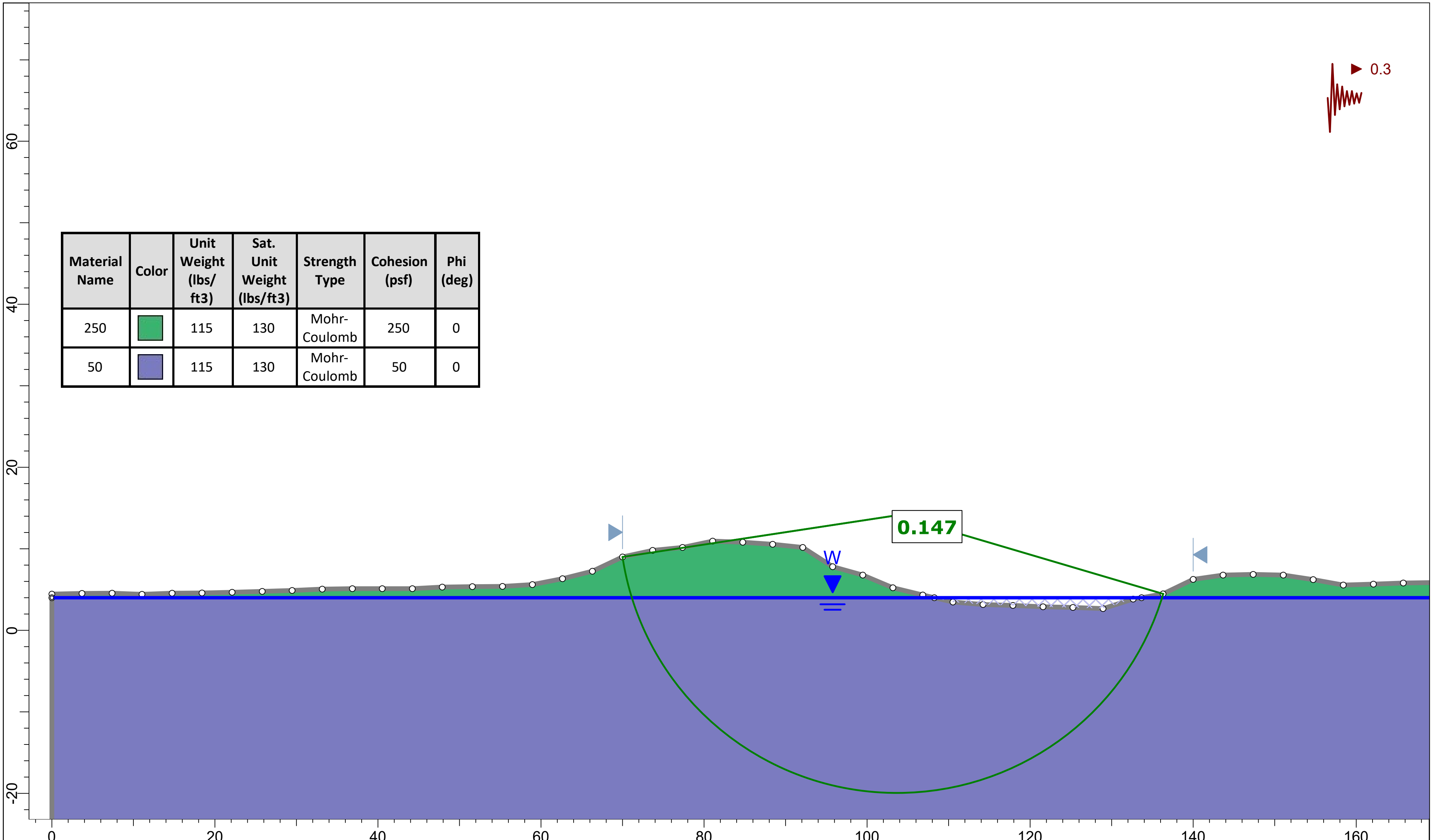
**1) Existing Conditions**

Static 10' WL

Scale 1:200 | 8/20/2023

Pali Consulting

Figure
C-2




| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) |
|---|---|---|---|---|---|---|
| 250 | | 115 | 130 | Mohr-Coulomb | 250 | 0 |
| 50 | | 115 | 130 | Mohr-Coulomb | 50 | 0 |

Noble Creek Levee
Slope Stability Analysis

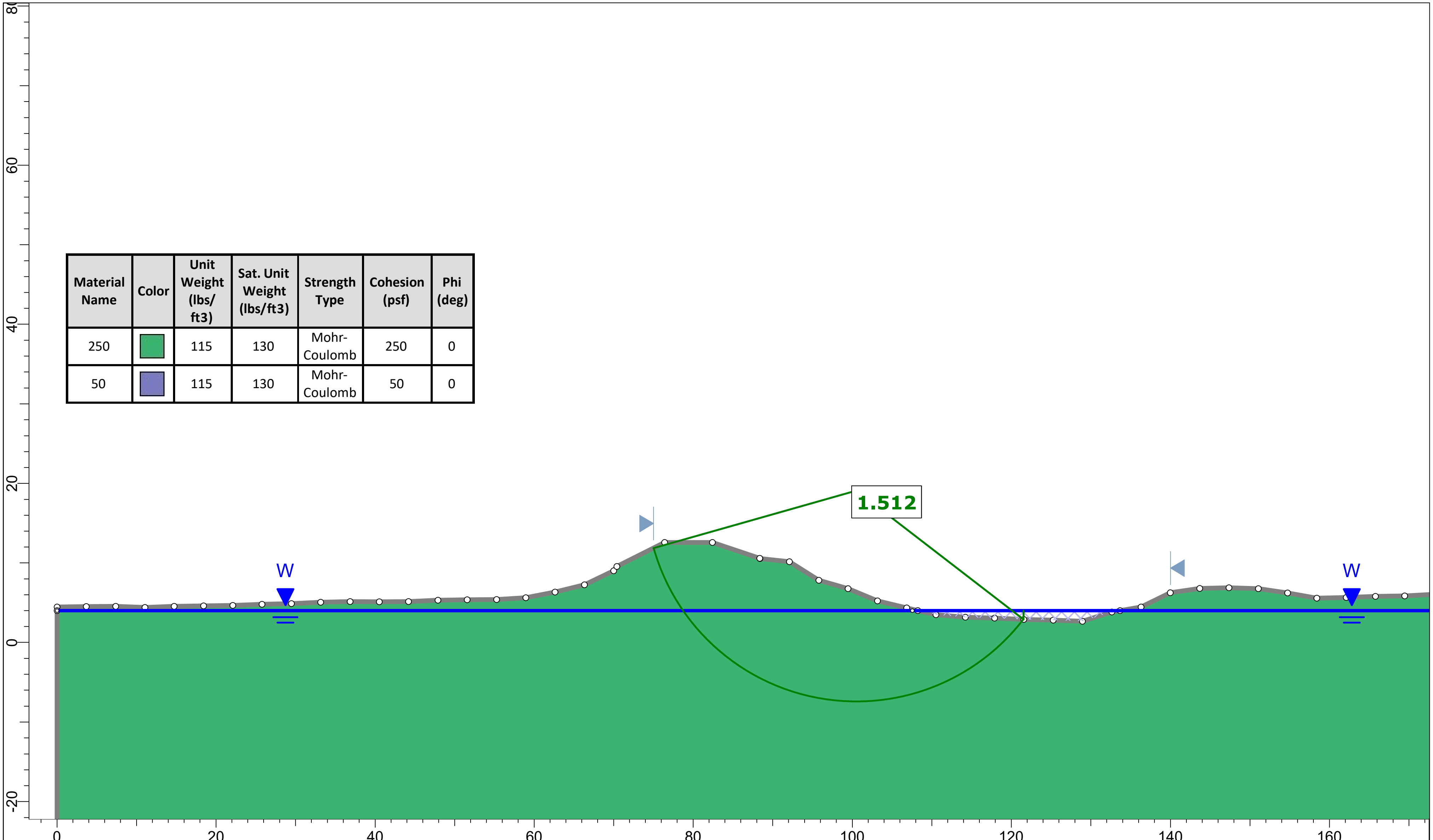
**1) Existing Conditions**

Seismic

Scale 1:200 | 8/20/2023

Pali Consulting

Figure
C-3




| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) |
|---|---|---|---|---|---|---|
| 250 | | 115 | 130 | Mohr-Coulomb | 250 | 0 |
| 50 | | 115 | 130 | Mohr-Coulomb | 50 | 0 |

Noble Creek Levee
Slope Stability Analysis

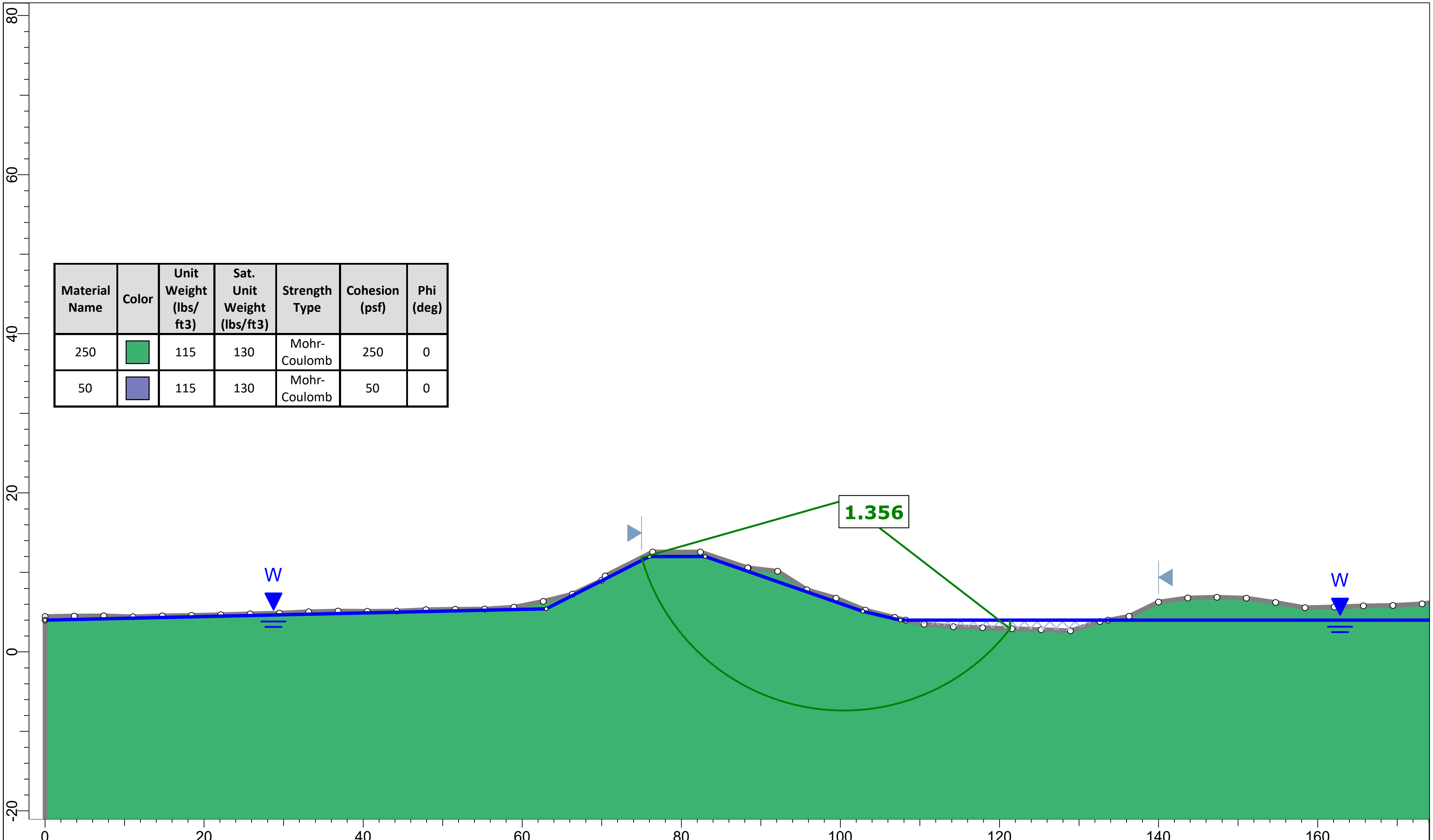
**2) 6' Crest**

Static

Scale 1:205 | 8/20/2023

Pali Consulting

Figure
C-4




| Material Name | Color | Unit Weight (lbs/ ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) |
|---|---|---|---|---|---|---|
| 250 | | 115 | 130 | Mohr-Coulomb | 250 | 0 |
| 50 | | 115 | 130 | Mohr-Coulomb | 50 | 0 |

Noble Creek Levee
Slope Stability Analysis

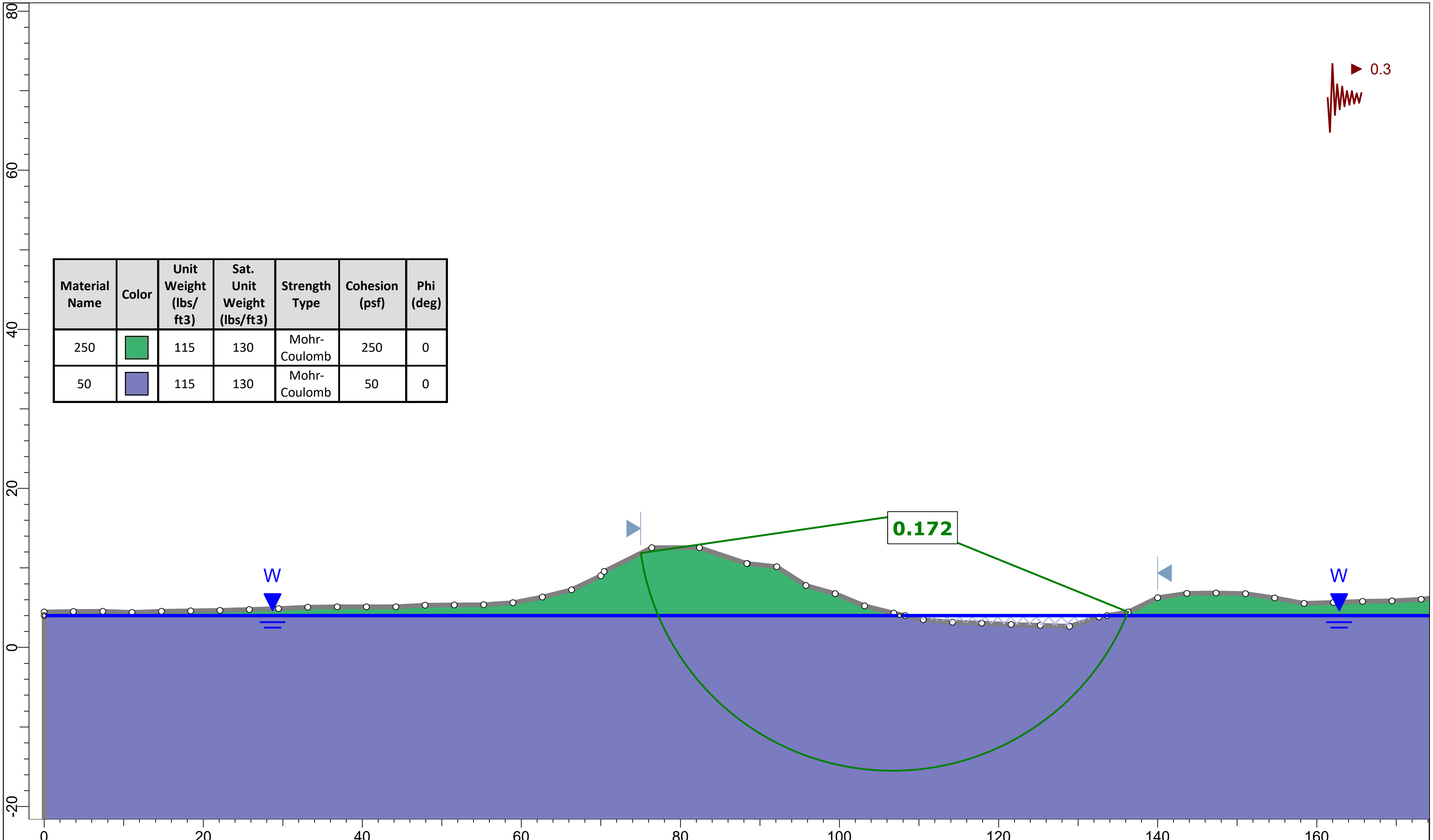
**2) 6' Crest**

Static 12' WL

Scale 1:205 | 8/20/2023

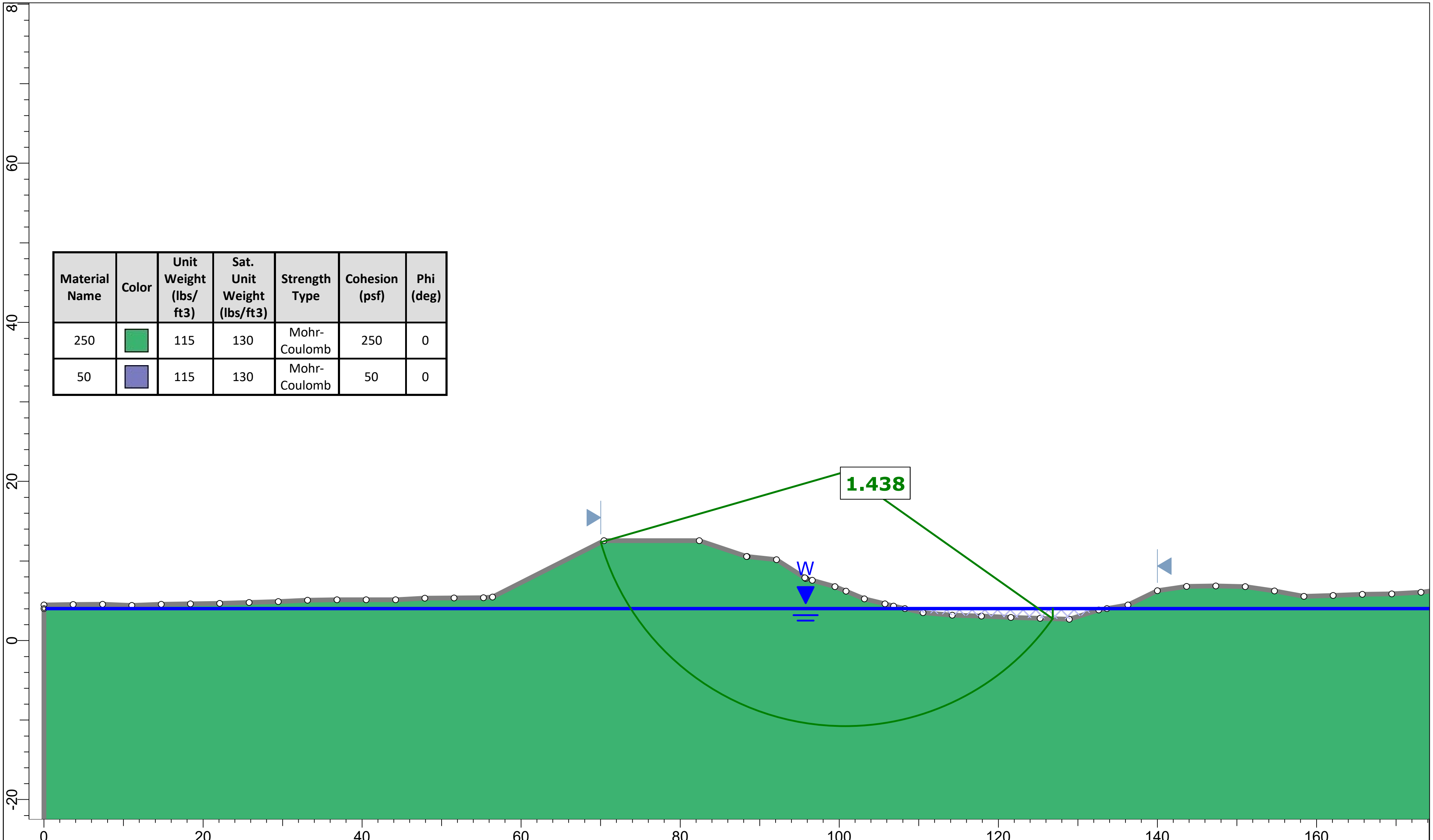
Pali Consulting

Figure
C-5




| Material Name | Color | Unit Weight (lbs/ ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) |
|---|---|---|---|---|---|---|
| 250 | | 115 | 130 | Mohr-Coulomb | 250 | 0 |
| 50 | | 115 | 130 | Mohr-Coulomb | 50 | 0 |

| | |
|---|---|
| Noble Creek Levee<br>Slope Stability Analysis | |
| **2) 6' Crest**<br>Seismic | |
| Scale 1:205 | 8/20/2023 |
| Pali Consulting | Figure<br>C-6 |




| Material Name | Color | Unit Weight (lbs/ ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) |
|---|---|---|---|---|---|---|
| 250 | | 115 | 130 | Mohr-Coulomb | 250 | 0 |
| 50 | | 115 | 130 | Mohr-Coulomb | 50 | 0 |

Noble Creek Levee
Slope Stability Analysis

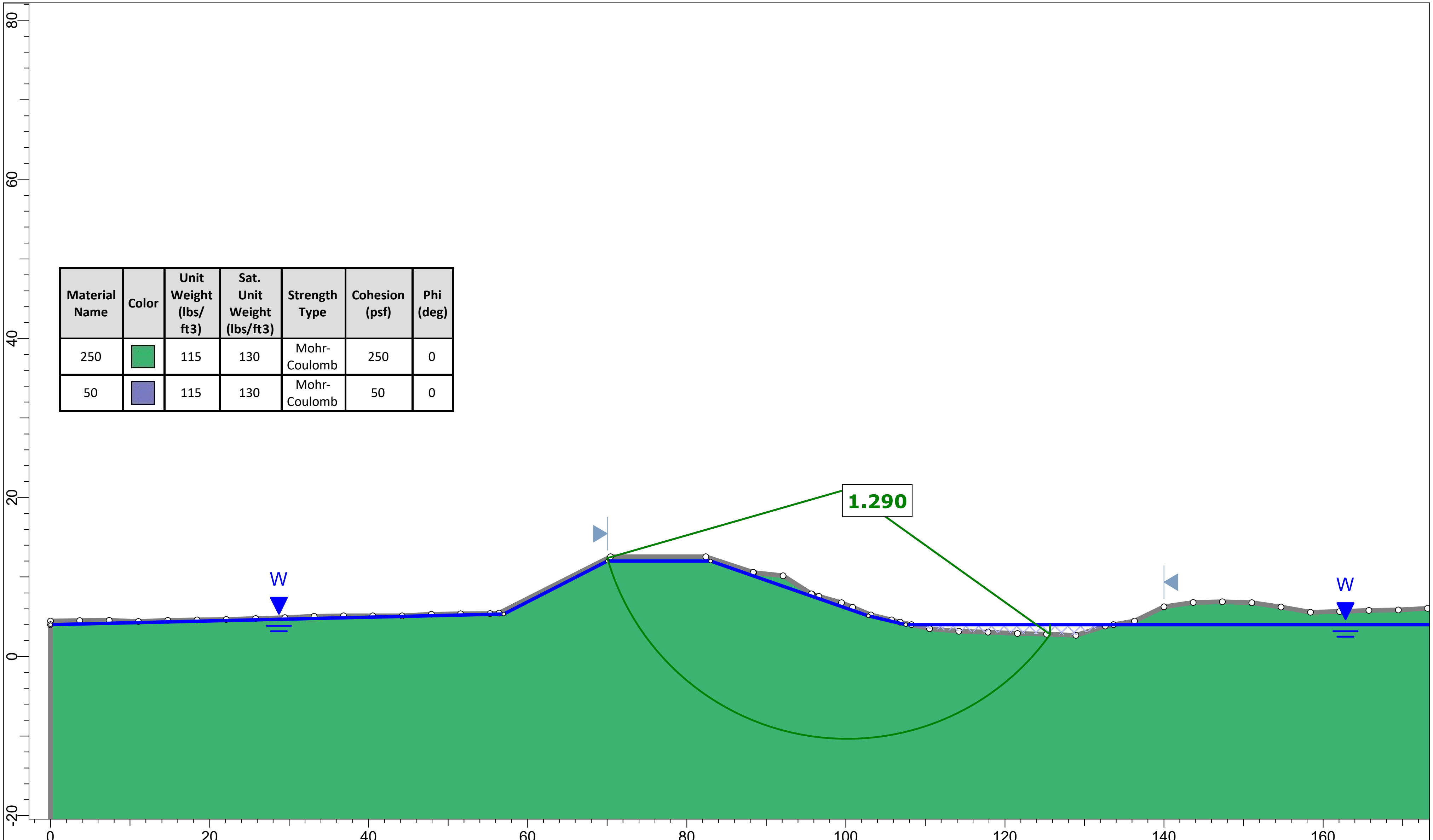
**3) 12' Crest**

Static

Scale 1:205 | 8/20/2023

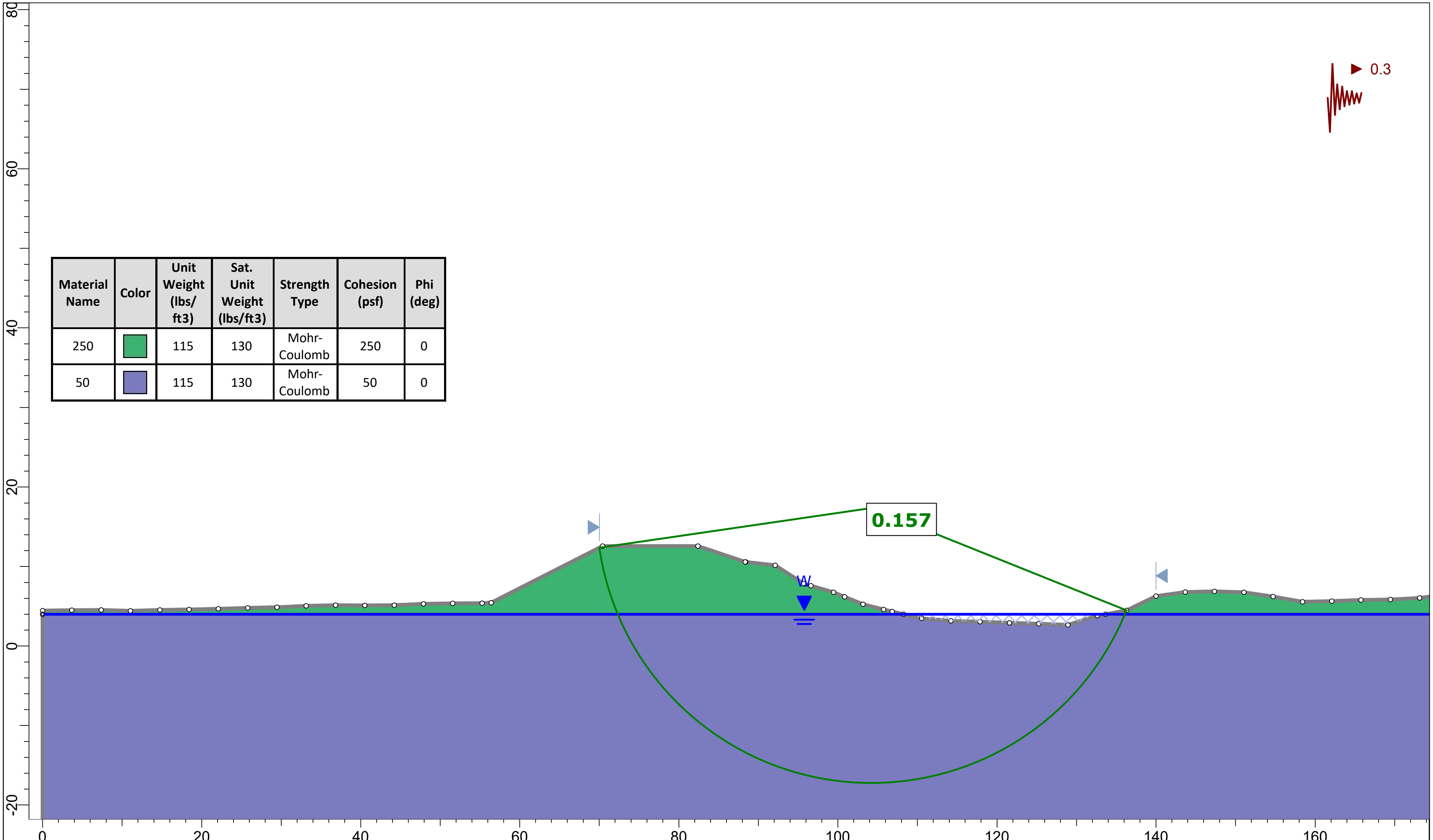
Pali Consulting

Figure C-7




| Material Name | Color | Unit Weight (lbs/ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) |
|---|---|---|---|---|---|---|
| 250 | Green | 115 | 130 | Mohr-Coulomb | 250 | 0 |
| 50 | Purple | 115 | 130 | Mohr-Coulomb | 50 | 0 |

| | |
|---|---|
| Noble Creek Levee<br>Slope Stability Analysis | |
| **3) 12' Crest**<br>Static 12' WL | |
| Scale 1:205 | 8/20/2023 |
| Pali Consulting | Figure<br>C-8 |




| Material Name | Color | Unit Weight (lbs/ ft3) | Sat. Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) |
|---|---|---|---|---|---|---|
| 250 | | 115 | 130 | Mohr-Coulomb | 250 | 0 |
| 50 | | 115 | 130 | Mohr-Coulomb | 50 | 0 |

Noble Creek Levee
Slope Stability Analysis

**3) 12' Crest**

Seismic

Scale 1:205 | 8/20/2023

Pali Consulting

Figure
C-9