

NOBLE CREEK TIDELANDS RESTORATION PROJECT

EXISTING CONDITIONS, ALTERNATIVES, AND 60% BASIS OF DESIGN REPORT



prepared for
Coos Soil and Water Conservation District



**Coos Soil and Water
Conservation District**

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1.0 INTRODUCTION AND NEED FOR PROJECT

The Noble Creek Tidelands Project, a multi-benefit collaborative effort led by Coos Soil and Water Conservation District (CSWCD) and Oregon Department of Fish and Wildlife (ODFW), aims to provide salmon habitat and community benefits in the unincorporated community of Green Acres, Oregon (**Figure 1**). Isthmus Slough, the largest tidal inlet of Coos Bay, ends at the confluence with Noble Creek.

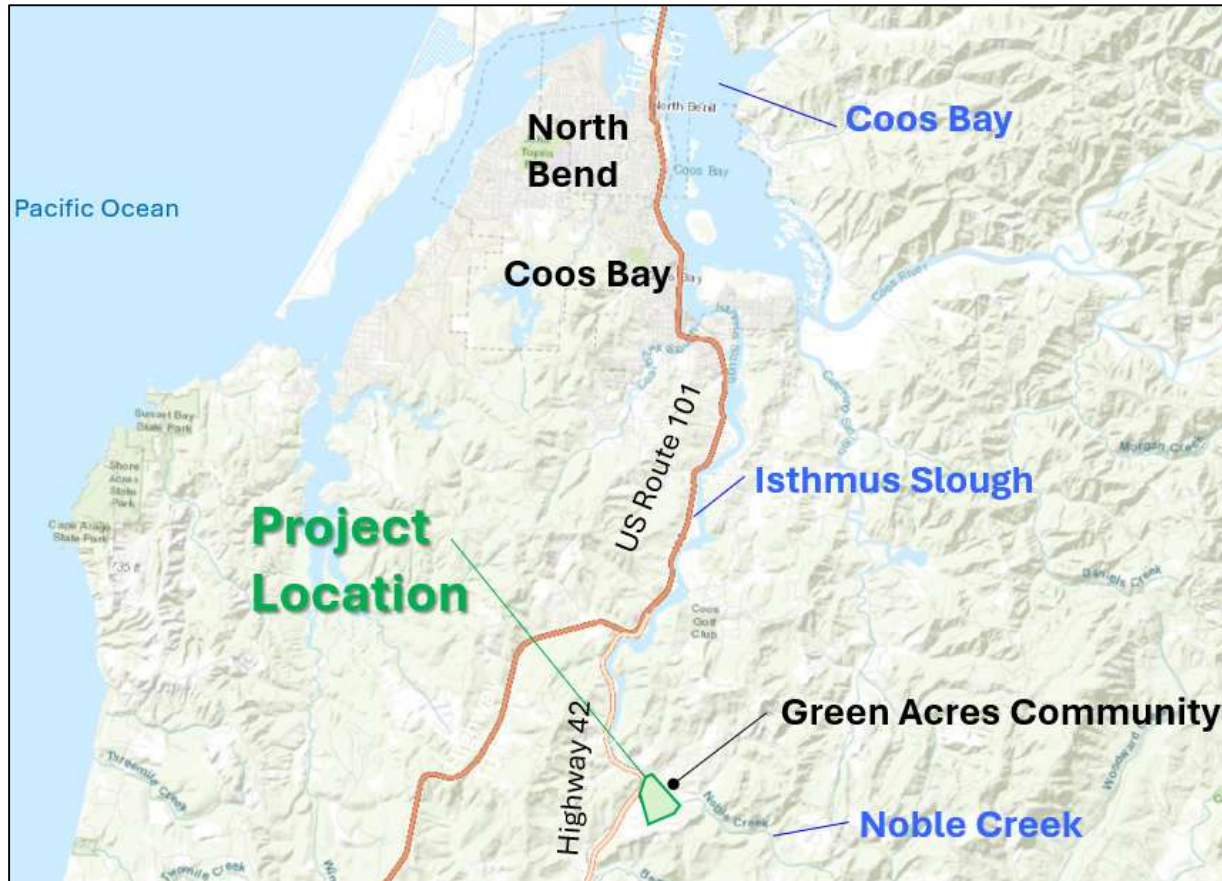


Figure 1. Location of Project Site South of Coos Bay

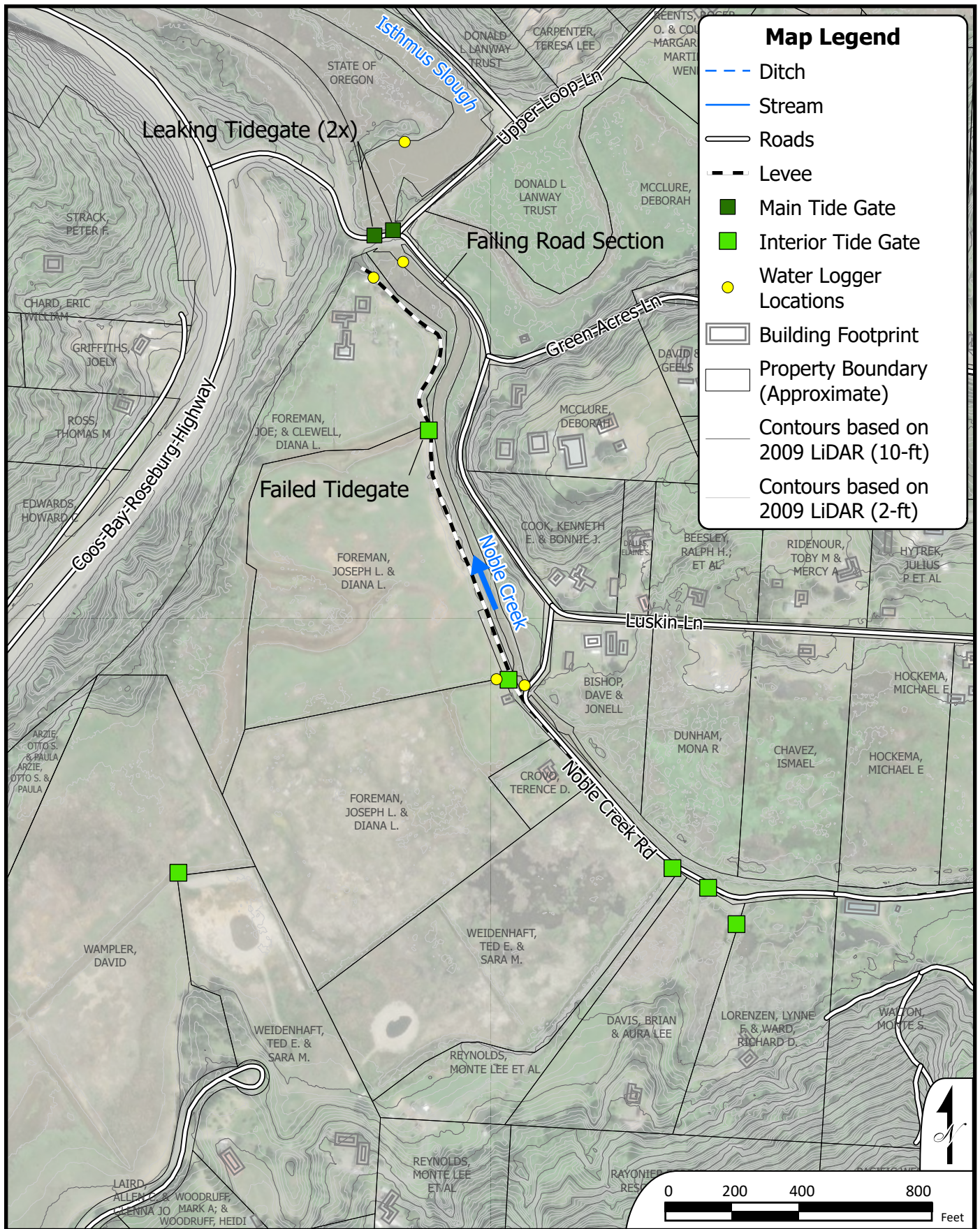
Two aging wooden culverts with leaking tide gates separate Isthmus Slough from Noble Creek, and partially reduce flooding of a low tidal flood basin (**Figure 2**). The two culverts and tide gates are directly under Green Acres Lane, a county road that is the only entrance and exit to the Green Acres community. The roadway prism of Green Acres Lane is eroding around the failing tide gates and in at least one other location along Noble Creek. Both the Green Acres community members and the Coos County Roadmaster recognize that the Green Acres Lane crossing must be repaired or replaced to prevent a future failure that could result in an unplanned loss of access to the community.

In addition to being a local community hazard, the failing tide gates at the crossing over Noble Creek also prevent salmon from accessing the large, lowland tidal floodplain at the entrance to the Green Acres community (**Figure 2**), referred to as the Noble Creek Tidelands. Loss of access to floodplain tidal wetlands is a primary factor inhibiting salmon recovery in the Coos Bay watershed. One of the primary

restoration objectives in the Coos Basin Coho Partnership’s Strategic Action Plan (CBCP, 2022) is to “reconnect tidal wetland and slough habitats (permanent and seasonal) in estuary and lower mainstems”. By 2045, the CBCP aims to restore 93.4 acres of disconnected fresh and salt marsh to tidal reconnection; restoring fish access to and enhancing a large part of the approximately 100-acre Noble Creek Tidelands would represent a significant step towards meeting this.

CSWCD and ODFW recognized the opportunity to develop a project that would simultaneously address the infrastructure concerns and provide significant habitat uplift. Since 2020, CSWCD worked to secure funding for the Noble Creek Tidelands project and oversaw the early phases of the work. The work to date on this project included community outreach, bathymetric and topographic survey, development and calibration of a complex hydraulic model, geotechnical and structural engineering, alternatives analysis, selection of a preferred alternative, and development of conceptual level (30%) design drawings. This report summarizes the findings of the work done to date, the alternatives considered, and the basis for the conceptual-level designs.

The 60% design drawings are provided as **Appendix A** of this report.



Existing Conditions Map

Nobble Creek
Tidelands Restoration
Project



**FIGURE
2**

2.0 EXISTING CONDITIONS

2.1 PROJECT SITE OVERVIEW

2.1.1 Green Acres Lane Crossing

The crux of the project area is the crossing of Green Acres Lane (GAL) over Noble Creek (**Figure 3**). The GAL crossing marks the upstream (southernmost) extent of Isthmus Slough. Noble Creek is connected to Isthmus Slough through two aging culverts under GAL.

Both culverts have tide gates on the northern (downstream) side of GAL, which are meant to reduce tidal inflows from the north while allowing Noble Creek flow to exit southward towards Isthmus Slough (**Figure 3**). The eastern culvert is a 3-foot X 9-foot wooden box culvert with a wooden tide gate attached by chains from the top; and the western culvert is a 6-foot diameter round metal culvert with a wooden top-hinged tide gate (**Figure 4**). Leakage through both tide gates results in a dampened tidal signal on the Noble Creek side of the GAL crossing. The leaking tide gates, combined with a deteriorated levee along the west side of Noble Creek, together prevent most flooding at Diana Foreman's house, which is built on the former tidal floodplain surface at 3 to 4 feet above sea level (NAVD 88). The current tide gates and levee are unreliable and could fail in a future high flow and/or tidal event, putting this house at an imminent risk of flooding.

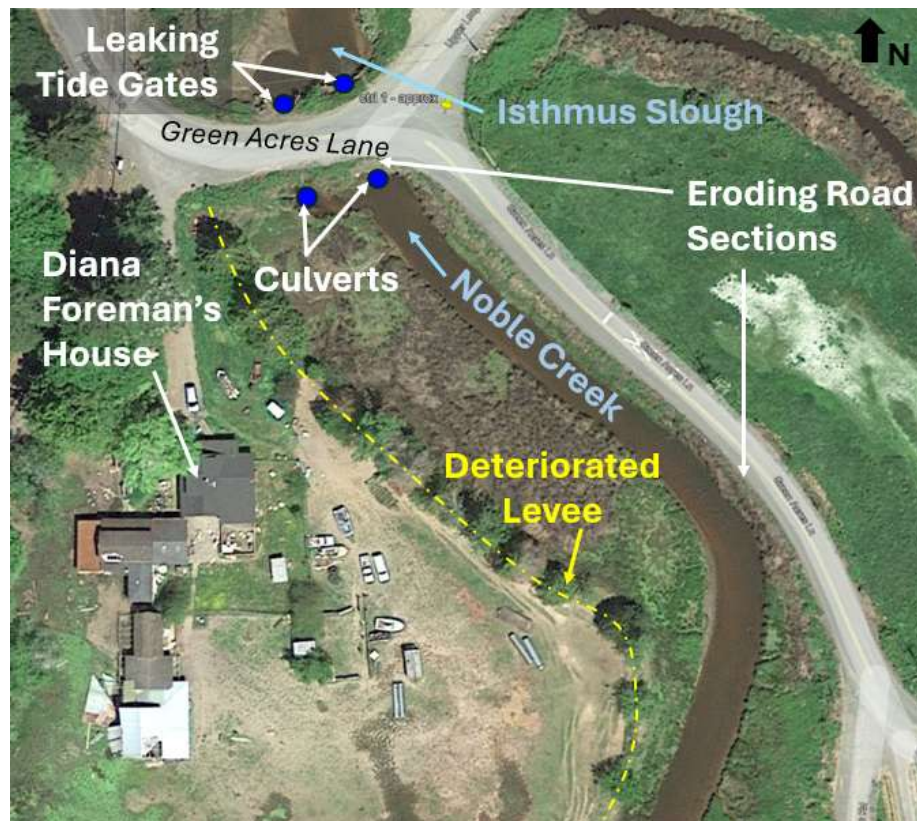


Figure 3. Annotated Air Photo of Green Acres Lane Crossing Over Noble Creek



Figure 4. Photo Looking Upstream (South) at Greenacres Lane Crossing

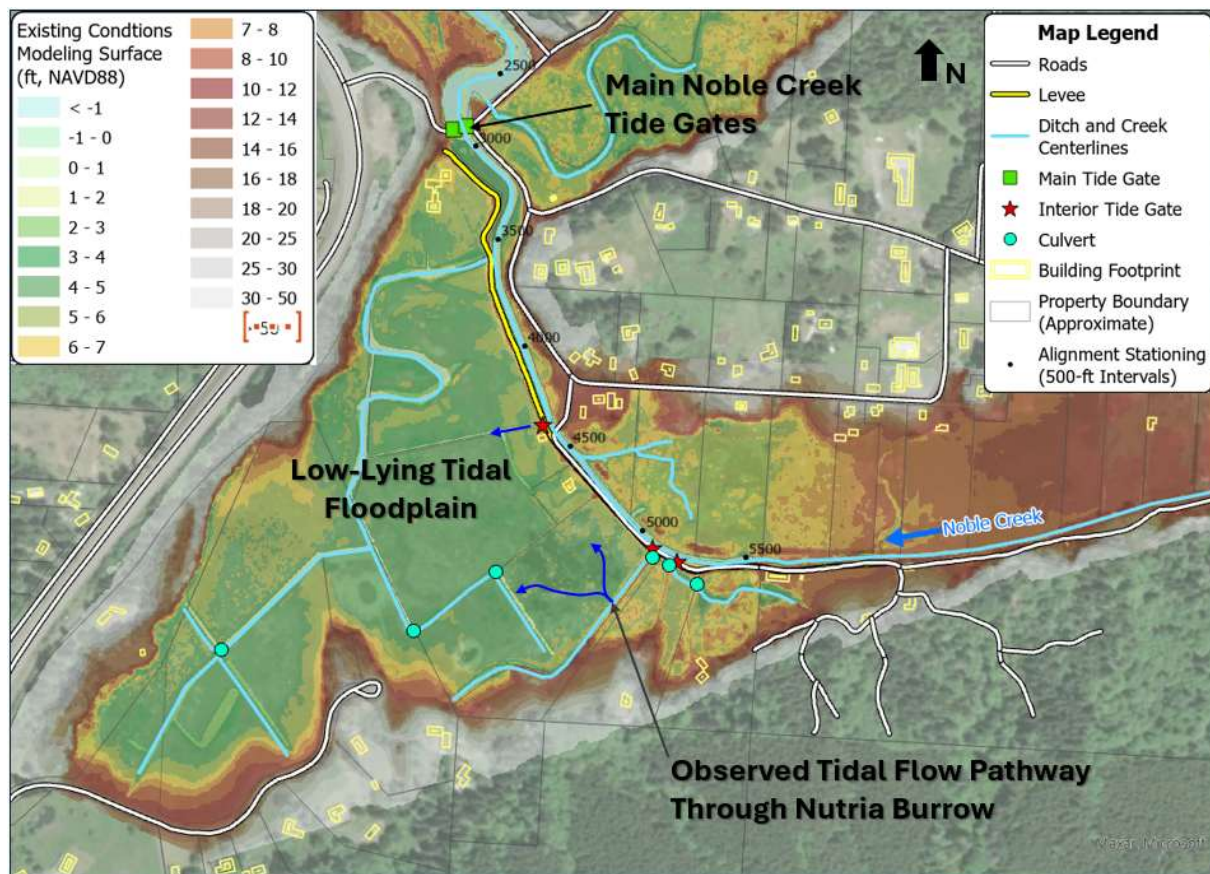
2.1.2 Interior Tidal Flood Basin

Behind (upstream of) the main Noble Creek tide gates is a broad, low-lying interior tidal floodplain occupying a geological structural basin in the coastal mountains. Approximately 100-acres of land behind the tide gates is below 5 feet elevation, and much of the interior flood basin lies at elevations between 3 and 4 feet above sea level. (**Figure 5**). Although the tidal signal is dampened by the tide gates at GAL, much of the lowland is still subject to some tidal flooding because of leakage through the tide gates and inflows from Noble Creek.

At present, tidal flows enter the floodplain through leaking interior tide-gated culverts, ditches, and deteriorated levee sections. A nutria burrow underneath a low berm is a primary pathway for tidal flows entering the floodplain and ditch network (see blue arrows in **Figure 5**). Tidal inflows move slowly up the ditch and channel network. Some flow pathways are so long that water in distal floodplain areas continues to rise hours after the peak high tide in the mainstem of Noble Creek.

In recent decades, as infrastructure has deteriorated, flooding has increased. Residents report that the amount and frequency of flooding in the fields has been increasing over the past several decades due to deterioration of flood protection (tide gate leakage, deterioration of berms and levees), land subsidence, and sea level rise. Presently, large portions of the tidal basin are flooded much of the year, and portions of the tidelands have gradually been taken out of agricultural production. In the absence of any actions, these factors will continue to increase flooding in these fields.

Upgrading the tide gates and crossing at Green Acres Lane will cause major changes to the timing and extent of inundation on the floodplain. Assuming fish passage is allowed, this offers a significant opportunity to create and enhance critical tidal habitats for salmonids in the interior tidal flood basin. Some of the landowners have indicated they are willing to allow habitat restoration activities, such as filling ditches, excavating tidal channels, and installing large wood structures.



2.2 GEOLOGY

The Noble Creek Tidelands occupy low areas within the Coast Range mountains along the central Oregon Coast. The project area is at the upstream tidal limit of Coos Bay, at the southernmost end of Isthmus Slough (see **Figure 1** for regional site map). In this region, the Coast Range consists of a series of ridges and valleys formed within faulted sedimentary rocks (**Figure 6**). The southern terminus of the Isthmus Slough Fault, a major regional fault with as much as 1,600 feet of vertical displacement, is about one quarter mile from the main tide gates at Green Acres Lane.

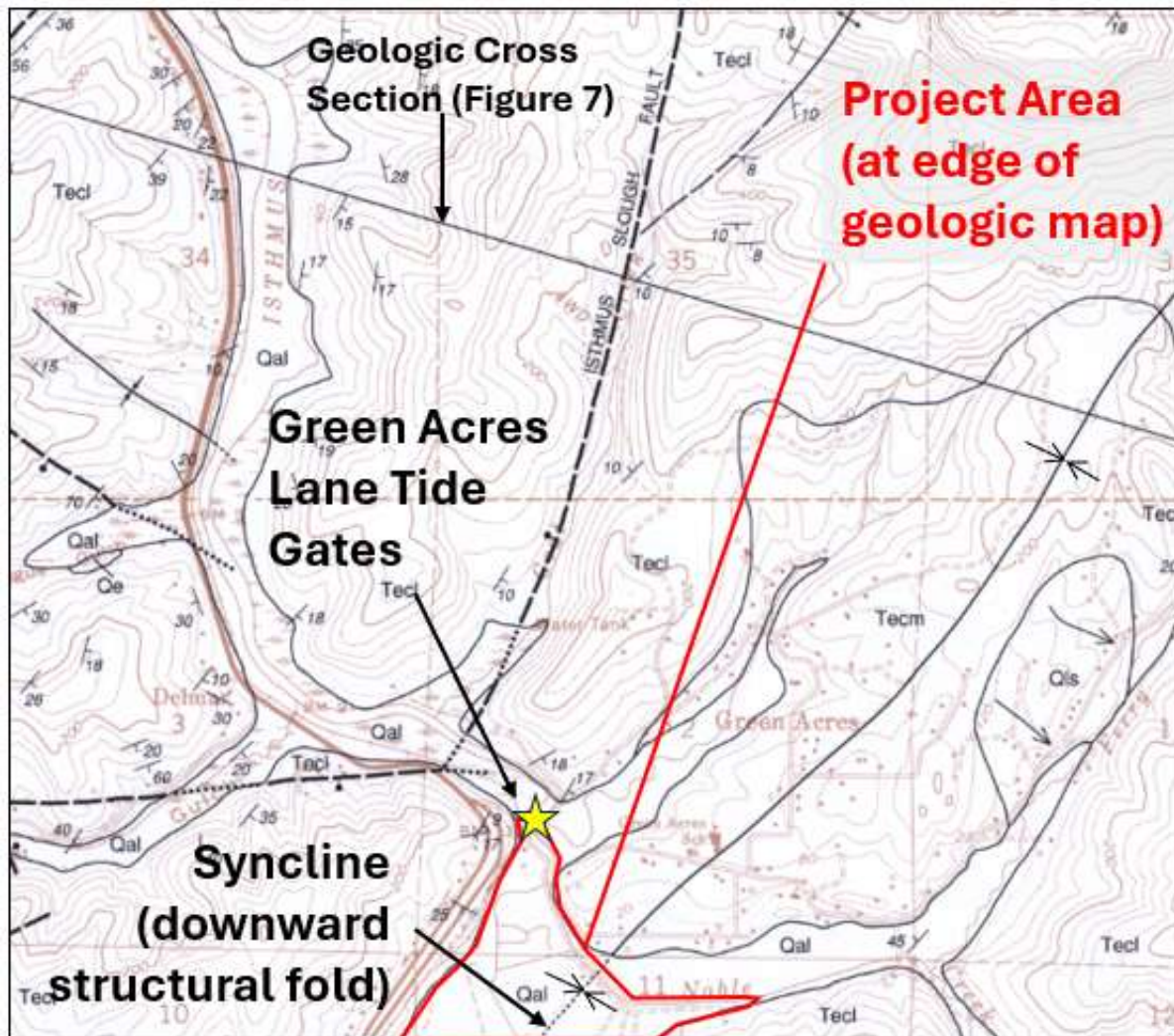


Figure 6. Geology near Noble Creek (from Black and Madin, 1995). Teci and Tecn denote Coaledo Formation (lower and middle members, respectively). The Noble Creek Tidelands Area is at the southernmost end of the geologic map.

The Noble Creek Tidelands lie within a downward warp in the folded bedrock layers (a syncline) parallel to and east of the Isthmus Slough Fault (**Figure 7**). Over the past 30 million years, tectonic compression along the boundary between the Pacific and North American tectonic plates caused

folding and faulting of the Earth's crust, creating ridges and depressional basins, one of which contains the Noble Creek Tidelands. Since the beginning of the tectonic compression, these structural low zones have gradually filled in with riverine and tidal sediment. Presently, the faults and synclines delineate much of the extensive network of sloughs and tidal floodplains in the Coos Bay region.

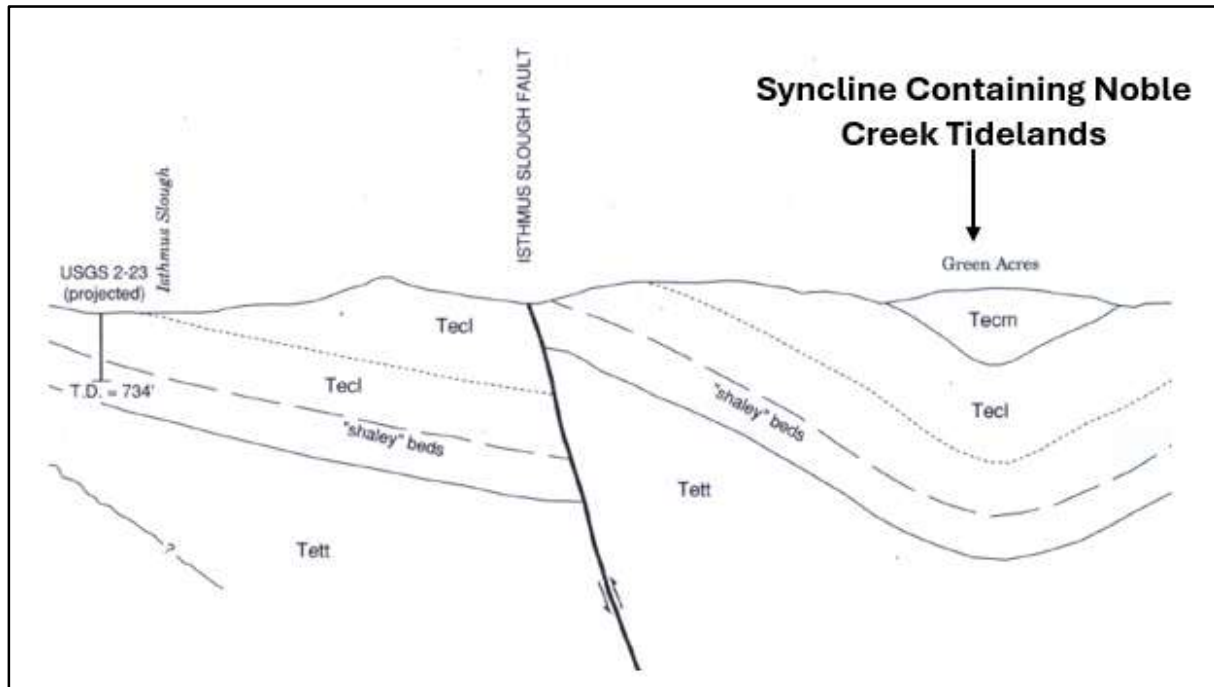


Figure 7. Part of Geologic Cross Section of Area Near Noble Creek and Green Acres (from Black and Madin, 1995) Just North of Project Area. Location of Cross Section is Shown in Figure 6.

In the vicinity of the project area, the sedimentary bedrock underlying the highlands is the Coaledo Formation, consisting of thick beds of mud deposited in a tidal setting in late Eocene time (38-45 Million Years Old), and later folded and faulted due to tectonic movements along the western edge of North America.

2.3 GEOTECHNICAL INVESTIGATION

The project will include constructing or repairing roads, culverts, bridges, and flood-control levees in poorly consolidated, water-saturated sediments in an earthquake prone setting. A geotechnical investigation was performed by Pali Consulting in May 2023 to document subsurface conditions to support the proposed design solutions. The investigation consisted of geotechnical borings at five locations, to a maximum depth of 66.5 feet below ground surface, followed by sampling and laboratory analyses. Two borings were completed in the vicinity of the existing tide gated culverts at Green Acres Lane, and two borings were completed on the levee on either side of the proposed interior tide gate with MTR (**Figure 8**). The Geotechnical Report documenting the full results of the investigation is provided in **Appendix B**. The results of the investigation are summarized briefly below.

2.3.1 Green Acres Lane Crossing

At Green Acres Lane (Borings B1 and B2), the subsurface soils are comprised of soft silts and sands to 30-40 feet below ground surface, overlying a siltstone bedrock. This is presumably the middle member Coaledo Formation (Tecm in Figure 7).

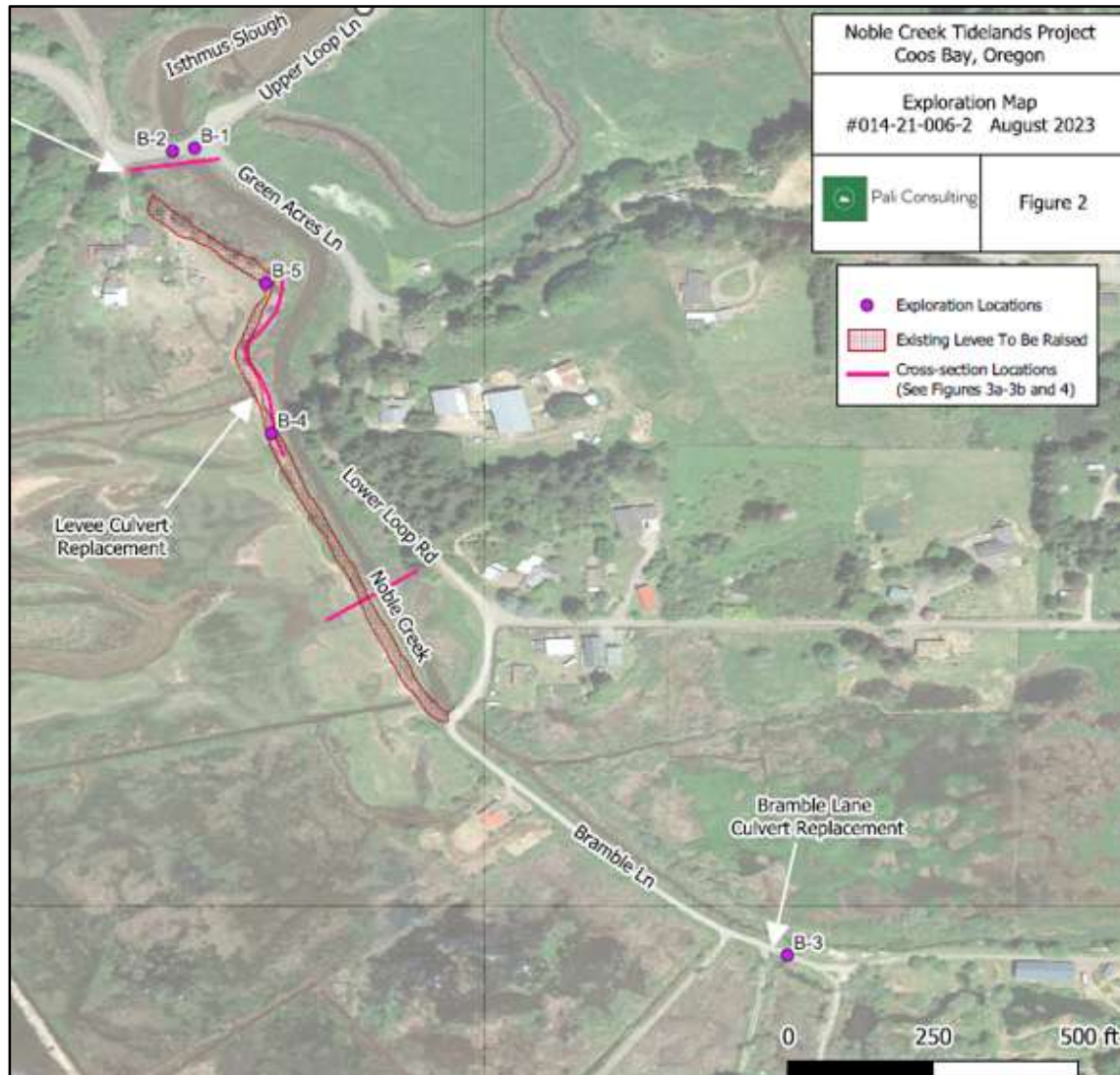


Figure 8. Locations of Geotechnical Borings in the Project Area by Pali Consulting (2023).

The initial concept designs for this project assumed that a cast-in-place culvert with a “fish-friendly” tide gate would be the preferred solution for replacing the existing culverts. The geotechnical investigation (**Appendix B**) showed that the saturated soils at the crossing were not strong enough to support a spread footing culvert, and that this approach would also require pile supported foundations. This finding led to changes in the design approach, as explained in the Alternatives Analysis Section (see Section 4.2.4).

2.3.2 Noble Creek Levee

The borings along the levee (Borings B-3, B-4, and B-5) showed similar subsurface conditions as the borings at Green Acres Lane, except the siltstone bedrock was only encountered at one of the borings. However, because the proposed culvert at these locations will be comprised of plastic pipe, rather than a cast-in-place concrete structure or bridge, there is less concern about the strength of the soils to support the culvert. The main question evaluated by the borings along the levee (especially B-4 and B-5) is what effect that potentially upgrading the levee will have on the underlying soils. The results of this analysis suggested that increasing the height of the existing levee will reduce the stability of the levees as a result of near term and long term settlement. The amount of settlement will be determined by the height and width of the proposed fill, and designs for the levee repair must account for this potential.

2.4 EXISTING CONDITIONS HYDROLOGY

The hydrology of the project area includes two features: (1) the hydrology tied to tides in Coos Bay and translated up Isthmus Slough to the project site, and (2) flows in Noble Creek controlled by rainfall and runoff in the Noble Creek watershed.

Tidal Hydrology

Water level data for the project area were needed to help develop alternative concepts, calibrate and validate the hydraulic model, and test the hydraulic performance of potential project scenarios. Coos Soil and Water Conservation District (CSWCD) installed one barometric pressure sensor and six water level sensors in the streams, sloughs, and ditches in the project area (**Figure 9**). These were maintained from December, 2021 through April, 2023. This period of record includes multiple king tide events, Noble Creek floods, and two data sets for each season of the year.

The data show that tidal fluctuations in Isthmus Slough just outside the main Noble Creek tide gates (**Figure 9A**, blue line) range from 10 feet at high tides and 0 feet at low tides, and can exceed 11 feet elevation during king tides. Just inside the main Noble Creek tide gates (orange line), the tidal fluctuations are muted by the leaking tide gates, rarely exceeding 8 feet elevation, and high tides typically around 6 feet. The interior ditches (**Figure 9B**, purple and red lines) are muted tidal systems, in which high tides are typically closer to about 4 to 5 feet elevation. Upstream of the project area (**Figure 9B**, blue line), water levels reflect flows in Noble Creek and there is a small tidal signal observable during lower stream flow periods.

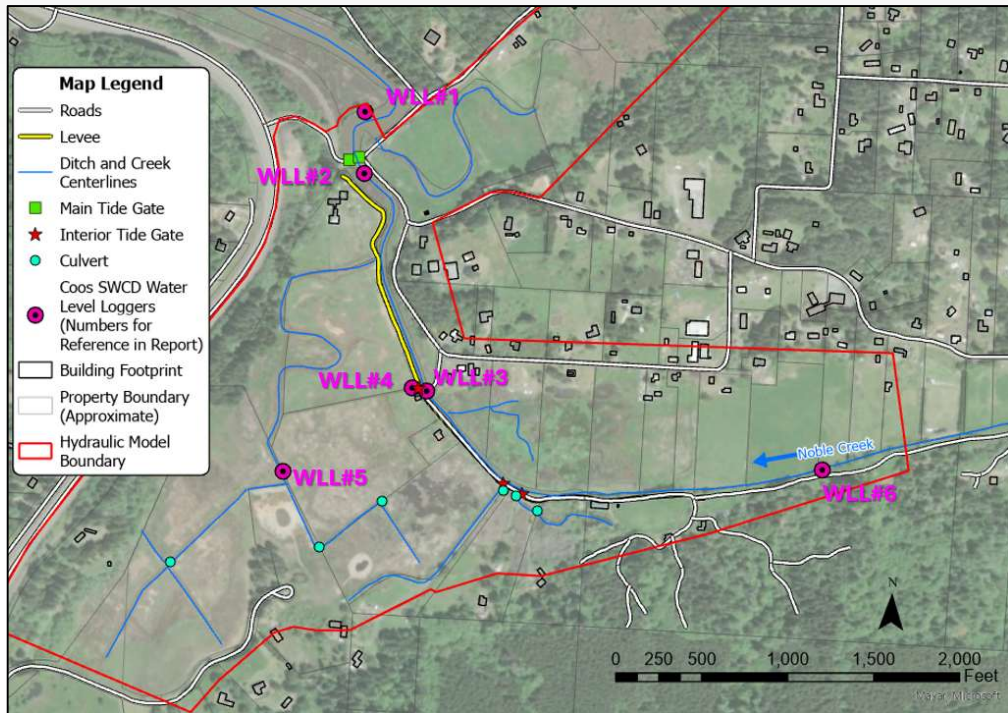


Figure 9. Locations of Water Level Loggers in the Project Area Installed and Maintained by CSWCD.

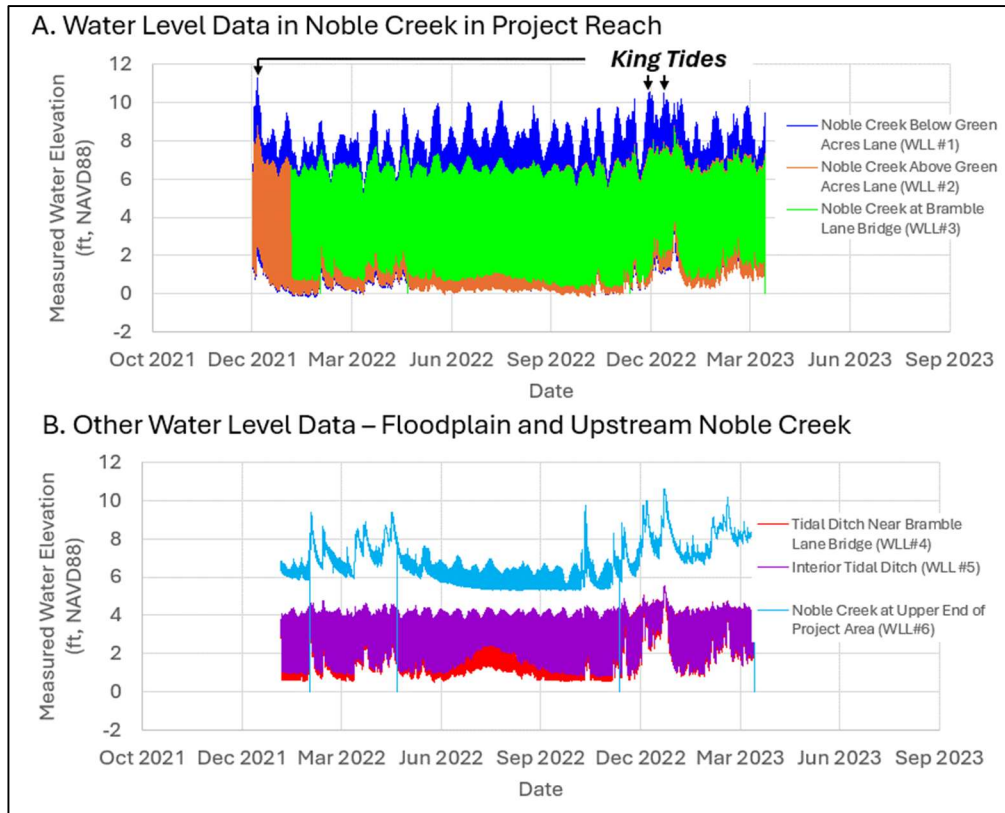


Figure 10. Water Level Data Summary.

Noble Creek Hydrology

Noble Creek drains a 5 mi² watershed in the coastal mountains east of the project area. Flow in Noble Creek is usually small, remaining below about 4 cfs about half the time, and remains below 60 cfs more than 90% of the time. More significant flows occur following large rainstorm events. Based on the calculations of USGS StreamStats' program, discharge in Noble Creek approaches 300 cfs once every 2 years on average, and reaches 1,000 cfs about once per century (**Table 1**).

The peak flow estimates based on StreamStats are significantly higher than the values computed using an alternative method based on a flood frequency analysis using nearby stream gage data. The Oregon Water Resources Department operates a "near real time" stream gage on Johnson Creek near Bandon, OR, a watershed with a similar drainage area and precipitation as Noble Creek. The flood frequency analysis suggest that the estimate of the 2-year and 100-year recurrence interval flows in Noble Creek using this data are about 130 cfs and 610 cfs, respectively, compared with 300 and 1,000 cfs using StreamStats. Because the purpose of the analysis is to evaluate the potential for flooding, using the StreamStats values is more conservative than using the estimates based on the nearby OWRD gage. Therefore, for the subsequent analysis the values from StreamStats shown in **Table 1** are used to simulate flooding in Noble Creek.

Table 1. Noble Creek at Green Acres Lane Watershed Statistics (from USGS StreamStats) and Comparison with Johnson Creek at Bandon (Peak Flow Statistics based on Flood Frequency Analysis)

Statistic	Value from StreamStats	Values Computed from Johnson Creek in Bandon OWRD Gage	Units
Basin Statistics:			
Drainage Area	5.2	4.6	mi ²
Minimum Elevation	0.6		ft.
Mean Basin Slope	12.9	4.29	degrees
Mean Annual Precipitation	63.3	63.2	in.
Peak Flow Statistics:			
2-Year Recurrence Interval Peak Flow	294	130	cfs
10-Year Recurrence Interval Peak Flow	581	300	cfs
100-Year Recurrence Interval Peak Flow	1,000	610	cfs
Low Flow Statistics:			
Flow that is Exceeded 95 Percent of the Time	0.03		cfs
Flow that is Exceeded 50 Percent of the Time	4.4		cfs
Flow that is Exceeded 10 Percent of the Time	42		cfs
Flow that is Exceeded 5 Percent of the Time	55		cfs

3.0 EXISTING CONDITIONS HYDRAULICS

Understanding the complex hydraulics in the Noble Creek Tidelands area and predicting the effects of different management scenarios will be critical to achieving the project's objectives. The hydraulics of this site are impacted by flows in Noble Creek, tidal fluctuations in Isthmus Slough, two leaking tide gates at the Green Acres Lane crossing, three interior leaking tide gated culverts, an artificial ditch network, and an important flow pathway through a nutria burrow in an existing levee. This section covers the development and calibration of the Existing Conditions model. The findings of the Proposed Conditions modeling are covered in Section 6 of the report.

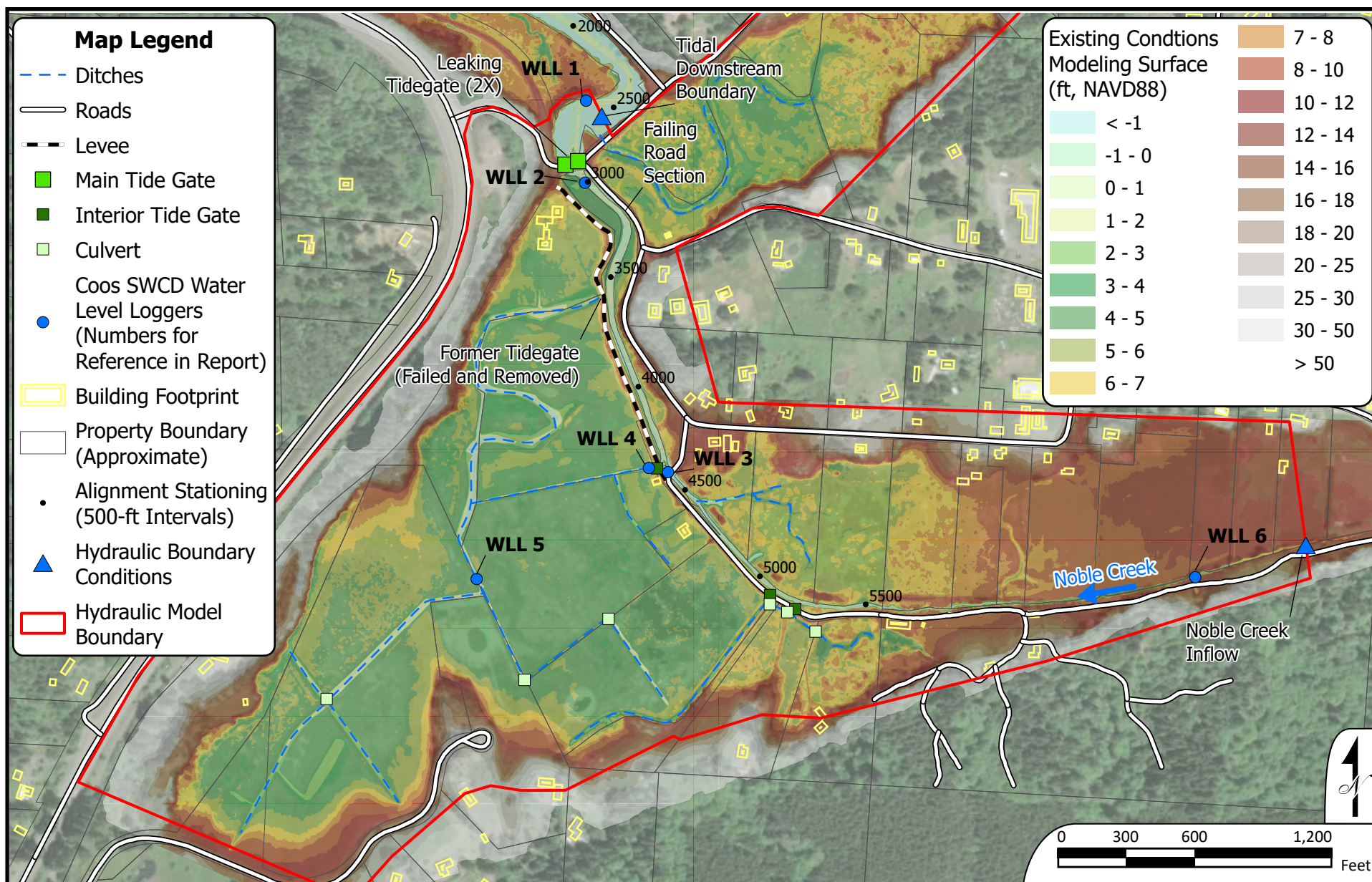
3.1 MODEL DEVELOPMENT

3.1.1 Data Sources and Model Set Up

Topographic and Bathymetric Data. Figure 11 shows the model extent and layout, along with the locations of modeled hydraulic structures, boundary conditions, and calibration points. The best available LiDAR data, collected in 2009, was used for topographic information for most of the interior floodplain and upland areas. Bathymetric data (river bed topography) in Isthmus Slough, Noble Creek, and parts of the interior ditch and channel network were collected in 2022 by Solmar Hydro Inc., using single beam sonar from a remotely operated hydrographic survey vessel. In addition, Waterways collected extensive ground based topographic survey data on the main road, culverts, tide gates, levees, ditches, houses, above ground utilities, and throughout the floodplain using RTK GPS and laser total station. Figure 12 shows the extent of the ground survey data. The LiDAR, bathymetric, and ground survey data were merged to create a single continuous topographic surface for modeling.

Leaking Tide Gates. One complexity with modeling this system is that the hydraulic conditions inside the Noble Creek Tidelands are presently driven by several leaking tide gates, especially the main Noble Creek tide gates at Green Acres Lane. The leaking tide gates transfer a “muted” tidal signal across the culverts. This leakage is a primary control on water levels on the interior of the project area, but the tidegate leakage rates are not known, and are difficult to measure directly. For the purposes of the Existing Conditions model, the leakage was modeled using a modification in the HEC-RAS model in which we included a secondary “dummy” culvert in the model at the location of each leaking tide gate. The dummy culvert allows flow in one direction only (inflow) when the water level in Isthmus Slough is higher than in Noble Creek on the inside of the tide gates. The flow rate representing the leakage was estimated through a calibration process, using water level data from the both sides of the leaking tide gates to estimate the diameter of the dummy culvert that best reproduced water level fluctuations at the interior water level sensor. Paired water level data were available at the main Green Acres Lane tide gates and at an interior tide gated culvert just at Bramble Lane Bridge over Noble Creek, near river station 4400 (WLL 3 and WLL 4 in Figure 11).

Hydraulic Roughness. The initial hydraulic roughness values in the model are based on a roughness map (Figure 13) created manually by digitizing roughness areas using aerial images, topographic information and field photos. These were adjusted during the calibration process.



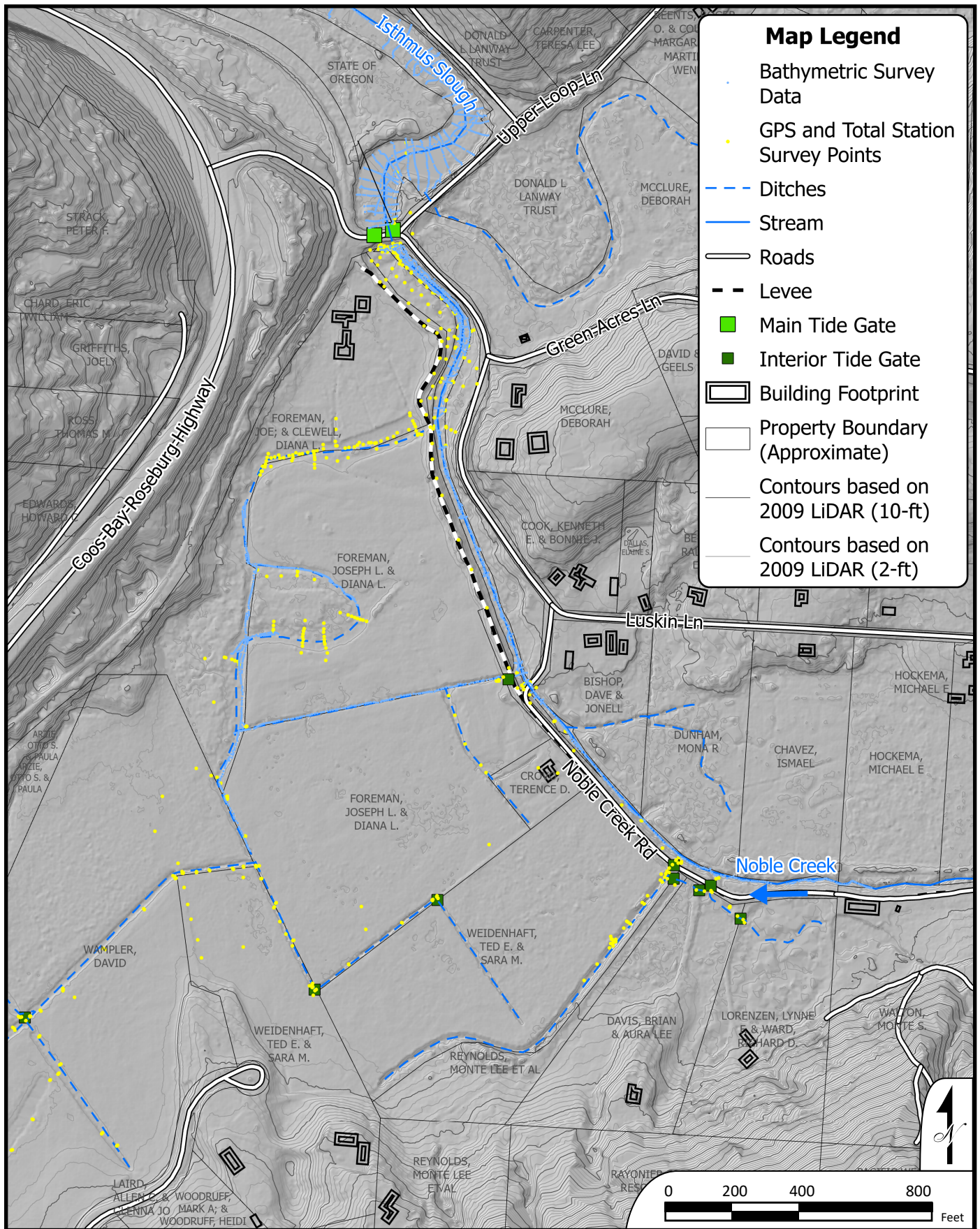
FIGURE

11

Noble Creek Tidal Hydraulics Model Overview

Noble Creek
Tidelands Restoration
Project

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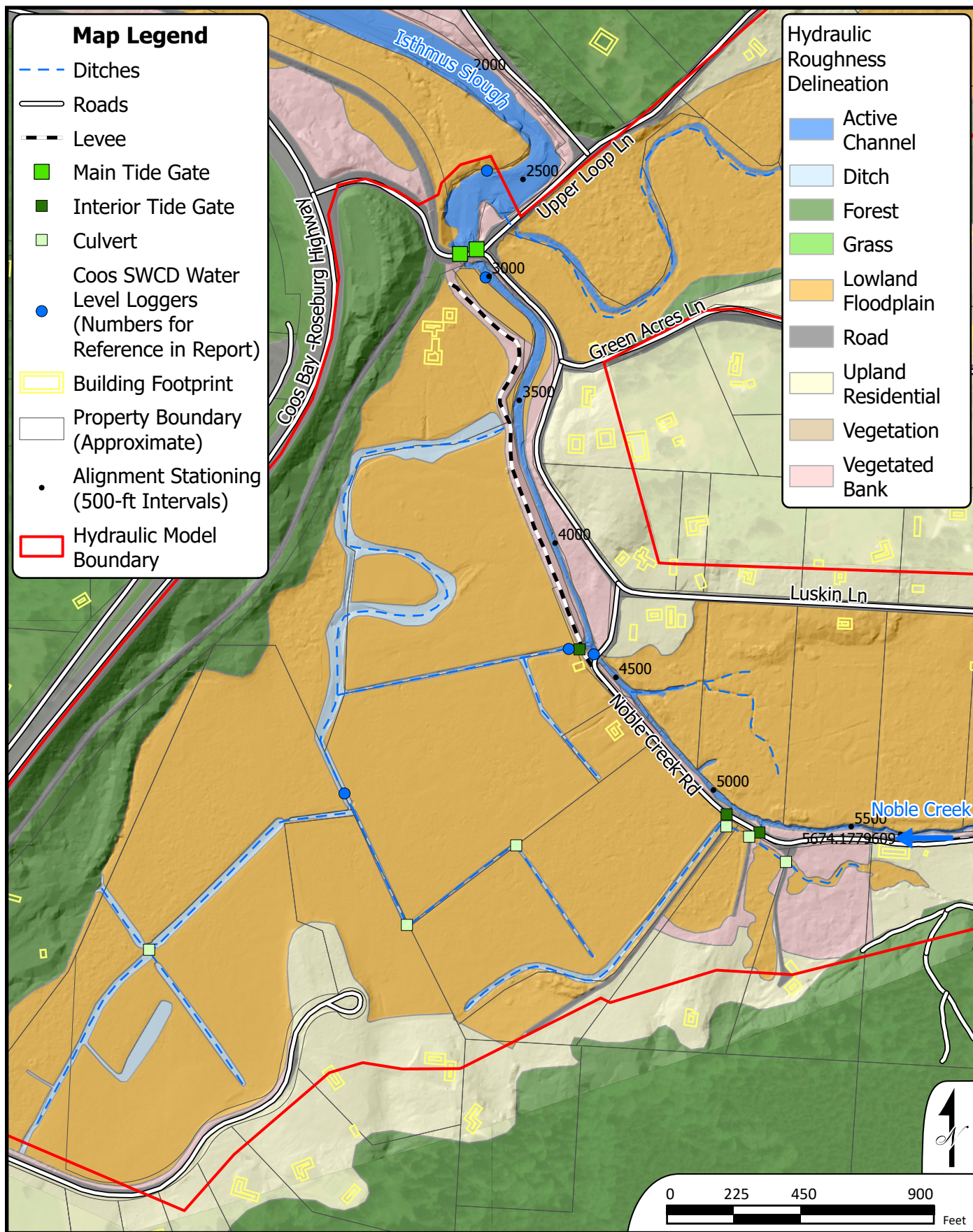


Ground Based Survey Data to Support Model

**Noble Creek
Tidelands Restoration
Project**

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**FIGURE
12**



Surface Roughness Regions for Noble Creek Hydraulic Model

Noble Creek
Tidelands Restoration
Project

3.1.2 Model Calibration and Validation

Model calibration consists of trying to improve the performance of a model by comparing the modeled predictions with measured field data and adjusting parameters in the model to better match the field data. In this model, calibration was used to estimate leakage across the leaking tide gates (discussed above), and to adjust roughness parameters in the channel, ditches and floodplain.

The selected calibration period was April 19-21, 2022. This coincided with the dates of the ground survey when Waterways staff also measured water levels at several locations at different points in the tidal cycle (**Figure 14** and **Table 2**).

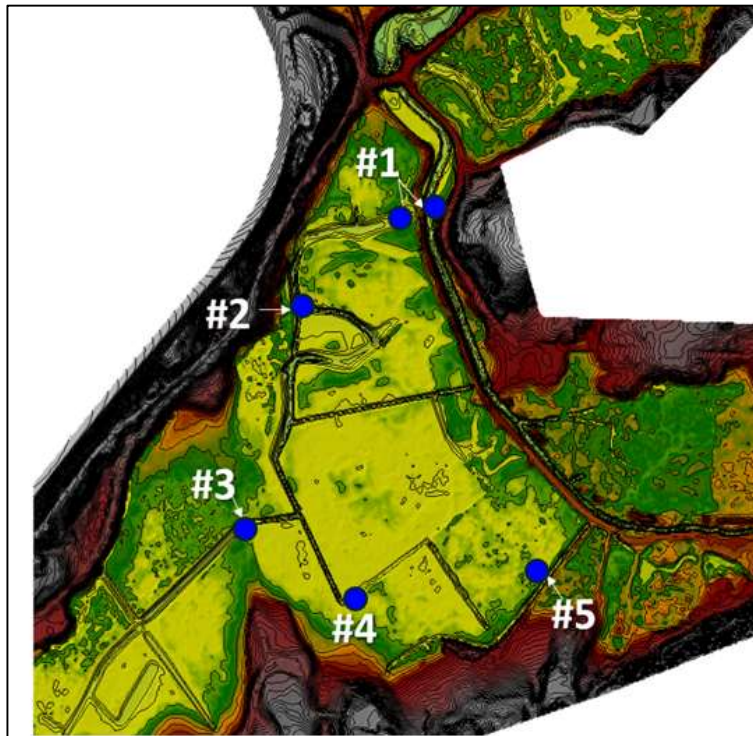


Figure 14. Locations of RTK-GPS Water Level Measurements in April 2022. Data are in Table 2.

The calibration process included 54 iterations of the model for the April 19-21, 2022 calibration period. Between model runs, roughness values and tide gate leakage values were adjusted and results were compared with field data. The calibration results are summarized in **Figure 15** and **Table 2**. **Figure 15** shows a time series of measured and modeled water levels for the final calibration run (Run 54), along with several early calibration runs to illustrate the effect calibration process on the model predictions. **Table 2** compares the surveyed water levels and modeled water levels at the corresponding times, based on the final model calibration run.

Figure 15 compares modeled and measured water levels for the April 19-21 calibration period, at each of the 6 water level loggers (logger locations are shown in **Figure 11**). The graphs show the model predictions for two of the 54 calibration runs: Run 1, the initial, uncalibrated model; and Run 54, which

was the final, calibrated model. Showing the initial and final calibration runs provides a good idea of the impact of the calibration.

Water Level Logger 1 (WLL #1), at Isthmus Slough (upper left graph) just outside the main tide gates, is the downstream model boundary condition; at that location, the modeled and measured water levels are the same. At WLL #2 and WLL #3, on the mainstem of Noble Creek behind the tide gate, the model calibrated well by the final calibration run. Using the “dummy culverts” and roughness adjustments, it was also possible to calibrate the model for interior ditches behind leaking tide gates (WLL #4 and #5) during the calibration period. The water levels at the upstream boundary (WLL #6, bottom right graph) are based on the flows measured at a nearby stream gage, Johnson Creek near Bandon, OR (ORWD Gage 14327122). Flows were adjusted slightly based on the difference in drainage area; however, little adjustment was needed because both basins are approximately 4.5 mi².

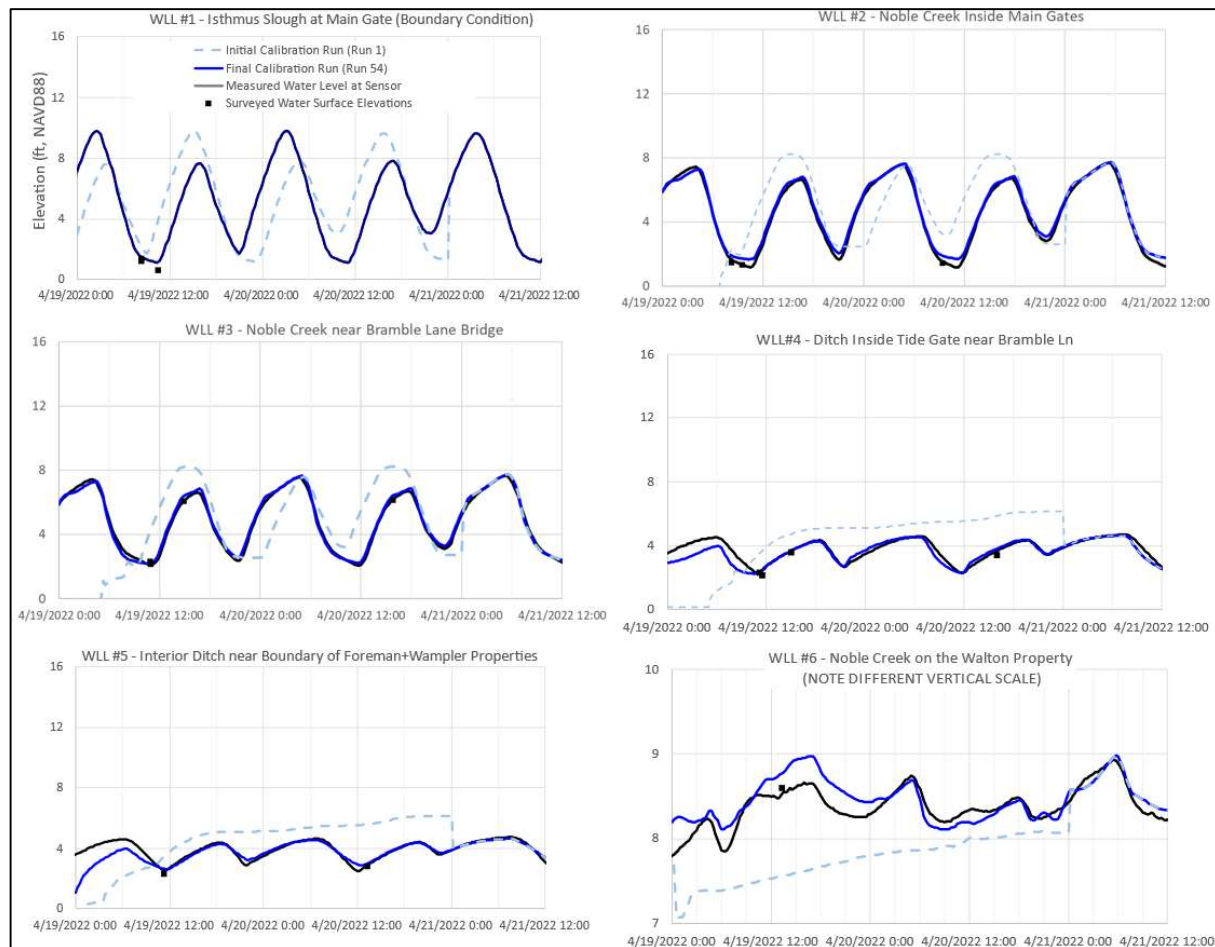


Figure 15. Final Calibration Results and Comparison with Initial Model Run

Table 2 compares instantaneous water surface elevation measurements with the model prediction at the same time and place. This comparison suggests that the instantaneous model error is typically between 0.2 and 0.6 feet, and can be as much as 1 foot. However, this discrepancy in instantaneous model predictions and surveyed elevations overestimates the limitations of the model, which calibrated well to the measured data. Most of the instantaneous error is related to the model's timing of the movement tidal signal through the system, rather than the maximum tidal elevation. The model's ability to predict maximum tidal inundation at a given location is more important than the timing, and based on **Figure 15**, the model matches the measured data well through the calibration period.

It is important to note that the model errors shown in **Table 2** and **Figure 15** are for the calibration period only. Predictions for non-calibration period, or for hypothetical events outside of the data collection window, will probably have larger errors.

Table 2. Selected Water Elevation Measurements with RTK GPS During April 19-21, 2022 Calibration Period

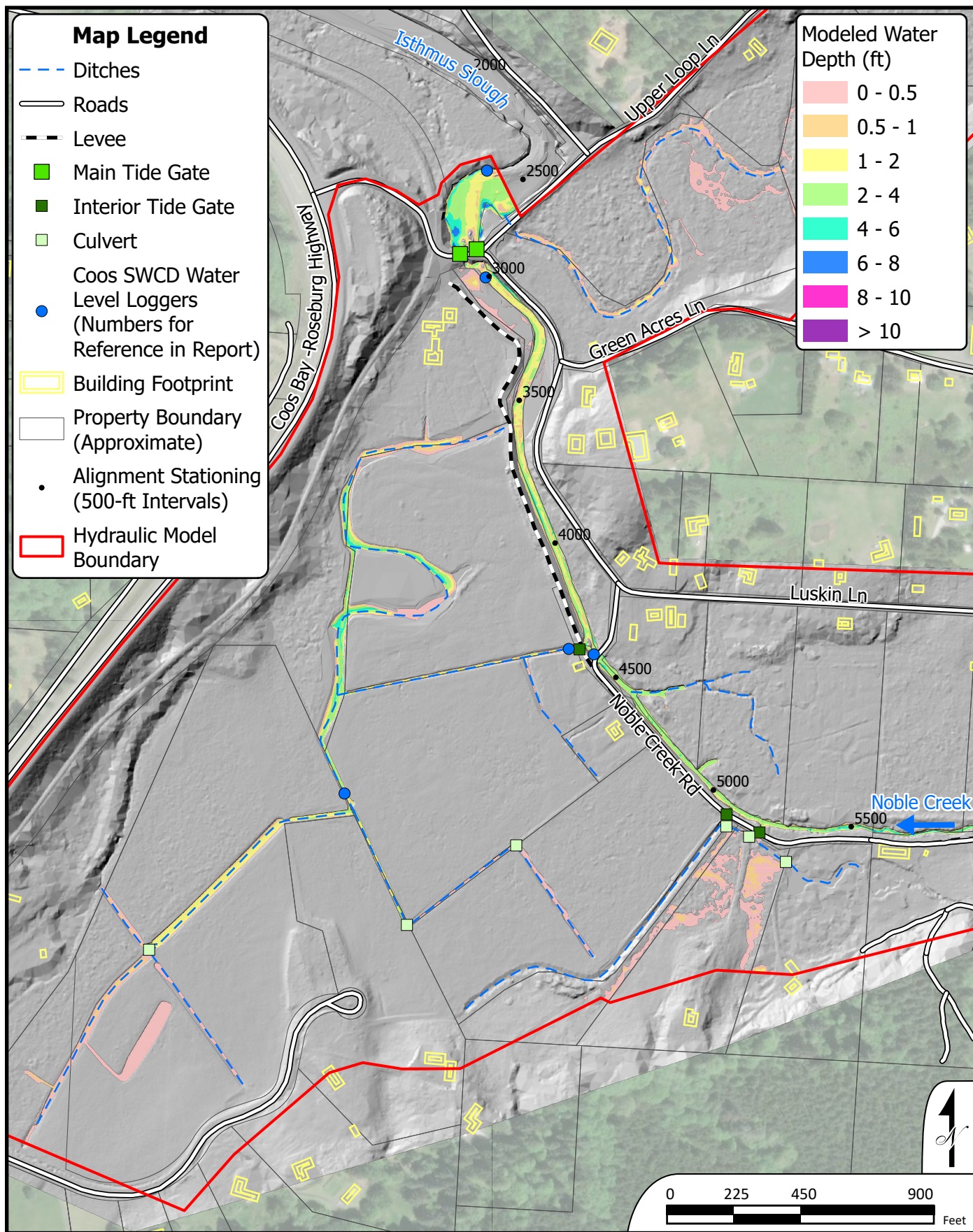
Location and Measurement Description	Date	Time	Measured Water Surface Elev. (ft, NAVD88)	Modeled Water Surface Elev. (ft, NAVD88)	Difference (Modeled minus Measured) (ft)
<i>Calibration Run #54</i>					
By Failed Tide Gate (#1)					
<i>High Tide:</i>					
2013 Inside Failed TG	04/19/22	3:45 PM	3.50	3.84	0.34
2012 Outside Failed TG (Noble Ck)	04/19/22	3:45 PM	6.21	6.61	0.41
<i>Near Low Tide:</i>					
3067 Inside Failed TG	04/20/22	10:18 AM	3.50	3.77	0.27
3068 Outside Failed TG (Noble Ck)	04/20/22	10:20 AM	1.70	2.03	0.33
North End Big Meander (#2)					
4034 North End of Big Meander	04/20/22	1:30 PM	2.25	3.23	0.98
Bend in Ditch (#3)					
4120 Bend in Ditch	04/20/22	2:56 PM	3.10	3.60	0.50
90 Degree Bend in Ditch (#4)					
4064 90 Degree Bend	04/20/22	2:28 PM	2.91	3.47	0.56
4065 90 Degree Bend	04/20/22	2:30 PM	3.06	3.47	0.41
Levee (#5)					
5031 Breach in Levee	04/21/22	9:39 AM	5.21	4.60	-0.61
5032 Breach in Levee	04/21/22	9:40 AM	5.02	4.60	-0.42

3.2 EXISTING CONDITIONS MODEL RESULTS

The calibrated model was used to simulate a variety of scenarios to better understand the site's hydraulic function. This section discusses the findings of the Existing Conditions modeling; modeling of several Proposed Conditions model is discussed under the Alternatives Analysis. The existing conditions model was run using the measured water level at WLL #1 as the downstream boundary condition. Flows into the model from Noble Creek were estimated using flow data from nearby stream gage at Johnson Creek (ORWD Gage 14327122) as discussed above. The model simulated the water levels and velocities throughout the project area over time. Some of the results are summarized in **Figure 16, 17, and 18**, which show the predicted inundation patterns at a typical summer low tide, a typical winter high tide, and during a king tide event.

The following bullet points summarize Waterways' interpretations of the Existing Conditions model results:

- During low tides in the summer (**Figure 16**), one to four feet of water remain in the mainstem of Noble Creek and in the large, curved abandoned channel in the center of the Foreman's property.
- Less than 1 foot of water remains in the ditches and ponded areas in the floodplain at low tides (**Figure 16**).
- During typical high tides (**Figure 17**), enough water gets into the ditches because of leakage through tide gates to fill ditches and cause minor flooding of fields. At high tide, the water depths in the center of the ditches and abandoned channel are 2 to 6 feet.
 - There is also a hole in a levee on the east side of the basin where nutria have burrowed under a levee on the east side of the basin, allowing water to enter the floodplain. This is included in the model as a small culvert, which probably underestimates the impact of the burrow.
- In a king tide, model shows more inundation across floodplain (**Figure 18**), but presumably much less flooding than if there were no tide gates in place.

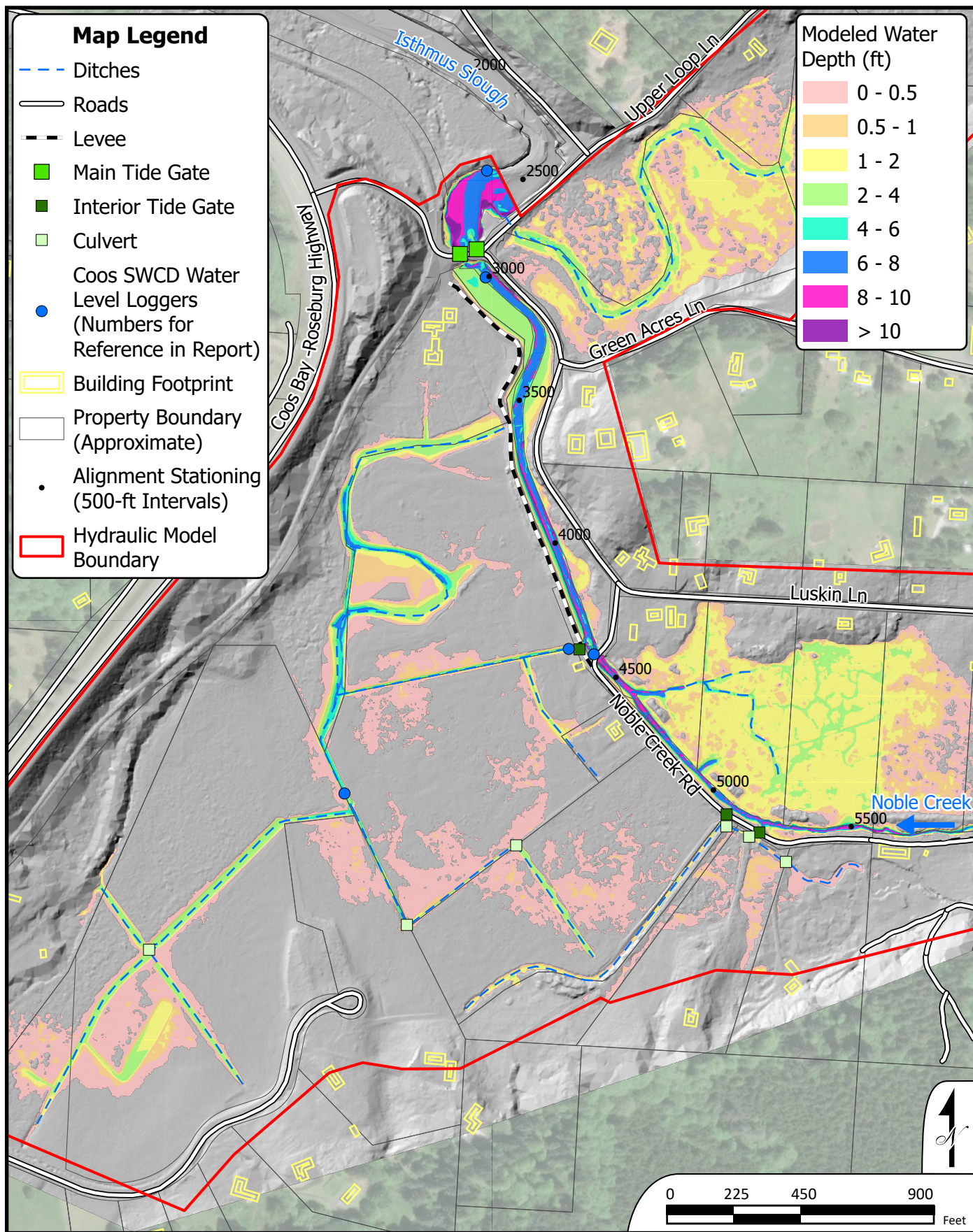


**Existing Conditions Model Depth
Typical Low Tide
May 15, 2022 7:30 AM**

**Noble Creek
Tidelands Restoration
Project**

WATERWAYS
CONSULTING, INC.
Santa Cruz, CA | watways.com | Portland, OR

**FIGURE
16**

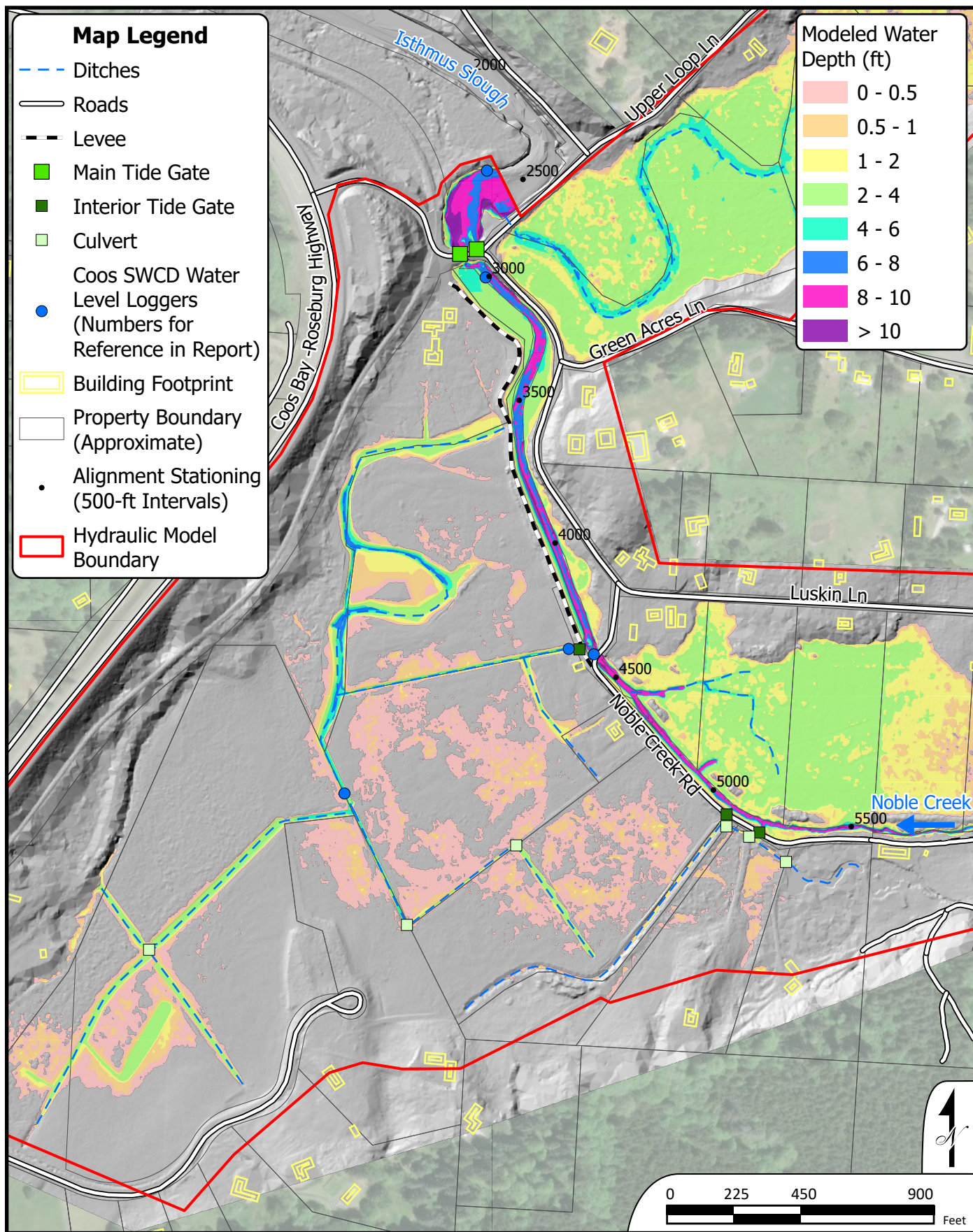


**Existing Conditions Model Depth
Typical High Tide
May 16, 2022 3:00 AM**

**Noble Creek
Tidelands Restoration
Project**

WATERWAYS
CONSULTING, INC.
Santa Cruz, CA | watways.com | Portland, OR

**FIGURE
17**



Existing Conditions Model Depth
King Tide
January 3, 2022 1:15 PM

Noble Creek
Tidelands Restoration
Project

WATERWAYS
 CONSULTING, INC.
 Santa Cruz, CA | watways.com | Portland, OR

FIGURE
18

4.0 ALTERNATIVES ANALYSIS

4.1 OPPORTUNITIES AND CONSTRAINTS

The Noble Creek Tidelands Restoration Project is a multi-objective effort that aims to accomplish multiple infrastructure, public safety, and habitat goals. The main opportunities and constraints for the project are summarized below.

Project Opportunities

- Work with Coos County to improve crossing to ensure reliable access to Green Acres community
- Replace existing deteriorated and leaking tide gates with “fish-friendly” tide gates to restore fish access to large area of critical tidal habitat.
- Upgrade interior levees where appropriate to provide better flood protection.
- Excavate new tidal channels, fill ditches, and add wood and other complexity elements to improve the quality of newly accessible tidal habitat.

Project Constraints

- Vehicle access to the Green Acres community must be maintained without interruption.
- Project must ensure the same or a higher level of protection from tidal inundation for residences in floodplain.
- Landowners must approve of the location and frequency of increased tidal inundation as a result of the project.
- All new tide gates must meet fish passage criteria as laid out by Oregon Department of Fish and Wildlife guidelines.

The alternatives analysis was subdivided into two parts: (1) Consideration of alternatives at Green Acres Lane, and (2) Consideration of alternatives for the interior tidal floodplain. Each of these two parts of the project offer separate, but dependent, options. For the purposes of this project, it was important to first clarify what the preferred alternative would be at the Green Acres Lane crossing. Once that was defined, it was possible to compare various options for improving the newly accessible habitat.

CSWCD organized a community meeting in December 2022 at which the design team presented the alternatives. The Alternatives Analysis presentation from the community meeting is provided in **Appendix C**, and the key points are summarized below.

4.2 ALTERNATIVES AT GREEN ACRES LANE

The crux of the project is the improvements to the crossing of Green Acres Lane (GAL) over Noble Creek. At this crossing Noble Creek enters Isthmus Slough through two aging and deteriorated wooden

culverts with leaking wooden tide gates. Just upstream from the GAL crossing is a failing section of road on river right along Noble Creek, which is slated to be upgraded by Coos County (**Figure 19**).

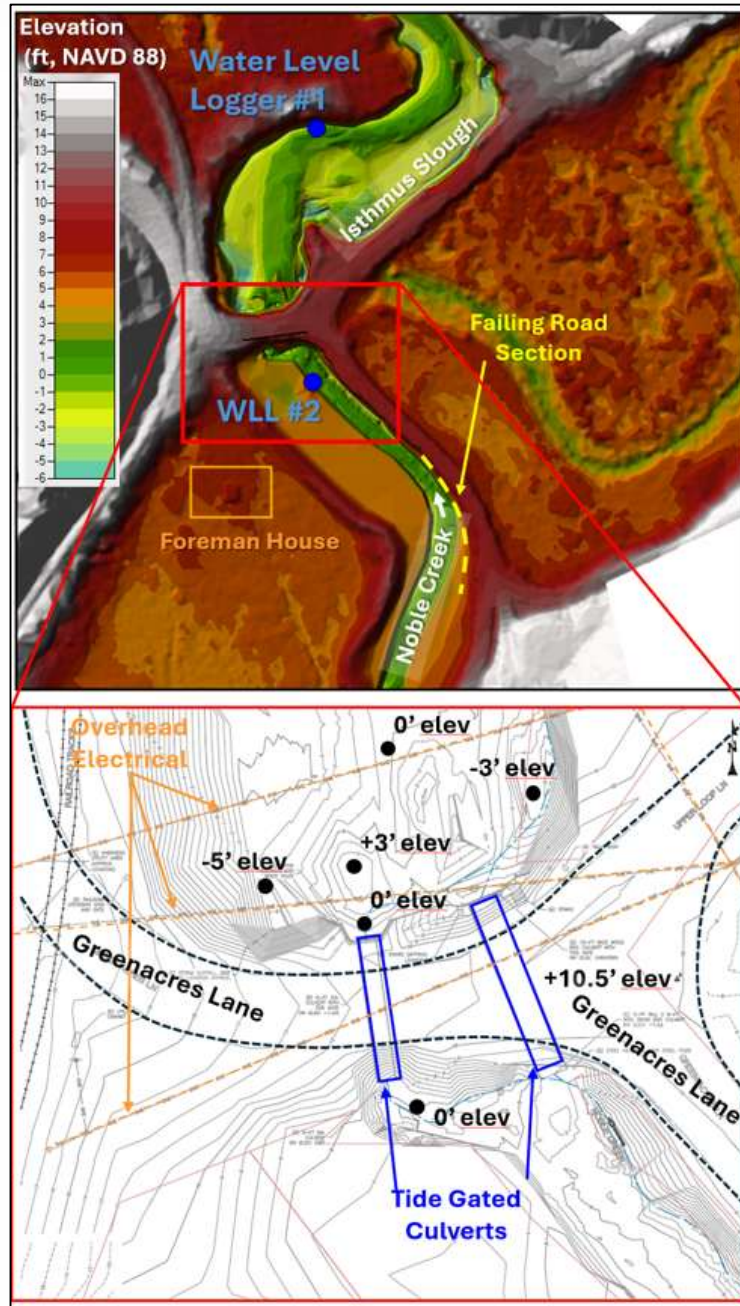


Figure 19. Existing Conditions Near the Green Acres Lane Crossing

The design team looked at three alternative options for upgrading the crossing (**Figure 20**): (A) Replace the GAL crossing and culverts with a new bridge; (B) Replace the GAL crossing with new, upgraded, tide gated culverts; and (C) Replacing the crossing with two separate structures: a bridge or culvert

road crossing for traffic, and a separate structure containing tide gates, which would be used to control the water levels.

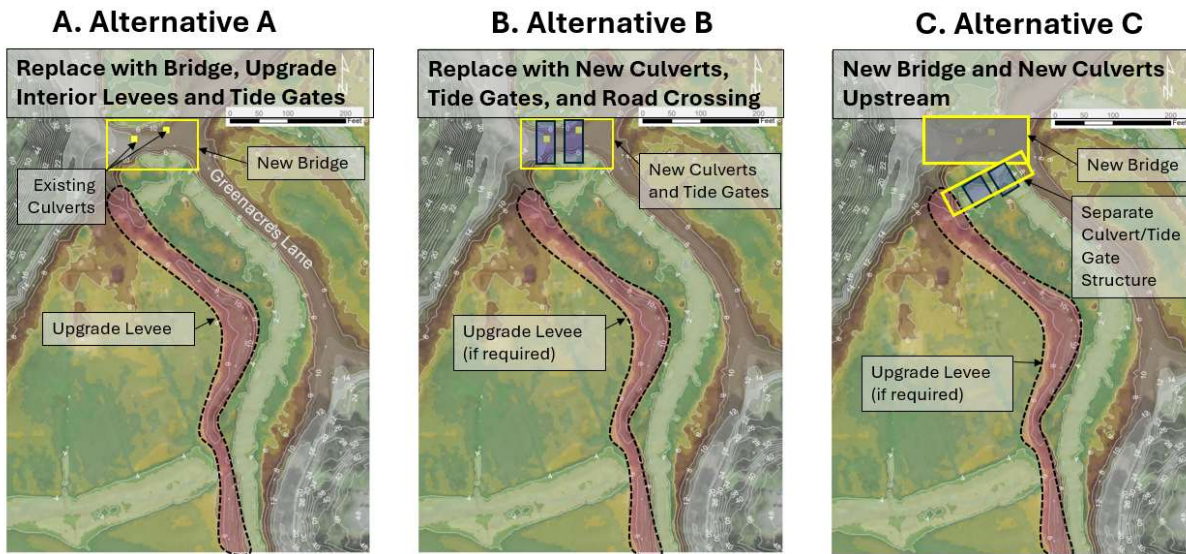


Figure 20. Alternatives for Near the Green Acres Lane Crossing

4.2.1 Alternative A - Bridge Only Option

Alternative A (a.k.a. Bridge Only Option) is the option for Green Acres Lane that assumes the existing tide gated culverts would be replaced with a single span bridge to provide fish passage at all stages. This would be coupled with increasing the heights of the interior levees and installing interior tide gates to protect against tidal flooding on adjacent properties (**Figure 20A**). This would be the simplest solution for the Green Acres Lane crossing but may not provide enough flood protection.

To evaluate the impact of replacing the culverts with a bridge, we modeled tidal flooding under existing and proposed conditions for measured a king tide event (January 3, 2022). In this version of the model, the tide gates were removed and replaced by a bridge, and the interior levee elevations were *not* raised to compensate for this. **Figure 21** shows the extent and depth of modeled inundation for existing conditions and this “Bridge Only” option. The model shows much more widespread flooding in a king tide event with the tide gates replaced by a bridge. Without any tide gates, in a king tide the water surface elevations would exceed levee elevations along almost the entire length of the southern levee and overtop the road. This implies that the entire southern levee along Noble Creek, which is about 2,700 feet long, would need to be raised by as much as several feet to provide the same level of protection to the adjacent pastures, houses, and other infrastructure. Extensive rebuilding of the levee would require soil import because of the poor quality of on-site soils for levee construction (see above, Section 2.3, and geotechnical report, **Appendix B**). In addition, this Alternative also would increase the extent of flooding for landowners to the north of Noble Creek, so additional flood protection may also need to be provided on the north side of the creek.

Given the findings from the hydraulic modeling and the levee soil properties discussed in the geotechnical investigation (Section 2.3 and **Appendix B**), this Alternative was considered infeasible and was discarded as a design solution.

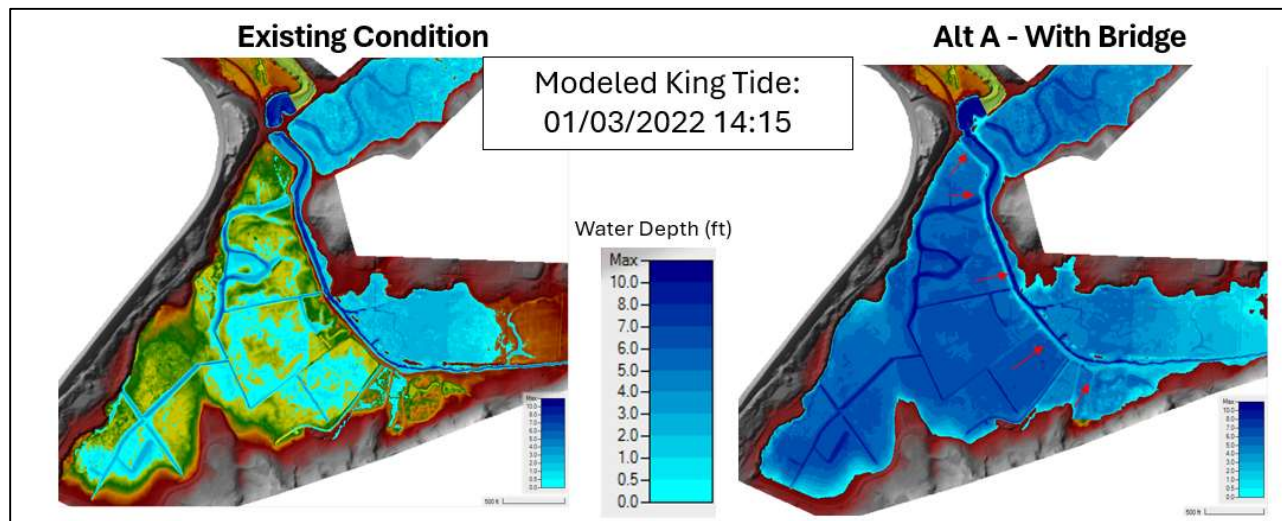


Figure 21. Comparison of Modeled Inundation King Tide Under Existing Condition and Alternative A. Red arrows in right panel indicate primary areas where levee would be overtopped.

4.2.2 Alternative B - New Crossing, Tide Gate and Culverts

Alternative B would replace the existing tide gated culverts at GAL with new culverts, tide gates, and muted tidal regulators (**Figure 20B**). Interior levees may need to be upgraded based on the water management plan, but they would not need to be fully constructed along their entire length as in Alternative A.

Based on initial modeling which showed that at least two 8-foot diameter culverts would be required to meet the fish passage requirements for the tide gates at this location, it was assumed that the culverts would be concrete culverts cast in place on a spread foundation. To maintain vehicular access to the Green Acres community throughout the construction process, two options would be possible: (1) build a temporary crossing either upstream or downstream of the current crossing, then remove and rebuild the crossing and tide gated culverts and remove the temporary crossing, or (2) build the new permanent crossing in a different location from the current crossing, and then remove the current GAL crossing and culverts after the new crossing is completed. Of these two, option 1 would be more likely because the current location is the most suitable long term crossing location due to the configuration of the roads, utilities, private properties and existing wetlands (see **Figure 19**).

In this scenario, long term maintenance responsibilities could be unclear. Coos County would maintain the road and crossing. However, the county does not want to be responsible for maintaining the

tidegate infrastructure, and this task may need to be taken on by project sponsors, landowners, or a community drainage district. If the road and tide gates are integrated into the same structure, maintenance responsibility under some scenarios could become complicated.

4.2.3 Alternative C - Separate Bridge and Tide Gate Structures

In this alternative, the crossing would be replaced by a bridge, and a separate tide gate and culvert structure would be built separate from the bridge (**Figure 20C**). In this scenario the ownership and maintenance responsibilities would be more clearly defined compared with Alternative B. There would be some constructability advantages with this option, which would be constructed over two summers. In year one, the separate tide gate structure could be constructed without interrupting traffic flow to the neighborhood. In the second construction season, traffic could be temporarily diverted over the tide gate structure while the new bridge was being built, and traffic could be re-directed over the bridge before the winter.

While Alternative C provides some advantages over Alternative B in terms of constructability and maintenance responsibility, it comes with a much higher cost because two separate permanent structures would need to be built. It was determined by the design team, in consultation with the community and Coos County roadmaster, that the benefits of Alternative C did not justify the added costs.

4.2.4 Selection of a Preferred Alternative for Green Acres Lane Crossing

Coos SWCD organized and hosted a community meeting at the Noble Creek Fish Hatchery on December 14, 2022 to discuss the project, to hear feedback regarding the community's preferences, and begin the process of selecting a preferred alternative. At the community meeting, CSWCD and ODFW provided an introduction to the project to the community. Waterways reviewed the alternatives discussed above and presented some initial modeling results, and CSWCD facilitated a community discussion about the project. The presentation materials outlining the alternatives are provided as **Appendix C**.

In general, the community members were supportive of a project to improve the reliability of the Green Acres Lane crossing while improving and expanding salmonid habitat. There were concerns expressed about the costs to community members of maintaining tide gates, as well as vehicle access to the community during construction. Following the meeting, CSWCD followed up with community members, landowners, and with the Coos County roadmaster. Waterways further developed the alternatives and the hydraulic analyses. Following this process, the design team (CSWCD, ODFW, and Waterways) selected Alternative B as the most effective and efficient option. Following the full geotechnical investigation in May 2023, however, it became clear that the culverts on a spread foundation would not be possible due to unstable soils, and that very deep piles would be required to anchor any bridge or culvert (**Appendix B**). This led to replacing the preferred alternative (Alternative B) with a variation on this Alternative B. In short, there will be a bridge crossing with deep piles, plus a separate tide gate structure immediately adjacent to the bridge. The tide gates will be attached to a

sheet pile wall, without the need for heavy concrete culverts. The selected option is shown in the design drawings (**Appendix A**, Sheet C4) described in more detail in the Basis of Design (Section 5.1).

4.3 ALTERNATIVES FOR INTERIOR OF PROJECT AREA (BEHIND MAIN TIDE GATE)

Once it was determined that the current Green Acres Lane crossing would be replaced with a new tide gated crossing at about the same location (Alternative B), it was possible to begin to evaluate alternatives for enhancements to the interior of the project area. Three alternatives for the interior project area were considered:

- No interior improvements
- Upgrade interior tide gates
- Upgrade interior tide gates with channel and floodplain restoration

4.3.1 Alternative A - No Interior Improvements

Alternative A for the interior project area is the “No Action” alternative. In this scenario, the project would focus on replacing the crossing and tide gates at the Green Acres Lane crossing and necessary levee upgrades, with no improvements or upgrades to interior tide gates, ditches, or channels for fisheries benefit.

Advantages of Alternative A. This option would improve the reliability of neighborhood access in the Green Acres community while also improving fish passage and allowing salmonids to access tidal habitat. This Alternative would be the simplest and cheapest option and allow the design and construction process to focus on the road crossing and a single work area. This option would also leave open the possibility for doing interior habitat enhancements and upgrading interior tide gates as part of potential future projects.

Disadvantages of Alternative A. The main disadvantage of this approach is that it would not maximize the habitat enhancement opportunities of the site. The Noble Creek Tidelands consist of nearly 100 acres of potentially tidally inundated channels and floodplains that are mostly disconnected from natural tidal processes due to levees, berms, ditches, and tide gates. While improving passage at the main GAL tide gates will allow fish to access the main stem of Noble Creek and part of the floodplain, the fisheries benefit from this approach may be limited. Although it is possible that future projects will continue to enhance the site, it is not certain that will happen. In addition, potential funders of the project are primarily interested in habitat benefits. Without more interior habitat improvements, it may be difficult to fund the project.

Therefore, Alternative A was discarded in favor of other options that will provide more habitat benefit.

4.3.2 Alternative B - Interior Tide Gates Only

In Alternative B, the Noble Creek crossing at Green Acres Lane would be upgraded as described in above, and interior (upstream of the GAL crossing) culverts and tide gates would also be improved to provide more fish access and better flood protection (**Figure 22**) For the initial evaluation, it was assumed that three interior culverts would be installed, removed, or upgraded:

- (1) A new culvert would be installed in the levee on river left, several hundred feet upstream of GAL crossing (**Figure 23A**). This new culvert would be at the same location where a culvert washed out in May of 2020. Depending on the water management plan at the main GAL tide gates that stakeholders agree on, this interior culvert may or may not need a tide gate. If a tide gate is required, it would probably be equipped with a muted tidal regulator (MTR) that would allow fish passage into the Tidelands. This site would be a primary access point for fish into the floodplain and tidal channel network.
- (2) A culvert and tide gate on river left just downstream of the Bramble Lane Bridge over Noble Creek would be removed (**Figure 23A**). This is currently a leaking tide gated culvert that allows part of the existing ditch network to drain into Noble Creek. This feature will not be needed if the other culverts are upgraded, especially if the ditches are filled. The culvert would be removed and the levee section would be reconstructed.
- (3) An interior culvert and wooden tide gate in the upstream part of the project area would be replaced (**Figure 23B**). Leakage through this culvert currently allows some tidal exchange; however, the tidal signal is muted and fish passage is limited to non-existent. The upgraded culvert and tide gate would be designed to allow more tidal exchange and fish passage into the southeastern part of the Noble Creek Tidelands.

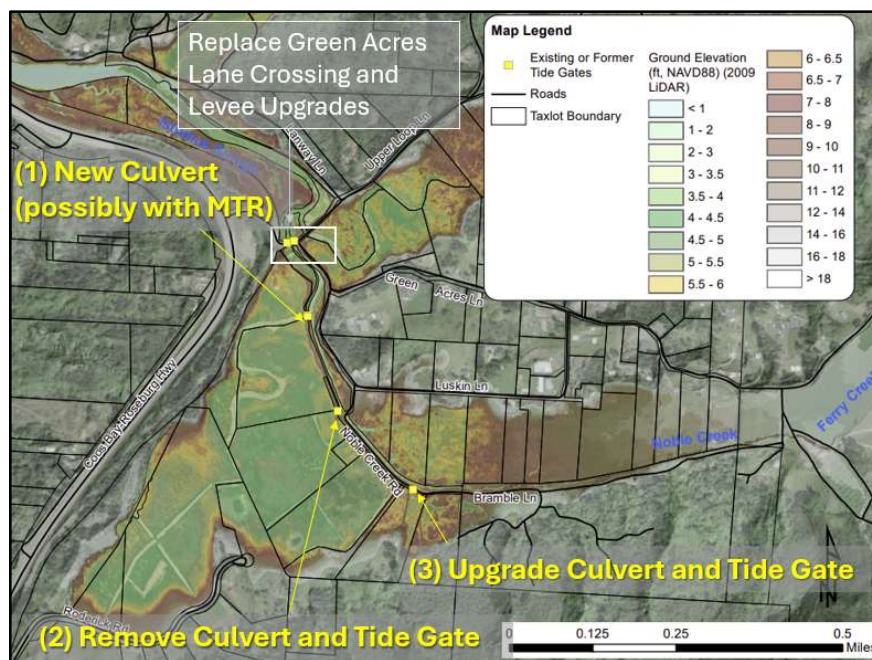


Figure 22. Alternative B for the Interior Project Area – Upgrade Interior Culverts and Tide Gates.



Figure 23. Photos of Current Locations Where Interior Culvert and Tide Gate Improvements are Proposed. (A) View Looking Downstream at Leveed Section of Noble Creek from Bramble Lane Bridge. (B) Photo of Tide Gate and Leaking Culvert in the Upper Part of Project Area

Advantages of Alternative B. Compared with Alternative A, this option would significantly improve fish access and tidal exchange in much of the 80-acre low elevation floodplain southwest of Noble Creek. This option would also upgrade failing and leaking culverts and tide gates on private properties with modern, engineered drainage systems compatible with current land uses. Upgrading the interior culverts and tide gates will also allow for more control and the ability to “fine tune” the water management plan and make adaptive changes to the WMP over time.

Disadvantages of Alternative B. This option would increase the complexity of the project compared with Alternative A, with work occurring in multiple private properties and land use types. The cost of design and construction of the project will increase compared to Alternative A. This option, while improving tidal exchange and fish access, does not improve the quality of the habitat behind the tide tide gates.

4.3.3 Alternative C - Interior Tide Gates with Restoration

Alternative C would be a comprehensive project that combines upgrading the main tide gates, the Noble Creek levee, the interior tide gates, and also adding earthwork and large wood supplementation to restore the tidal channel network southwest of Noble Creek. The earthwork would consist of filling ditches and excavating a more sinuous and complex tidal channel network that would provide more and higher quality habitat than the ditches would provide. The habitat quality in these channels would be further enhanced by placing hundreds of trees and root wads in the channels and floodplain to provide more shade, cover, and edge habitat (**Figure 24**).

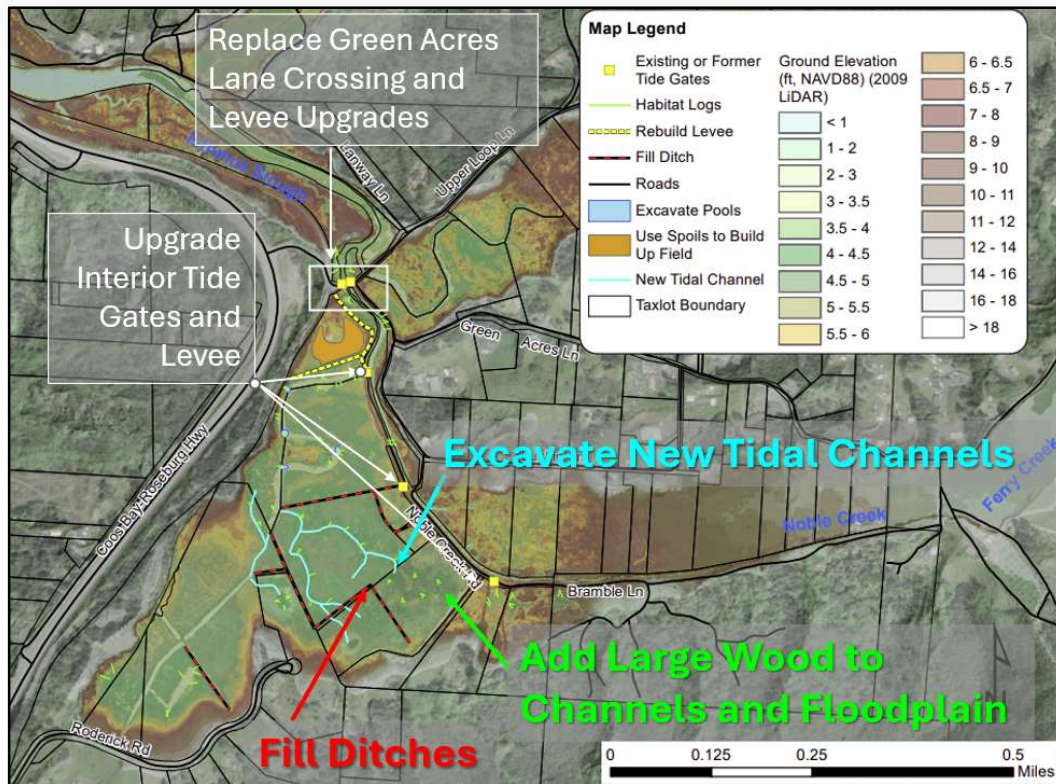


Figure 24. Alternative C for Interior Project Area- Tidal Channel and Floodplain Restoration

Advantages of Alternative C. Compared with Alternatives A and B, this option would improve the amount and quality of fish habitat in the 80-acre low elevation floodplain southwest of Noble Creek (no channel grading or other work is proposed on the 20-acre tidal lowland north and east of Noble Creek).

Disadvantages of Alternative B. This option would increase the complexity of the project compared with Alternatives A and B, with work occurring in even more private properties. The significant amount of grading and large wood proposed will increase the cost of the project substantially. Extensive disturbance of wetland soils will be required. Equipment operation could be difficult in the mucky areas. Permitting studies and cultural resources evaluations may be more complicated and more costly due to the larger disturbance area.

costs.

4.3.4 Selection of a Preferred Alternative for Interior of Project Area

The project team determined that the project would have be more impactful, and have a better chance of being funded, by maximizing the habitat benefits of the site. Based on this, the full restoration alternative (Alternative C) for the interior project area was the selected alternative.

For the purposes of modeling the MTRs, it was assumed that the gates will be set to close when water reaches an elevation of 6.5 feet. These model scenarios will be modified once the project team and affected landowners come to consensus on Water Management Plan that includes seasonal target closure heights (see Section 7.0).

5.2 ROAD REALIGNMENT

The horizontal alignment of Green Acres Lane to the west of the new bridge will be realigned to bring this section of roadway and the intersection with Upper Loop Rd to current traffic design standards and to provide a buffer between the edge of roadway and the active channel. This will help to alleviate erosion and road slumping issues on Green Acres Lane associated with the road being too close to Noble Creek. The realigned roadway will replace a portion of pastureland that likely contains wetlands (not currently delineated). The portion of roadway abandoned by this realignment is proposed to be graded down to the same elevation as this pasture to create new wetlands as an offset to the potential wetlands lost. Once the road is moved the existing creek bank will be armored with rock slope protection materials with log structures installed to prevent further lateral channel migration and provide additional habitat benefit.

Additional geotechnical investigation was done by Pali to characterize the subsurface soil conditions, calculate stability, and make recommendations for roadway construction. A second geotechnical report focused on this component of the design is included in Appendix B. The investigation indicates that the soil consistency is similar to the consistency of other soils in the area and that settlement should be anticipated for any new fill placed above existing grade. Seismic analysis of the road fill indicates that the road would not be stable in an earthquake regardless of near surface fill treatments and that seismic stability could only be achieved with deep foundations similar to those for the bridge. This issue was discussed with the County Engineer and a determination was made that designing the road for seismic stability would be cost prohibitive for this project. Further discussion will be needed in later phases of the design to discuss how to address road settlement issues.

5.3 INTERIOR CULVERTS AND TIDE GATES

The selected alternative includes: (1) installing one new interior culvert through the Noble Creek levee, (2) removing an existing culvert in the Noble Creek levee, and (3) upgrading one culvert and tide gate under Bramble Lane (**Figure 22** and **Figure 23**). The interior culverts have the following design criteria:

- **Culvert through Noble Creek levee (Appendix A, Sheet C6):** This culvert will be a 7-foot diameter HPDE culvert. For the 60% designs, it is assumed the culvert would be equipped with a side hinge tide gate and a muted tidal regulator (MTR) to allow for control over the seasonal extents of floodplain inundation allowable as determined by the water management plan. However, the need for a tide gate and MTR is not yet certain, and this decision will be based on ongoing discussions with multiple landowners, future modeling results, and the final water management plan.

- **Culvert removal near Bramble Lane Bridge:** A tide gated culvert of unknown diameter on river left just downstream of Bramble Lane Bridge will be removed or abandoned in-place, because the associated ditches draining to this culvert will be filled and all flow will be directed to the Noble Creek Levee Culvert described above. At the location where the levee will be excavated to remove the culvert, an engineered log structure (ELJ) will be constructed (Sheet in Noble Creek to provide instream habitat complexity (Sheet C12).
- **Culverts and tide gates along Bramble Lane (Appendix A, Sheet C9):** Multiple culverts and tide gates will be replaced in the area around two driveways south of Bramble Lane, in the eastern portion of the project area. An existing 30-inch diameter culvert and tide gate on Bramble Lane to the east of the Bramble Lane bridge will be replaced with a 4-foot diameter culvert with side hinged tide gate and an auxiliary door (i.e. pet door) that provides additional flow through the gate when the gate is closed. Several other road crossing culverts will be replaced under private driveways to provide fish passage into adjacent tidal wetlands and large wood will be added where accessible to improve habitat complexity.

5.4 LEVEES AND BERMS

The existing levee along Noble Creek in the vicinity of the Foreman property is deteriorating thereby exposing the residence on the property to flooding risk. This risk will continue to increase as the levee and tide gate infrastructure continue to deteriorate. The low point elevation of this levee is approximately 8.9 feet. The geotechnical analysis indicates that increasing the height of these levees could further weaken the underlying soils. Given these constraints, Waterways recommends that levee repair should be limited to widening existing levees to more stable side slopes, but to minimize increases to top elevations over existing conditions. These improvements are shown on sheets C11 and C18 in Appendix A. This should also be considered when establishing the water management plan.

The 60% designs also show two new levees/berms on the interior of the floodplain to minimize inundation on working lands. One of these features is shown along the southern edge of the Foreman property and the other is shown around a small parcel adjacent to the Bramble Lane Bridge. Based on the geotechnical report, both of these features are likely to experience settlement. Top elevations of these features will be confirmed as part of the 90% designs and final water management plan.

5.5 FLOODPLAIN HABITAT ENHANCEMENT

Floodplain habitat enhancement will include a variety of measures to expand and improve the habitat value of the newly accessible tidal floodplain. The primary species and life stage targeted by these enhancements is juvenile coho salmon rearing habitat, but other fish, amphibian, and bird species will also benefit.

The primary feature of the floodplain habitat enhancement is earth movement, including excavation of new sinuous tidal channels that mimic natural channels, and using the excavated material to fill sections of existing straight ditches. Tidal pools will be excavated in historic channel meanders to

provide off-channel rearing habitat, especially within an abandoned meander of Noble Creek that is likely to provide the greatest habitat uplift in the project area (see Sheet C11).

Logs with root wads will be installed throughout the channel network and ponds to provide additional shade, cover, and habitat complexity. In the current version of the 60% designs, logs are shown mostly as individual or paired root wads driven into the surrounding ground. However, modifications of the placement pattern of wood should be revisited in the next phase of design, as it may be more beneficial to concentrate the wood into fewer, larger structures that could contain larger amounts of slash.

The revegetation plan as shown in the current version of the designs is limited to seeding and planting along the newly constructed levees (Sheet C18). A robust and extensive planting of the large lowland floodplain and tidal channels with willow and other native species was not included in the scope of work for this design. While establishing a extensive network of native plants would be a very large benefit to multiple species, it is not currently known whether landowners and funders would accommodate an extensive willow planting, monitoring, and adaptive management program. Furthermore, it is unclear whether woody vegetation could become established in these elevations, soils, and anticipated hydrology. This is an aspect of the project that should be further discussed and developed in future design phases.

6.0 PROPOSED CONDITIONS HYDRAULIC MODELING

At the 30% stage of design, the proposed post-project conditions were modeled to better understand how replacing the tide gates at Green Acres Lane and reconnecting the tidal basin behind it would impact flow and inundation patterns in the Noble Creek Tidelands. While the 60% plans in Appendix A have been modified in places, it is not anticipated that those modifications will have substantive changes to the extent and frequency of inundation and flows. Because of the complexity of the model, long run times, and extensive interpretations, no new modeling was done for this version of the designs. Instead, the design team anticipates updating the Proposed Conditions modeling associated with the 90% designs, when the designs will be close to final, and the Water Management Plan has been decided upon. Therefore, the descriptions below refer to the Proposed Conditions model that reflects the 30% version of the design drawings.

In the Proposed Conditions model, the leaking tide gates at Green Acres Lane are replaced by two parallel culverts with the same size and configuration as shown in the 30% Designs (Appendix A, Sheet C4). Because HEC-RAS software does not include any routines for operating the tide gates as Muted Tidal Regulators, some additional code was written to simulate the operations of an MTR on each of the two culverts. In the model, the MTR was set to an elevation of 6.5 feet – which means that the gate closes when the water level reaches 6.5 feet elevation and is forced open when the water level is below that height. Using this feature, the model was run using measured water levels from Isthmus Slough.

The model was run for the following scenarios to understand the range of potential conditions:

- Typical dry season tidal cycle (May 10-12, 2022)
- Typical winter tidal cycle (January 16-21, 2022)
- King tide cycle (January 2-5, 2022)
- Typical winter tidal cycle in which Noble Creek is flooding (2-, 10-, and 100-year floods)

Appendix D contains a set of side-by-side map figures comparing the maximum water depths and inundation patterns for each of these scenarios, and the implications of these results are discussed in this section.

Important Note: The results presented here reflect a simulated project that does not include any interior flood control measures such as interior tide gates or levee upgrades. As a result, the model results may show more flooding than what is acceptable to landowners. These results are meant to inform a discussion between CSWCD, ODFW, landowners and the County about the amount, locations, and timing of flooding that is acceptable. Once stakeholders clarify the answer to this question, additional measures to reduce flooding may be incorporated into later stages of the designs.

6.1 PROPOSED CONDITIONS MODEL SET UP

To develop the Proposed Conditions model, we made the following modifications to the Existing Conditions Model to reflect features of the proposed designs:

- Added two new 8-foot x 10-foot tide gates equipped with Muted Tidal Regulators (MTR) at the Green Acres Lane crossing, with an assumed MTR setting of 6.5 feet (NAVD 88)
- Added a new 7-foot diameter interior culvert without a tide gate under the Noble Creek levee on river left on the Foreman property.
- Added a new 4-foot diameter interior culvert without a tide gate under Bramble Lane at the upstream end of the project area.
- Existing ditches were filled and new channels excavated as depicted in the Design Drawings (**Appendix A**, Sheet C9)

Modeling Muted Tidal Regulator. A primary focus for the model was to understanding flows and inundation patterns after replacing the existing failing tide gates with two new tide gates equipped with Muted Tidal Regulators (MTR). The MTRs are equipped with a float and activator arm to fully open to allow fish passage at lower tides and shut when water reaches a set elevation to prevent flooding. Modeling an MTR in HEC-RAS required developing additional code outside of the pre-programmed default settings. In this application, MTRs were simulated using Storage Area (SA)/2D flow connection in combination with a rules-based gate condition. The HEC-RAS default tide gate settings allows simple tide gates to be modeled with flap gates that block flow from either the upstream or downstream direction. To simulate an MTR, which is a tide gate that shuts at a specified water elevation, the gate needs to open and close across multiple tidal cycles based on the float valve elevation. To accomplish this a SA/2D connection was created at Green Acres Lane to connect the exterior (Isthmus Slough) to the interior (Noble Creek). A tide gate with appropriate dimensions and invert elevations was added to the SA/2D connection. After defining the gate, a ‘rules’-based boundary condition allowed for code to be written that determines when the gate would be open and shut. For the purpose of the model, the float valve was located on the Isthmus Slough side of the gate. The primary variable determining whether the gate was open or closed in the model was the tidal elevation in Isthmus slough (note: in the actual design calls for the float and activator to be on the Noble Creek side of the tide gate).

For the current set of runs, the MTR was set at elevation 6.5-feet (NAVD88). Therefore, in these runs the MTR remained closed during high tide periods above elevation 6.5-feet, then opened on the receding tide when the tidal elevation fell below 6.5-feet. A secondary condition was written into the code specifying that if the water surface elevation on the Noble Creek side is greater than the tidal elevation in Isthmus Slough, due to flows in Noble Creek, the MTR remains open to allow water to drain. If water levels on the Noble Creek side are higher than the Isthmus Slough side, this will push the gate open to allow water to drain towards Isthmus Slough. If this condition was not added to the model, water could accumulate behind the Noble Creek tide gates when a flood flow coincides with high tides.

The model runs shown below reflect the 30% design drawings, as described above.

It is important to note that this version of the model did not reflect important flood control measures that are included in the 60% design drawings, including a tide gate with MTR and a new levee on the Foreman property to prevent flooding and moderate interior water levels. Therefore,

the results below show more flooding than would occur if the project was constructed as designed in Appendix A.

6.2 PROPOSED CONDITIONS MODEL RESULTS

6.2.1 Summer Low and High Flows

Low Tides. Figure 26 compares the Existing Conditions and Proposed Conditions model results for a low tide scenario. During low tides, the water depths and inundation extents are similar periods, with relatively little change because of the project at low tide. Generally, at low tides there will be one to several feet of water in the main channel, ditches, and floodplain channels, regardless of the project. There will be slightly more areas of inundation under Proposed Conditions, but this is mostly because of standing water in the floodplain left over from the previous high tide.

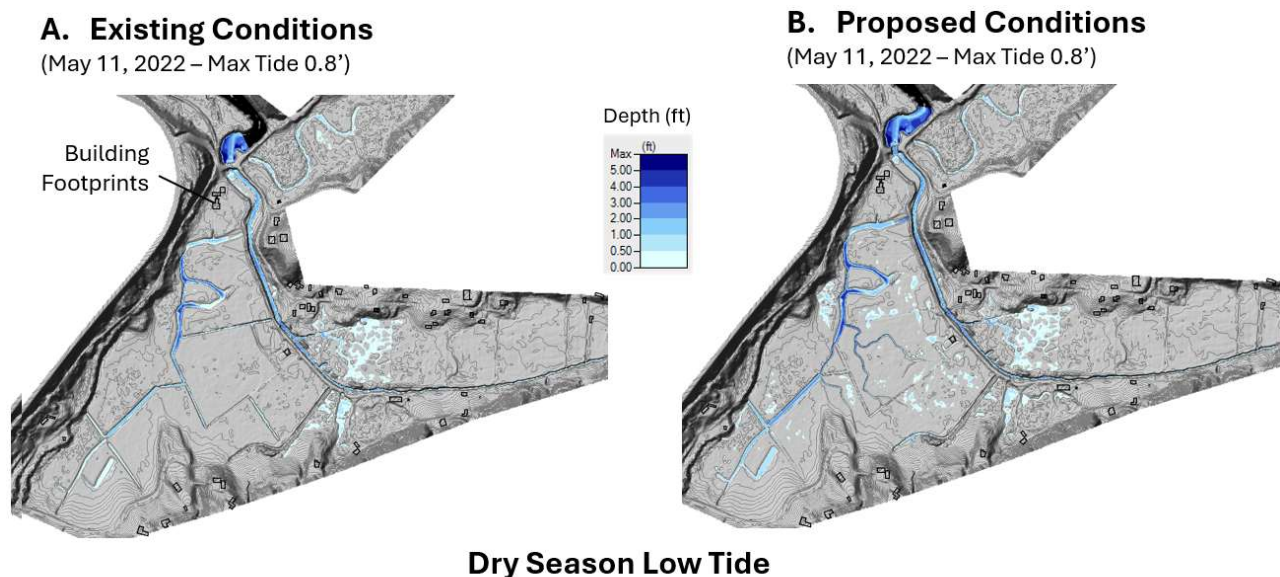


Figure 26. Modeled Water Depths During a Typical (+0.8') Dry Season Low Tide

Dry Season High Tides. Each day, there are typically two high tides with different amplitudes. To understand how flooding on these fields would be impacted during the dry (grazing) season, model results are presented below for both “lower high” and “higher high” tides during the dry season.

During a “lower high” tide of 5.85 feet elevation (**Figure 27**), there would be a significantly more inundation under proposed conditions. By design, the project would allow more water into the floodplain and open up more of the Noble Creek Tidelands for fish rearing. During a lower high tide, much of the floodplain west and south of Noble Creek would have 0.5 to 2 feet of water. The newly excavated tidal channels would have as much as 6 feet of water, providing substantial off-channel tidal rearing habitat for salmonids. Most of this water would enter the floodplain through the 7-foot culvert

under the Noble Creek Levee on the Foreman property, where inflows would exceed 100 cfs during incoming tides. During a lower high tide in the dry season, there would also be slightly more inundation in the tidal wetland to the north of Noble Creek across from Bramble Lane, as a result of the increased flow entering Noble Creek through the main GAL tide gates.

The modeled inundation patterns during a “higher high” tide of about 7.2 feet elevation (**Figure 28**) resembles the pattern during the lower high tide. Although the maximum tide is almost 1.5 feet higher, there is only an incremental increase in the amount of flooding. During the dry season higher high tide, the model shows the high tide water approaching, but not reaching, the Foreman house near the main Noble Creek tide gate. If the house and the adjacent field need additional protection, this could be provided by building a low berm along the north side of the channel conveying water into the floodplain.

Counterintuitively, the model shows reduced flooding in the zone to the tidal area north and east of Noble Creek under this scenario compared with existing conditions (**Figure 28**). The reduction in inundation in that portion of the project area is because so much of the flow that enters through the GAL tide gates would flow into the floodplain south of Noble Creek, leaving less water in the channel to enter that wetland.

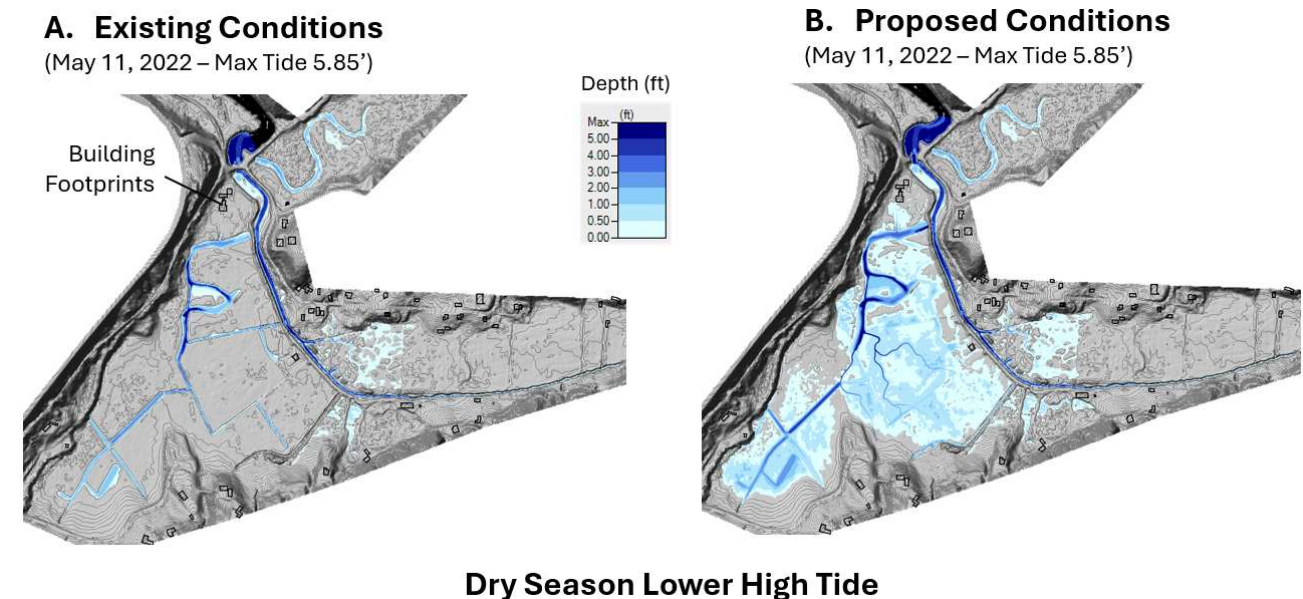
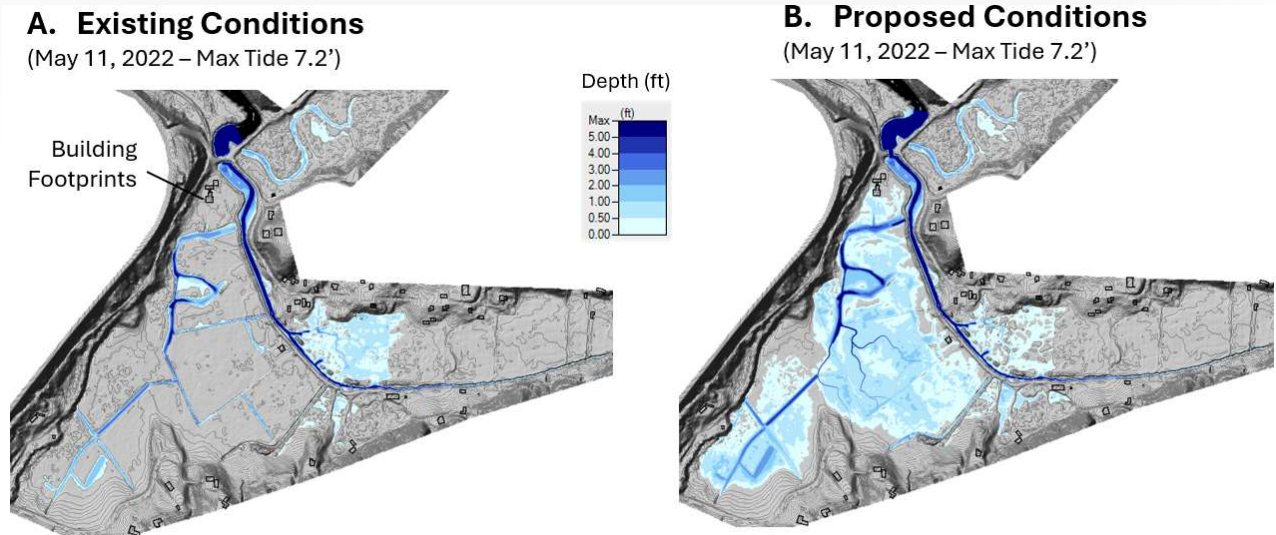


Figure 27. Modeled Water Depths During a Typical Dry Season Lower High Tide (5.85')



Dry Season Higher High Tide

Figure 28. Modeled Water Depths During a Typical Dry Season Higher High Tide (7.2')

6.2.2 Winter High Tides and King Tides

Typical Winter High Tide. Figure 29 shows the maximum inundation in a typical winter high tide of about 8.8 feet elevation. Under existing conditions, there is a distinct difference between the extent of flooding in the winter high tide (8.8 feet) compared with high tide in the dry season (7.2 feet): in the 8.8-foot tide, the relict channel and ditch network overtops and spreads out onto the floodplain, whereas in the 7.2-foot high tide, water remains mostly confined to channels and ditches (**Figures 28B and 29B**). Under proposed conditions, however, the extent of inundation for the two high tides is nearly identical (**Figures 28B and 29B**). The reason for this discrepancy is that under proposed conditions, water is more freely exchanged with the floodplain and water will flood nearly all the land below the high tide level in Noble Creek, which is limited by the MTR. Under existing conditions, the extent of flooding is controlled more by the amount of water that flows through the leaking tide gates, because the tide recedes long before the floodplain fills with water. The amount of leakage depends on head differences between inside and outside the main and interior tide gates, which is greater during a higher tide cycle.

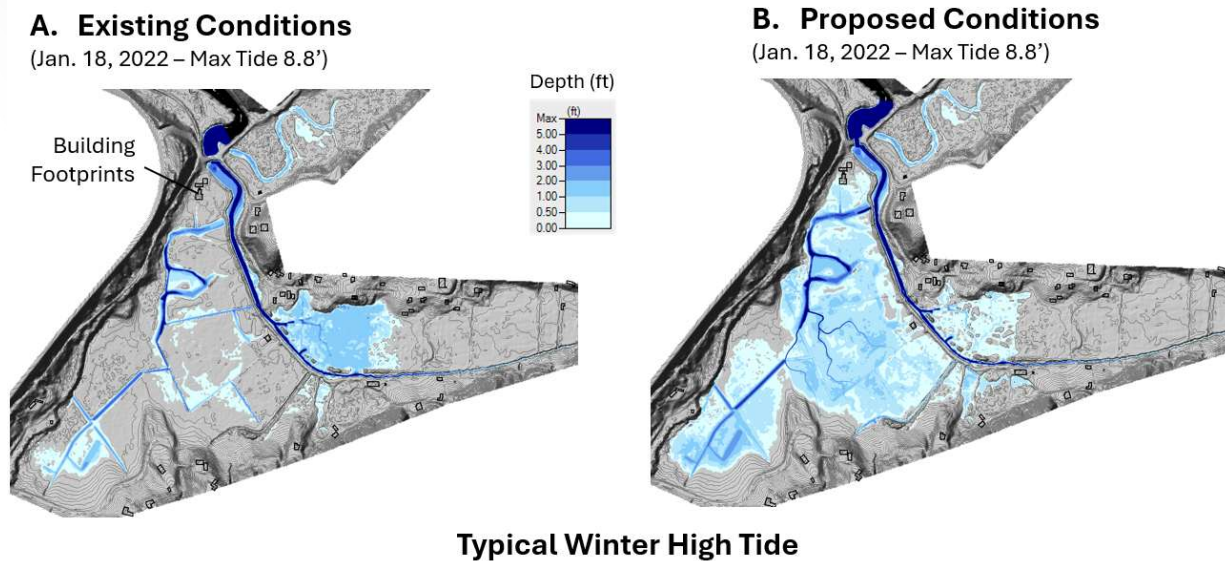


Figure 29. Existing and Proposed Conditions for a Typical Winter High Tide Scenario

King Tide. Figure 30 shows the extent of flooding during a king tide, as measured on January 3, 2022 in Isthmus Slough. The extent of modeled flooding under existing and proposed conditions is very similar, and both show flooding at the Foreman house. However, the mode of flooding is different; under existing conditions, according to the model, water would overtop the road to the north of Noble Creek in a king tide, which would allow floods to overtop the Noble Creek levee and enter the large floodplain south of Noble Creek. In contrast, under proposed conditions enough water would flow through the new culvert that the levee would not overtop.

It is not clear whether the Existing Conditions model is correct in predicting that the road would overtop and flooding occur in very high tides. This may be accurate, or it may be due to a model error in predicting water elevations at king tides. A small error in the predicted water elevation could cause the model to show widespread flooding, even if it does not actually occur. It would be helpful to know from locals whether the road overtopped during the 2022 king tide event, and if there was as extensive flooding as shown in Figure 30A.

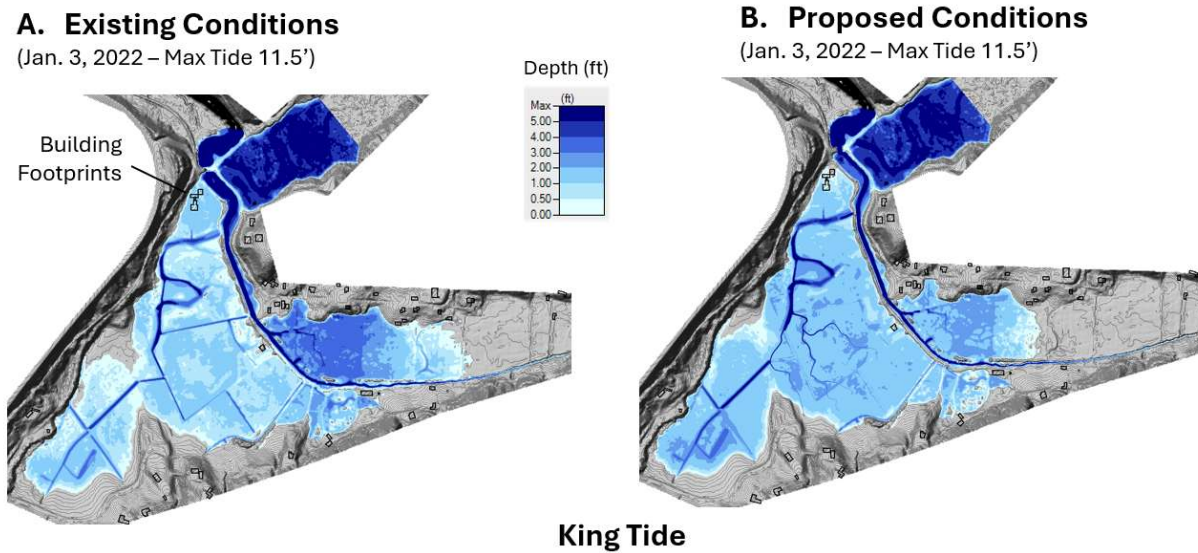


Figure 30. Existing and Proposed Conditions for a King Tide Scenario

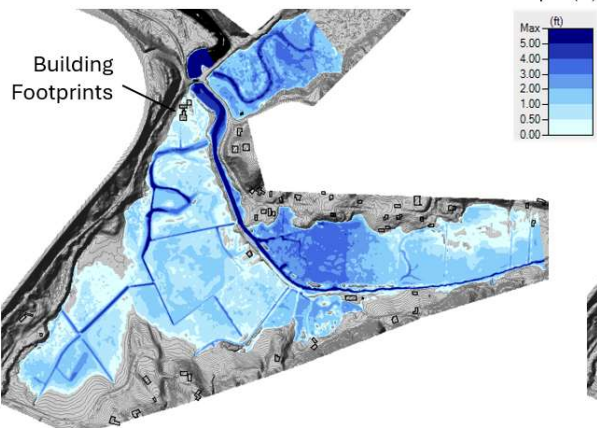
6.2.3 Flooding in Noble Creek Coinciding with a Winter High Tide

A final set of model runs simulated how flooding in Noble Creek would affect inundation throughout the project area. For these model runs we assumed that the flood peak in Noble Creek coincided with the winter high tide (8.8 feet). This is considered conservative, because if the flood peak and high tide coincide this would increase the amount of flooding, since Noble Creek flows would not be able to exit the GAL tide gates. This flooding scenario is also conservative because of the flows used for Noble Creek. Two methods were used to develop the 2-year, 10-year, and 100-year floods: (1) the USGS program StreamStats, which uses basin characteristics and regional regressions to estimate peak flows, and (2) data from a stream gage on a nearby creek with a similar drainage area and topography (Johnson Creek in Bandon, operated by Oregon Department of Water Resources). Flows estimated using StreamStats were almost twice as much flow as those estimated using the ODWR gage. Therefore, it is likely that the flows used for this analysis overestimate the amount of water in Noble Creek.

Figures 31, 32, and 33 show the predicted inundation maps for the 2-year, 10-year, and 100-year floods in Noble Creek. Based on the model, the inundation extent for existing and proposed conditions would be similar; however, water depths would be generally deeper in the flooded areas under proposed conditions. Under existing conditions most of the flow entering the tidelands would be over the road and levee; under proposed conditions, the levees would not overtop as much but instead, flow would enter the floodplain through the 7-foot culvert under the Noble Creek levee.

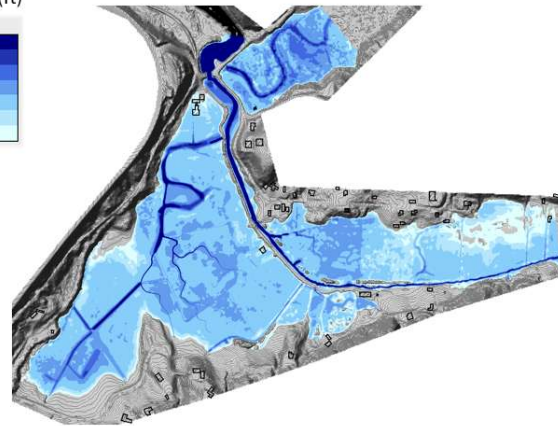
A. Existing Conditions

(Jan. 18, 2022 – Max Tide 8.8')



B. Proposed Conditions

(Jan. 18, 2022 – Max Tide 8.8')



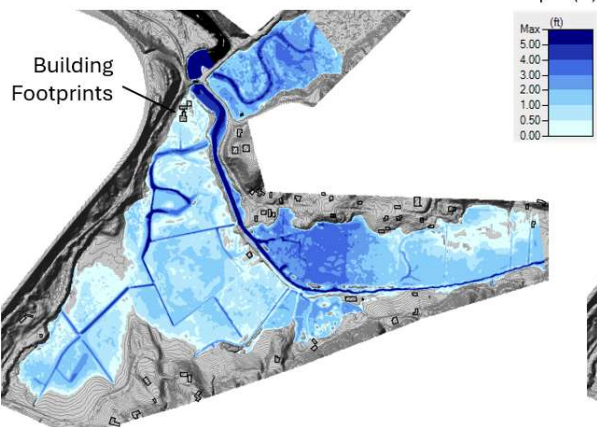
Typical Winter High Tide With 2-Year Flood* in Noble Creek

*Peak Flow in Noble Creek 294 cfs, based on StreamStats (which is 2X estimate based on nearby gage)

Figure 31. Existing and Proposed Conditions for a 2-Year Flood in Noble Creek Coinciding with a Typical Winter High Tide

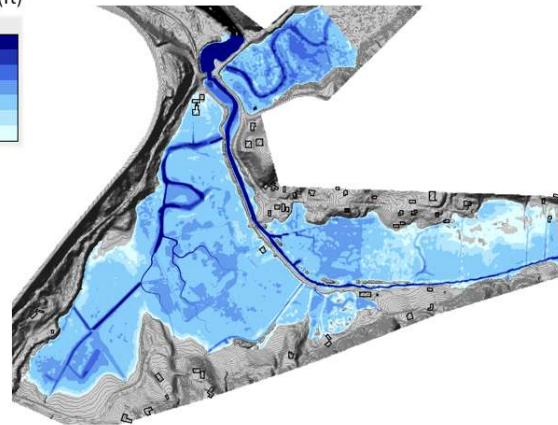
A. Existing Conditions

(Jan. 18, 2022 – Max Tide 8.8')



B. Proposed Conditions

(Jan. 18, 2022 – Max Tide 8.8')



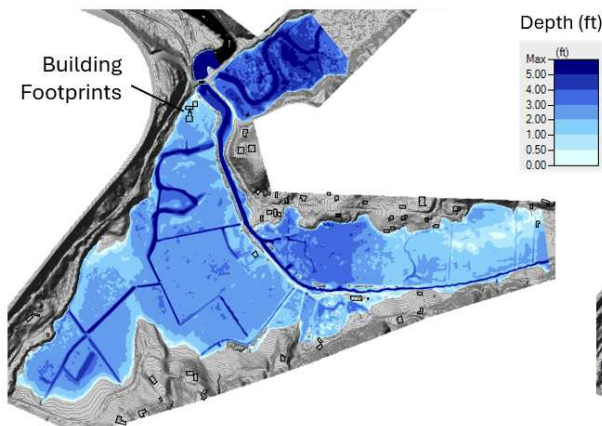
Typical Winter High Tide With 2-Year Flood* in Noble Creek

*Peak Flow in Noble Creek 294 cfs, based on StreamStats (which is 2X estimate based on nearby gage)

Figure 32. Existing and Proposed Conditions for a 10-Year Flood in Noble Creek Coinciding with a Typical Winter High Tide

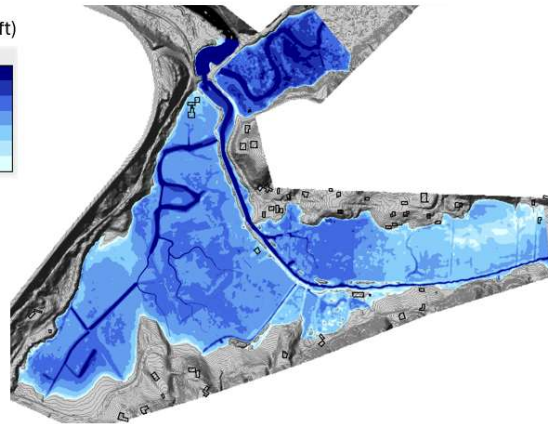
A. Existing Conditions

(Jan. 18, 2022 – Max Tide 8.8')



B. Proposed Conditions

(Jan. 18, 2022 – Max Tide 8.8')



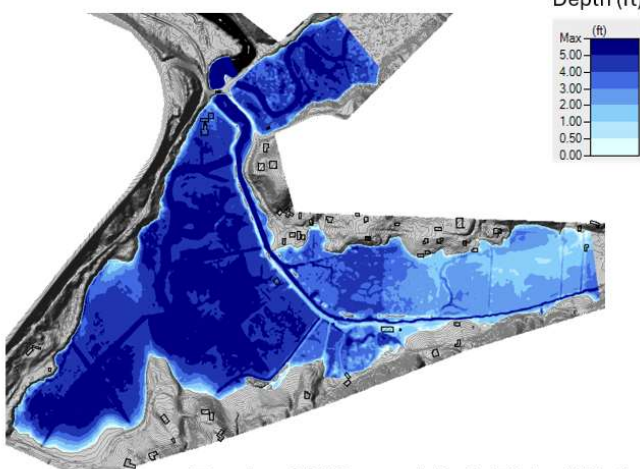
Typical Winter High Tide With 10-Year Flood* in Noble Creek

*Peak Flow in Noble Creek 580 cfs, based on StreamStats (appears high based on nearby gage)

Figure 33. Existing and Proposed Conditions for a 10-Year Flood in Noble Creek Coinciding with a Typical Winter High Tide

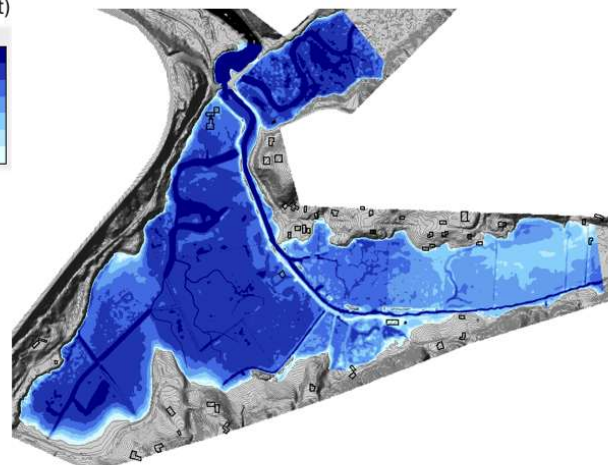
A. Existing Conditions

(Jan. 18, 2022 – Max Tide 8.8')



B. Proposed Conditions

(Jan. 18, 2022 – Max Tide 8.8')



Typical Winter High Tide With 100-Year Flood* in Noble Creek

*Peak Flow in Noble Creek 1,000 cfs, based on StreamStats (appears high based on nearby gage)

Figure 34. Existing and Proposed Conditions for a 100-Year Flood in Noble Creek Coinciding with a Typical Winter High Tide

6.3 PRELIMINARY FISH PASSAGE ANALYSIS

Because the project will involve installing at least one new tide gate at the Green Acres Lane crossing, the project will need to meet Oregon Department of Fish and Wildlife (ODFW) requirements for fish passage at new tide gates. Current tide gate replacement fish passage criteria in Oregon (OAR 635-412-0035) require new tide gates to meet the following criteria:

- A. Be a minimum of 4 feet wide unless the natural channel conditions are less than 4 feet wide;
- B. Consist of an aluminum tide gate door or other equivalent light weight material;
- C. Be a side hinged door configuration;
- D. Meet the requirements of OAR 635-412-0035(2) or 635-412-0035(3)(b) within the design streamflow range and for an average of at least 51 percent of tidal cycles, excluding periods when the channel is not passable under natural conditions;
 - i. Design streamflow range shall include tidal exchange, freshwater stream discharge and water storage volumes draining to the tide gate;
 - ii. Design streamflow range should consider sub-surface flows if appropriate at the project location;
- E. Design invert elevation of tide gate and associated structure to be placed at 1 foot below Mean Lower Low Water elevation or as otherwise appropriate for the site to prevent perched low flow fish passage conditions and allow proper tide gate function;
- F. Consider the use of pet doors, mitigators, self-managed and self-regulating tide gate devices to maximize fish passage, time of tide gate door openness, water exchange, and tidal inundation if the tide gate is associated with high priority restoration habitat; and
- G. Submit a water management plan for projects implementing self-managed or self-regulating devices.

The designs in **Appendix A** meet requirements A, B, C, E, and F, and any revised design drawings will also meet those requirements. A water management plan (criteria G) will be developed by CSWCD and ODFW based on a later (60%) version of the design.

The designs must also meet criterion D, which requires the tide gates remain open and water velocities below 2 feet per second for 51% of the time on average. At the present (30%) level of design, there are still some significant uncertainties related to fish passage. For one, it is not certain whether interior tide gates will be included in the project. In addition, some of the details of the main tide gates at Green Acres Lane may still change from the 30% plans. Therefore, a more thorough analysis of fish passage permitting will be prepared based on the 60% designs.

To ensure that the next phase of designs will not have to be drastically modified to meet ODFW guidelines, a simple informal preliminary analysis was done based on the current designs to confirm that the project is on track to provide necessary fish passage requirements. In this analysis, the model with the MTR operation code was run for the period January 22 through February 10, 2022 – a period that included a relatively large tidal amplitude ranging from more than 9.8 feet to a minimum of -0.1

feet. During this period, with an MTR gate setting of 6.5 feet, the tide gate would remain open for 83.5 percent of the model duration. Water velocities going through the tide gate opening in the sheet pile wall were estimated by dividing the modeled discharge through the opening by the cross-sectional area of flow. The modeling showed that velocities will sometimes exceed the 2 foot per second criteria, especially when the gate first opens. The maximum velocity for the model period was 2.9 feet per second. However, with a tide gate setting of 6.5 ft/s, the project will still be fish passable (i.e., gates open and velocity less than 2 feet per second) about 60 percent of the time. There is a chance that much lower MTR settings than this could make it more difficult to meet the requirements.

Based on this analysis, the next phase of design and the development of the water management plan should be done in conjunction with more modeling to ensure the fish passage criteria will be met.

7.0 ISSUES TO BE RESOLVED/NEXT STEPS

The next step in the process will be to present this revised 60% level of design drawings to landowners and other stakeholders, and confirm that the plans are acceptable to all parties prior to seeking permits and construction funding for the project. It will be necessary to work with landowners, stakeholders, and permitting agencies to finalize some of the design parameters, so they meet the requirements of all parties. The following bullet list identifies the next sets of issues to be resolved prior to seeking permits, funding, and finalizing the designs:

- **Confirm Landowner Requirements for Flood Protection.** The current plans are based on multiple discussions with landowners about their needs, but those specific needs have never been clearly defined. With the current designs it will be easier for landowners to better understand the extent of the project, and define which locations need to be protected from flooding, which areas can flood, and at what frequency. The Proposed Conditions model results in **Appendix D** can be used as a communications tool to show landowners the anticipated extent and timing of flooding in the interior of the Noble Creek Tidelands that is acceptable to landowners.
- **Confirm Interior Flooding Measures.** Depending on the extent of acceptable flooding, confirm which interior flood control measures should be included in the next phase of the designs. The flooding shown in **Appendix D** will be reduced by one or more of the following measures, some of which are reflected in the 60% design drawings. Landowner and agency outreach should be done to confirm this adequately reflects a consensus on how to manage water levels in the project area:
 - a) Include an MTR on the larger 7-foot interior culvert in the Noble Creek levee on the Foreman's property (as shown in the 60% Design Drawings).
 - b) Include tide gate with an auxiliary door (pet door) on the 4-foot interior culvert along Bramble Lane (as shown in the 60% Design Drawings).
 - c) Build berms or flood walls around specific fields or structures (as shown in the 60% Design Drawings).
 - d) Reduce the MTR setting at the main tide gates at Green Acres Lane.
 - e) Reduce the MTR setting at the interior tide gate.

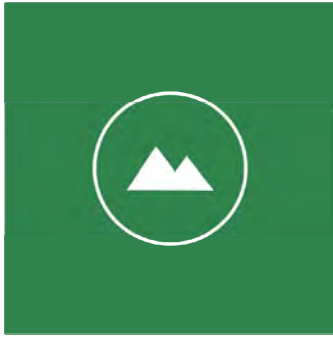
- **Agree on a Water Management Plan.** Once the interior flood protection measures are determined, the project leaders (CSWCD and ODFW) will work with landowners and designers to develop a Water Management Plan (WMP) for both the main tide gates and interior tide gates.
- **Add More and More Complex Wood Structures.** More wood structures could be added to the current version of the designs to increase uplift without significantly increasing the cost of the project. The 60% design added several engineered log jams in Noble Creek and Isthmus Slough, especially aimed at providing holding areas for fish waiting for access or egress the project area. The present version of the designs shows most of the floodplain enhancements as individual or paired logs with root wads. It may be more advantageous to concentrate the log treatments in smaller areas, creating larger and more complex jams for fish to use, rather than individual isolated logs and root wads, which may become less effective at providing shelter and shade after several years of weathering in place.
- **Decide on a Revegetation Plan.** The current Design Drawings do not include any revegetation aside from standard erosion control seeding. Once the water management plan is determined, the Project Team should assess what types of native plants are expected to survive based on the new hydrologic regime. This may include revegetation with herbaceous plugs and/or woody plant stock. Any revegetation plans should be approved by a revegetation professional familiar with native plants of the area. Extensive willow plantings along the tidal channel network, if they would be expected to establish and survive, could provide a lot of habitat value to the newly accessible tidal channels.
- **Fencing and Livestock Crossings.** The current Design Drawings do not include any fencing or livestock crossings for the proposed channels. The project leaders (CSWCD and ODFW) will work with landowners to determine the extent of fencing and locations of livestock culverts prior to 60% design.
- **Initial Consultations with Wetland and Permitting Experts.** Early consultations with agencies, and with wetland and permitting subconsultants would be critical at the current stage of design to understand the permitting pathway and any constraints that could affect the designs.

APPENDIX A

60% DESIGN DRAWINGS (INCLUDED AS SEPARATE ATTACHMENT)

APPENDIX B

GEOTECHNICAL REPORTS FOR THE NOBLE CREEK TIDELANDS PROJECT



Pali Consulting

August 21, 2023

Waterways Consulting, Inc
1020 SW Taylor Street, Ste 380
Portland, Oregon 97205

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 Tye 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 ($F_{pga} \times PGA$)	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- foot 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 and 175 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:

- 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.
- 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 $\tan(\phi)$. The passive earth pressure coefficient (K_p) 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-foot-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 $\leq 1\frac{1}{2}$ inch	Crushed Rock Maximum Particle Size $> 1\frac{1}{2}$ 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

 Project Location

Noble Creek Tideland Project
Coos Bay, Oregon

Vicinity Map
#014-21-006-2 August 2023

 Pali Consulting

Figure 1



Note:
All locations approximate. Base map ©ESRI.





Pali Consulting

Figure 2

- Exploration Locations
- Existing Levee To Be Raised
- Cross-section Locations
(See Figures 3a-3b and 4)

Green Acres / Upper
Loop Lanes Bridge &
Tidegate

B-2

B-1

Isthmus Slough

Upper Loop Ln

Green Acres Ln

B-5

B-4

Lower Loop Rd

Noble Creek

Levee Culvert
Replacement

Hwy 42

Bramble Lane
Culvert Replacement

B-3

Bramble Ln



Note:
All locations approximate. Base map ©Google.

0

250

500 ft



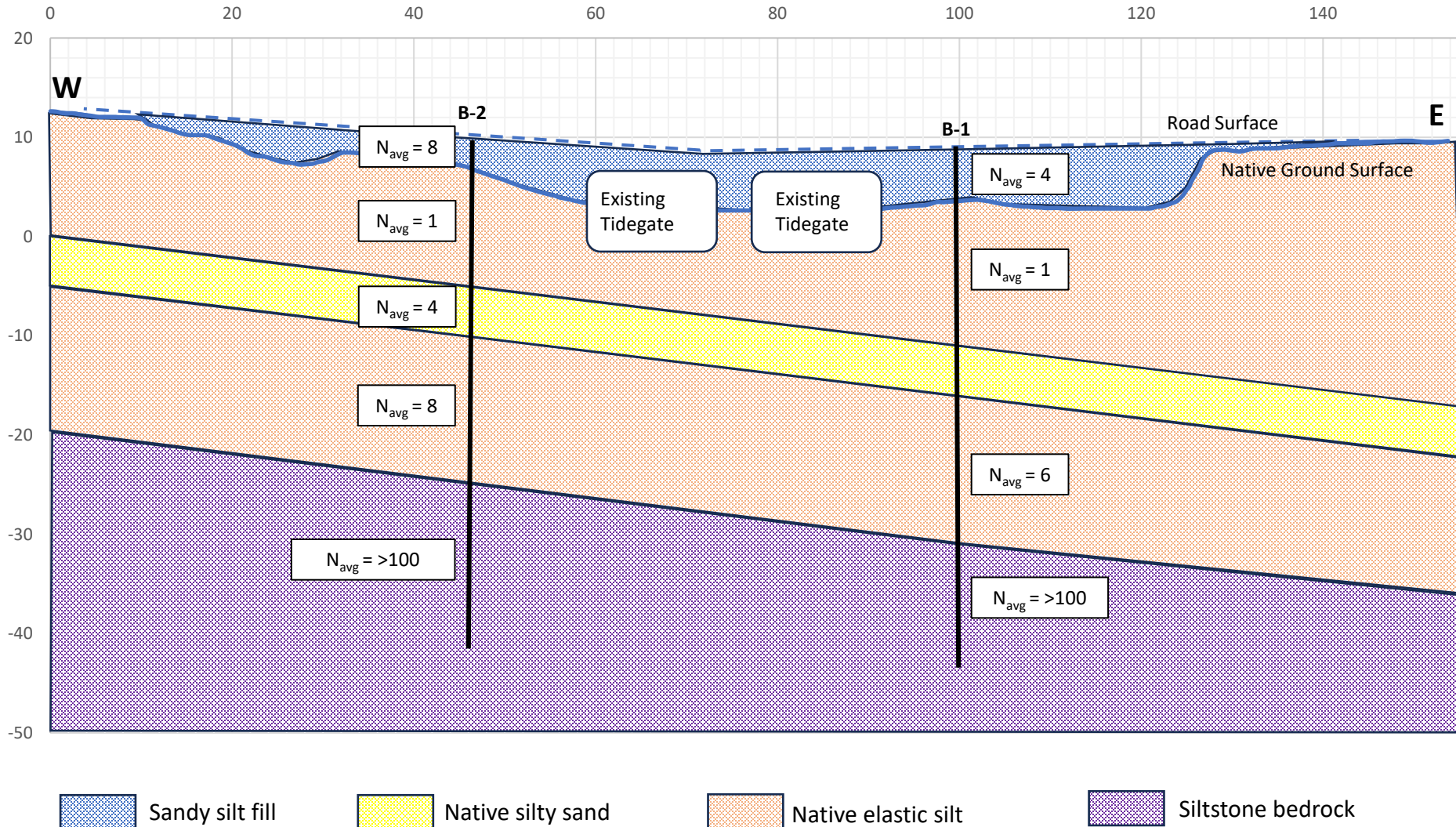
Pali Consulting

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





Pali Consulting

Figure 3b

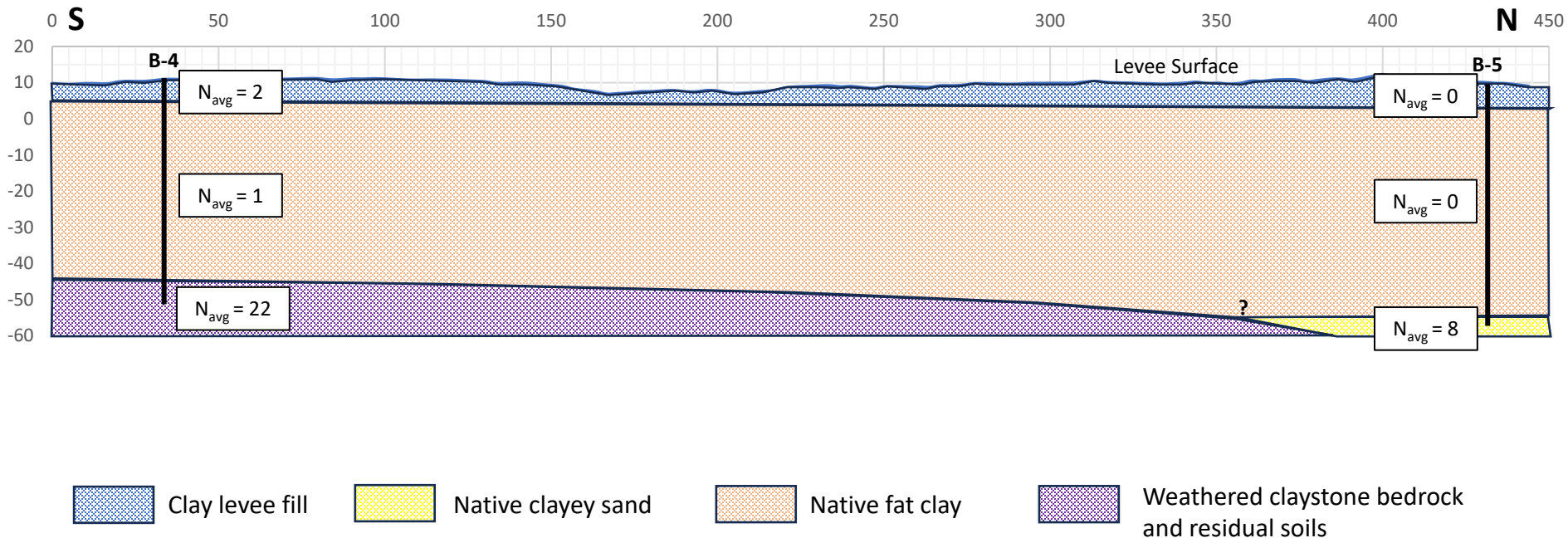
Notes: Cross sections based on LiDAR data provided by DOGAMI. All distances and elevations are approximate. Native soils are as interpreted by Pali Consulting staff, see Report and Appendix B for further description of subsurface conditions. N_{avg} refers to average SPT blow counts (see Report and Appendix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

Noble Creek Levee
Coos County, Oregon

Interpretive Cross Section

#014-21-006

August 2023





Pali Consulting

Figure 4

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

Noble Creek Levee
Coos County, Oregon

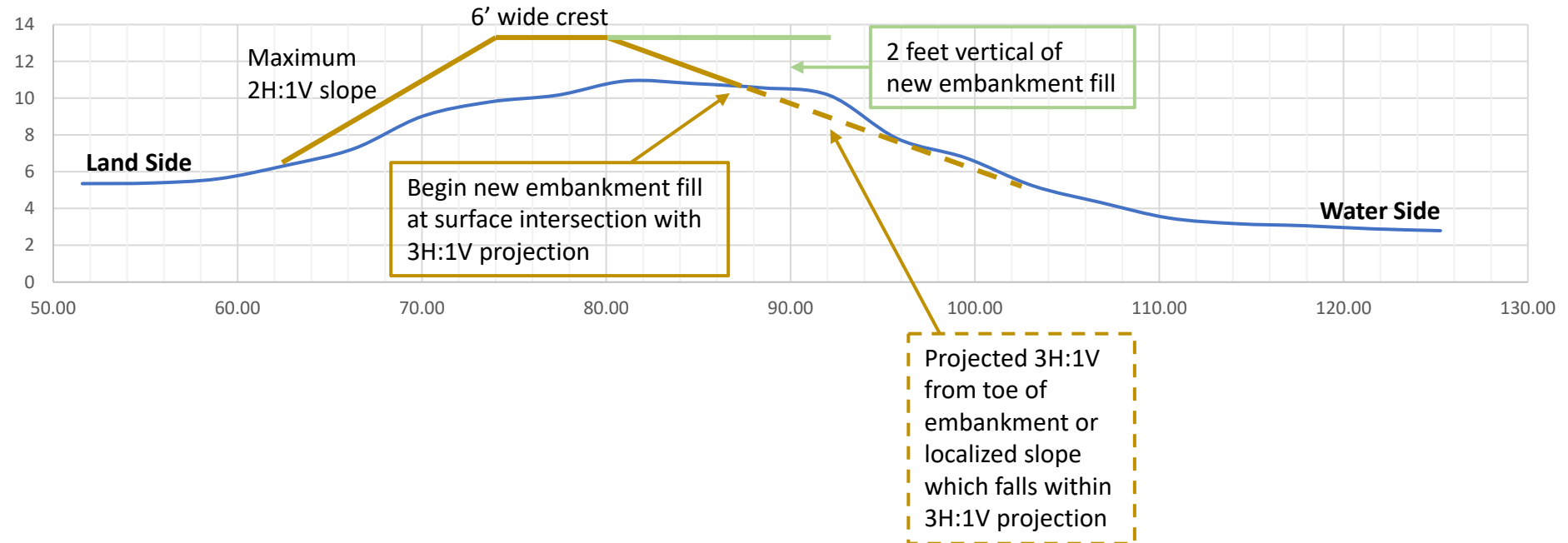
Representative Levee Section

#014-21-006

August 2023

Levee Surface

Representative Levee Section



**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

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



EXPIRES 12/31/2023

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.



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 bridge, 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 00330.15 *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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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



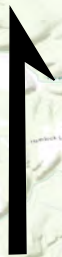
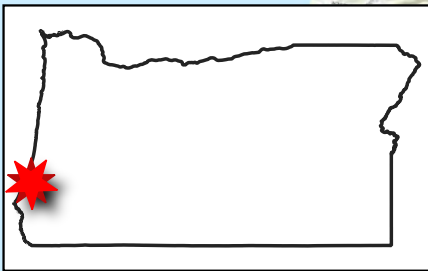
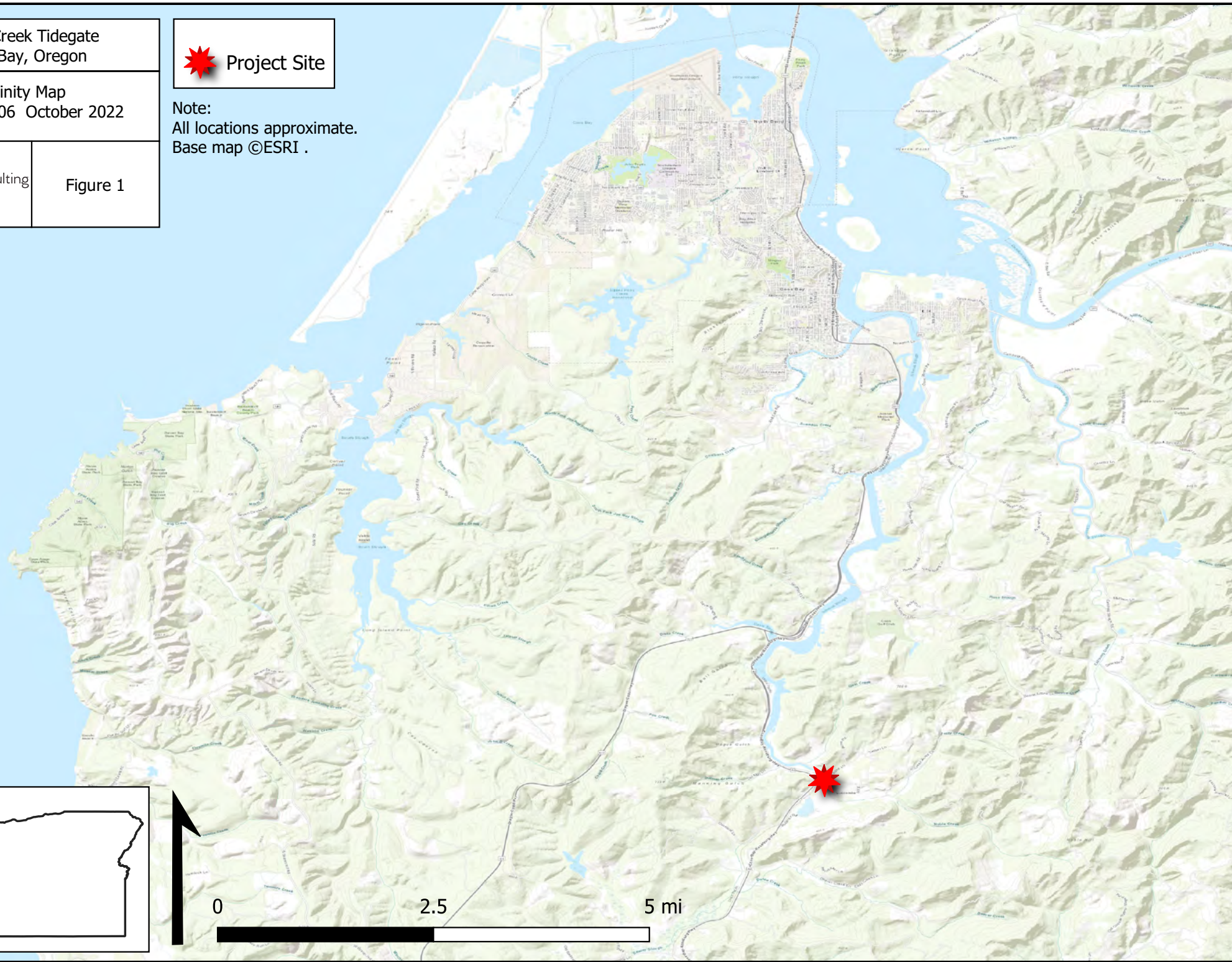
Pali Consulting

Figure 1



Project Site

Note:
All locations approximate.
Base map ©ESRI .



0

2.5

5 mi



Noble Creek Tidegate
Coos Bay, Oregon

Site Map
#014-21-006 October 2022

 Pali Consulting

Figure 2

-  Exploration Locations
-  Levee

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	TYPICAL DESCRIPTIONS
			LETTER	
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL-GRADED GRAVELS, GRAVEL - SAND - MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM	SILTY GRAVELS, GRAVELS - SAND - SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL- SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN 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 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

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

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

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

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

Depth	Remarks	Moisture (%)	Dry Density	Blow Counts	Sample Type	Water Table
0						
			3		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	1			Increasing sand
2	%F=41	62	3		SM	Loose, wet, blue-grey, clayey FINE SAND
			4		END	Grades to clayey medium sand
			4			HA-1 terminated at 2.67 ft bgs due to caving
			6			
			7			
			11			
			9			
			8			
			7			
			6			
			9			
			13			
			10			
			12			
			8			
			14			
			13			
			13			
			15			
			17			
			28			
			33			
			35			
			35			
14						

LOG OF BORING



Pali Consulting

Noble Creek
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
0					CH	Soft, moist, dark blue-grey FAT CLAY with much organics
1	120					Grades to wet
2	132					Grades to mottled light and dark grey
3					END	HA-2 terminated at 2 ft bgs due to caving
4						
5						
6						
7						

LOG OF BORING



Pali Consulting

Noble Creek
Project No. 014-21-006

A-5

HA-3

Surface Elevation: 4'

Boring Date: 8/26/22

Boring Location: Green Acres Ln Embankment

Drilling Method: Hand Auger

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

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
3					END	HA-3 terminated at 2.67 ft bgs due to caving
4						
5						
6						
7						

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

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

Depth	Remarks	Moisture (%)	Dry Density	Blow Counts	Sample Type	Water Table
0						
1	%F=71	27				
2						
3						
4		71				
5		89				
6						
7						

GWT not encountered

SM

Stiff, dry, tan, SANDY SILT with occasional small gravels, roots (levee fill)

Grades to tan with minor light grey mottling

Grades to medium stiff, tan and grey, decreasing sand

With minor rusty red mottles

Grades to moist, decreasing sand

CH

Grades to soft, mottled, brown and grey, sandy FAT CLAY with charcoal (native)

Grades to grey with rusty red mottles, decreasing sand, increasing moisture, with much charcoal and organics

Grades to wet, blue-grey

HA-4 terminated at 6.25 ft bgs due to caving

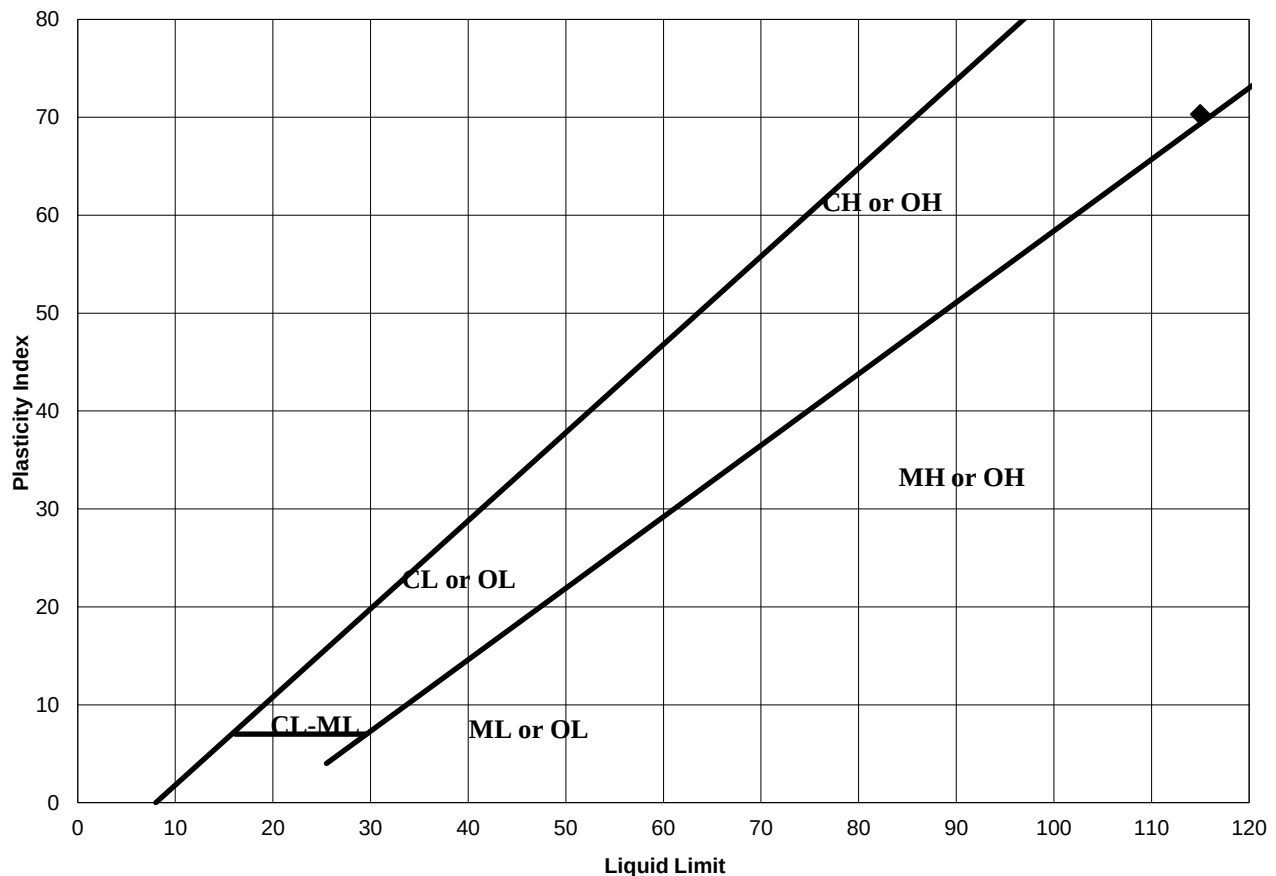
LOG OF BORING



Pali Consulting

Noble Creek
Project No. 014-21-006

A-7



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
 Plastic Limit = 45
 Plasticity Index = 70

Date: 10/13/2022
 Tested By: JLE
 Checked By: TWB

NOTE: This report may not be reproduced, except in full, without written approval of Pali Consulting. Test results are applicable only to the specific sample on which the test was performed, and should not be interpreted as representative of samples obtained at other times or locations, or generated by other operations or processes.



Pali Consulting

ATTERBERG LIMITS - ASTM D4318

Page A-8



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, Inc.
1419 Washington Street, Suite 101
Oregon City, Oregon 97045

SOIL CLASSIFICATION CHART

MATERIAL TYPES	MAJOR DIVISIONS		GROUP SYMBOL	SOIL GROUP NAMES & LEGEND		OTHER MATERIAL SYMBOLS
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

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 conditions 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	
S1		20		2-2-2	4	GWT not encountered	0		AC ML	Asphalt roadway surface underlain by 6" well graded gravel (Roadway Base Rock) Soft to medium stiff, moist, brown, SANDY SILT (FILL)	43	AL, PI=60, LL=100, PL=40 DD=27 pcf AL, PI=71, LL=120, PL=49 CS %F=64	
S2		50		0-1-1	2		5		CH	Soft, wet, grey, FAT CLAY with minor charcoal (NATIVE) Grades to dark grey	66		
S3		75											
S4		100		0-0-0	0		10		MH	Very soft, wet, dark grey ELASTIC SILT with variable grey sand	83		
S5		100											
S6		100		0-0-1	1		15			With interbedded very loose, wet, dark grey SILTY SAND, with minor charcoal and wood	83		
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 Grades to light grey	39	AL, PI=26, LL=57, PL=31	
S9		100					30						

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

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

B-2

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
S1		35		4-4-4	8		0		AC		
N/A		0		0-0-0	0		5		MH		
S2		100							SM		
S3		35		2-2-1	3		10		MH		
S4		70		0-2-2	4		15		SM		
S5		100							MH		
S6		100		1-3-5	8		20		MH		
S7		100					25		MH		
S8		100		3-4-4	8		30		MH		

Date: 8/18/2023

File: C:\Users\june.pali\Consulting\Dropbox\1-Projects\Active-Projects\014-21-006\NobleCreek\02-GeotechnicalDesign\Analysis\NobleCreekLogs.log

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		



Pali Consulting

Noble Creek Tidelands Project

Coos County, OR

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

B-3

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
S1		67		0-1-2	3		0		CL 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				

Date: 8/18/2023

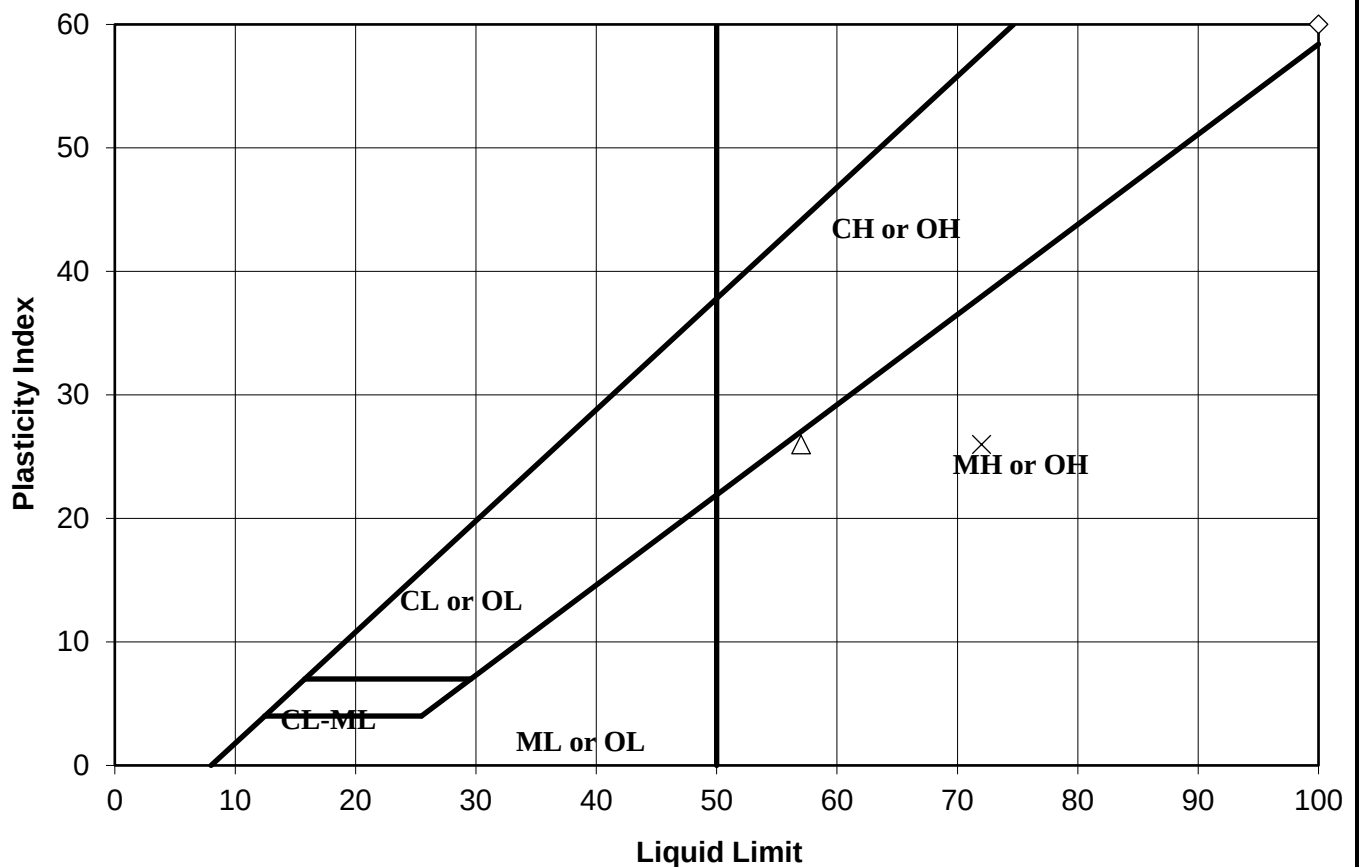
File: C:\Users\June\Pali Consulting Dropbox\1-Projects\Active-Projects\014-21-006\NobleCreek\02-GeotechnicalDesign\Analysis\NobleCreekLogs.log

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	
S1		67		0-1-1	2	GWT not encountered	0		CH Soft, moist, varicolored (yellow/red/gray/brown) FAT CLAY with minor sand and organics (LEVEE FILL) Grades to grey (NATIVE) Grades to very soft, moist to wet, grey with minor orange mottles. Increasing charcoal / organic material	54	DD=33 pcf		
S2		100					5						
S3		50		0-0-0	0		10						
S4		50					15						
S5		50					20						
S6		100		0-0-0	0		25		Grades to dark grey	113	Drillers report much wood in cuttings AL, PI=98, LL=146, PL=48 DD=22 pcf		
S7		75				30							
S8		33		0-0-1	1								
									OH Very soft, moist, dark grey to brown to black, ORGANIC CLAY (much wood and organic material)				

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	
S9		100		0-0-0	0		35		CH Very soft, moist to wet, grey FAT CLAY with minor charcoal, no organics	55	Drillers report increased difficulty drilling at 57'	
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)			
S14		100		14-17-19	36		60		Grades to extremely soft to very soft (R0-R1) sedimentary rock Boring completed at 61.5' BGS			
							65					

Pali Consulting							Noble Creek Tidelands Project Coos County, OR			B-5		
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: 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
S1		100		0-0-0	0		0		CL	Very soft, moist, varicolored (yellow/orange/gray/brown) SANDY CLAY with minor charcoal and organics (LEVEE FILL) Grades to brown-grey	53	DD=66 pcf
S2		100					5					
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
S4		0					10			Grades to dark grey, moist to wet, with increasing sand		
S5		100										
S6		100		0-0-1	1		15				76	%F=53
S7		100					20			Grades to grey. decreasing sand		
S8		100		0-0-0	0		25			Grades with increasing sand, wood, organics, and charcoal		
S9		100		0-0-0	0		30			Grades to brown-grey, with minor sand	115	

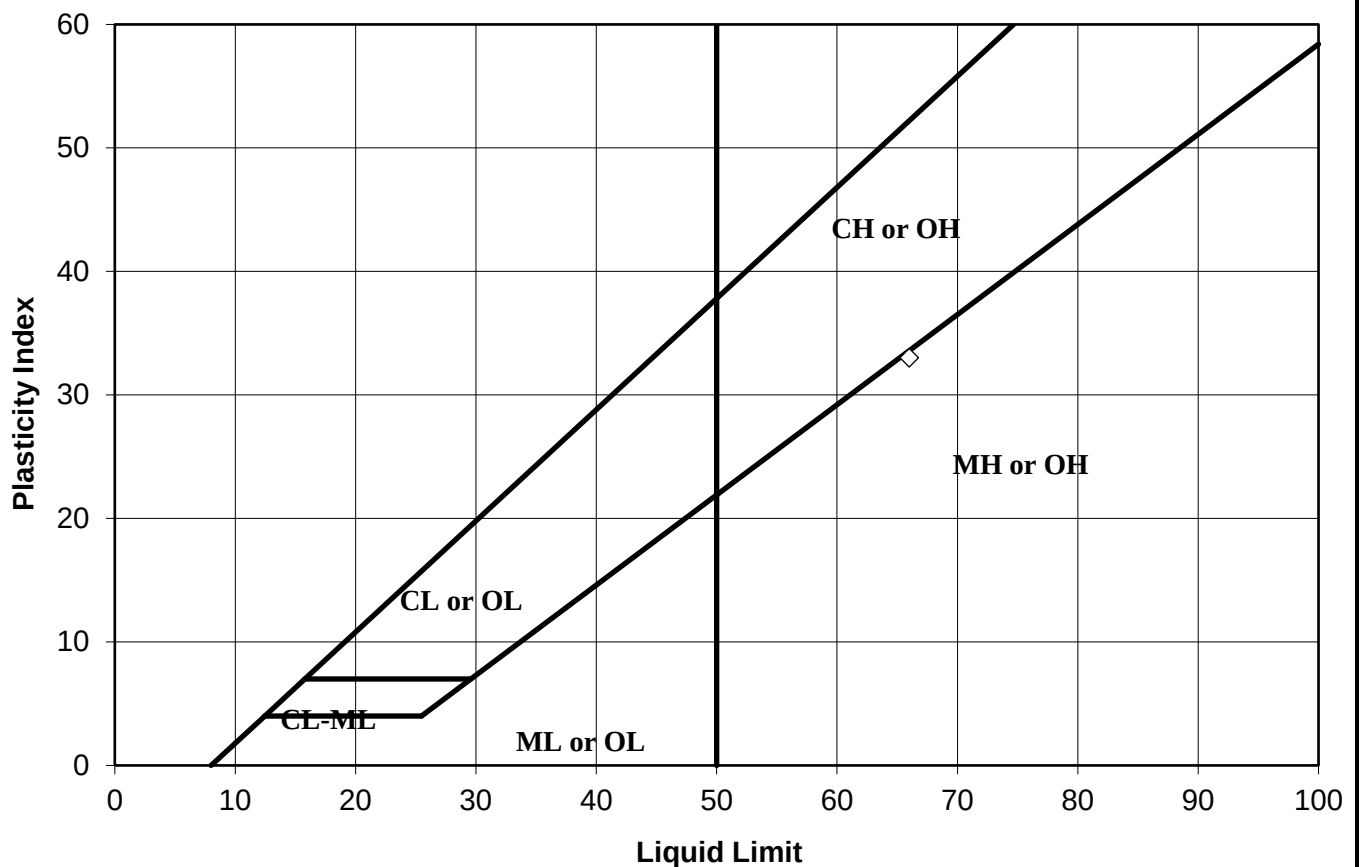
 Pali Consulting							Noble Creek Tidelands Project Coos County, OR			B-5		
Project: Noble Creek Tidelands Project							Driller: Western States Soil Conservation, Inc					
Proj No. 014-21-006-2							Date: 5/9/23					
Drilling Method: Mud Rotary							Elevation: 9'					
Diameter: 4"		Water Table: N/A					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		DD=47 pcf



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

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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
×							

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ATTERBERG LIMITS - ASTM D4318	
Noble Creek Tidelands Project Project No.: 014-21-006-2	
 Pali Consulting	Figure B-1b

TECHNICAL REPORT

Report To: Tim Blackwood, PE, GE, CEG
Pali Consulting, Inc.
1120 SW Fifth Avenue, Suite 1302
Portland, Oregon 97204

Date: 6/16/2023

Lab No.: 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.

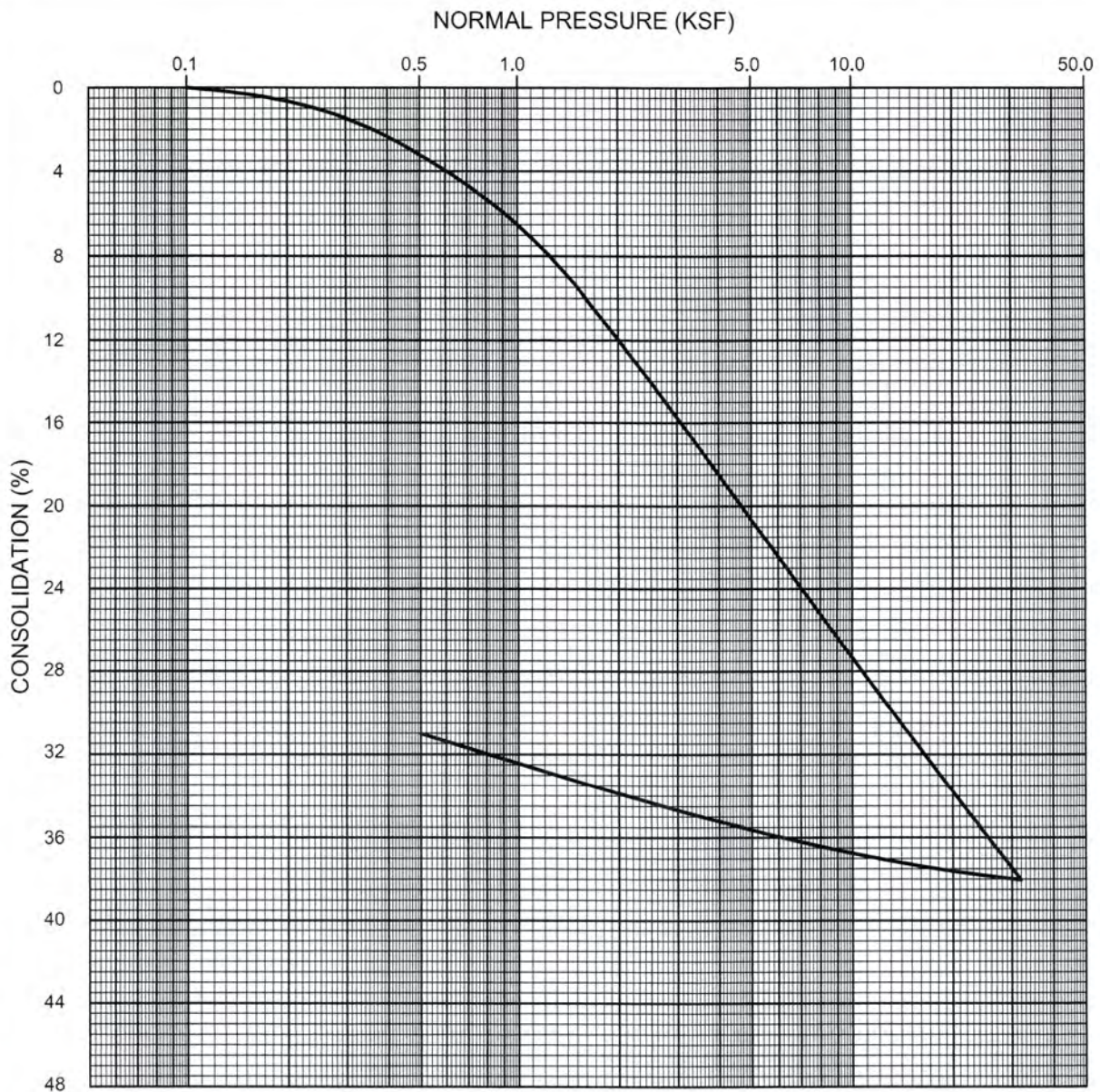
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SHEET 1 of 3

REVIEWED BY: Michael A. Ginsbach 

TECHNICAL REPORT

K:\Lab Reports\2023 Lab Reports\00-223425-1 - Pali Consulting, Inc\23-095\23-095 - One-Dimensional Consolidation with Time Rates.docx



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.
NOBLE CREEK

LAB NO. 23-095

TECHNICAL REPORT

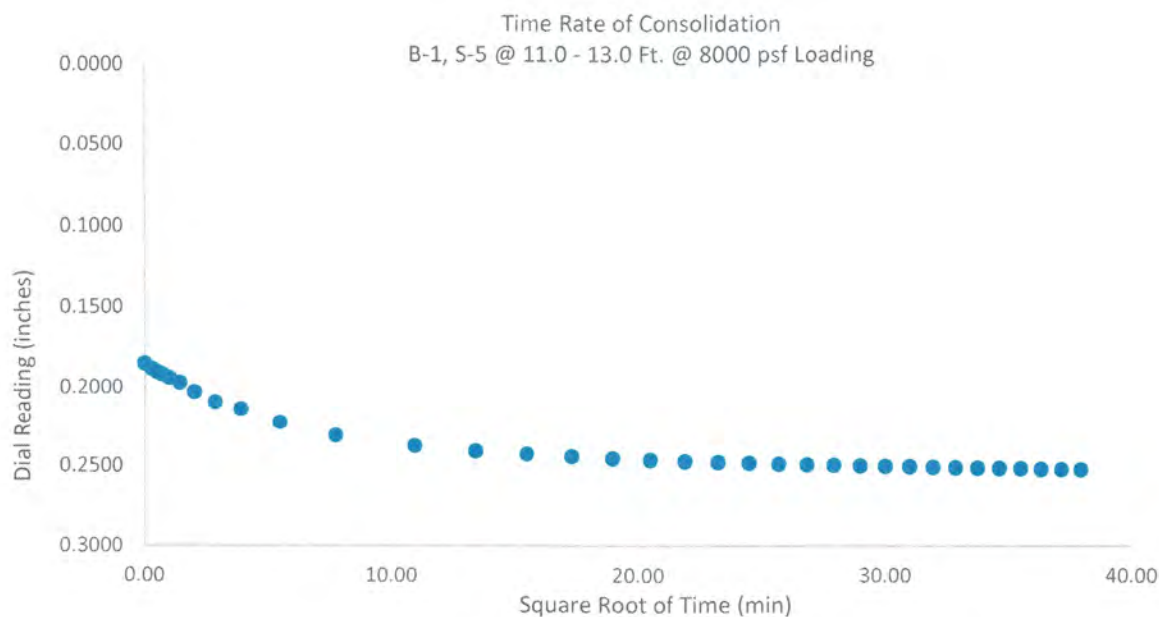
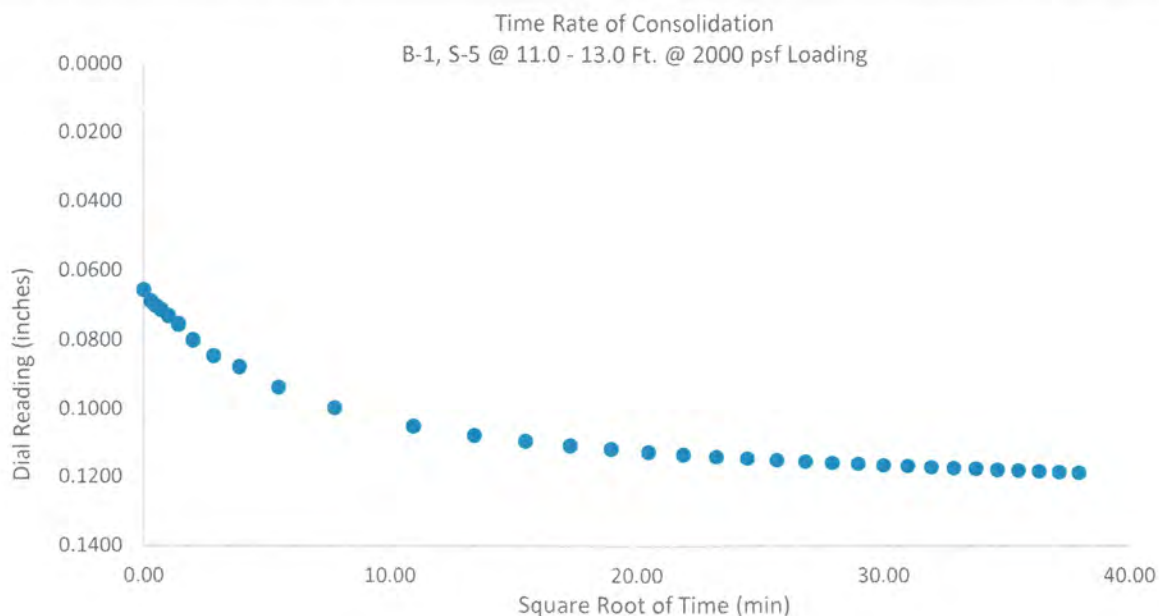
Report To: Tim Blackwood, PE, GE, CEG
Pali Consulting, Inc.
1120 SW Fifth Avenue, Suite 1302
Portland, Oregon 97204

Date: 6/16/2023

Lab No.: 23-095

Project: Noble Creek

Project No.: 00-223425-1



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SHEET 3 of 3

REVIEWED BY: Michael A. Ginsbach

TECHNICAL REPORT

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TECHNICAL REPORT

Report To: Tim Blackwood, PE, GE, CEG
Pali Consulting, Inc.
1120 SW Fifth Avenue, Suite 1302
Portland, Oregon 97204

Date: 7/10/2023

Lab No.: 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.

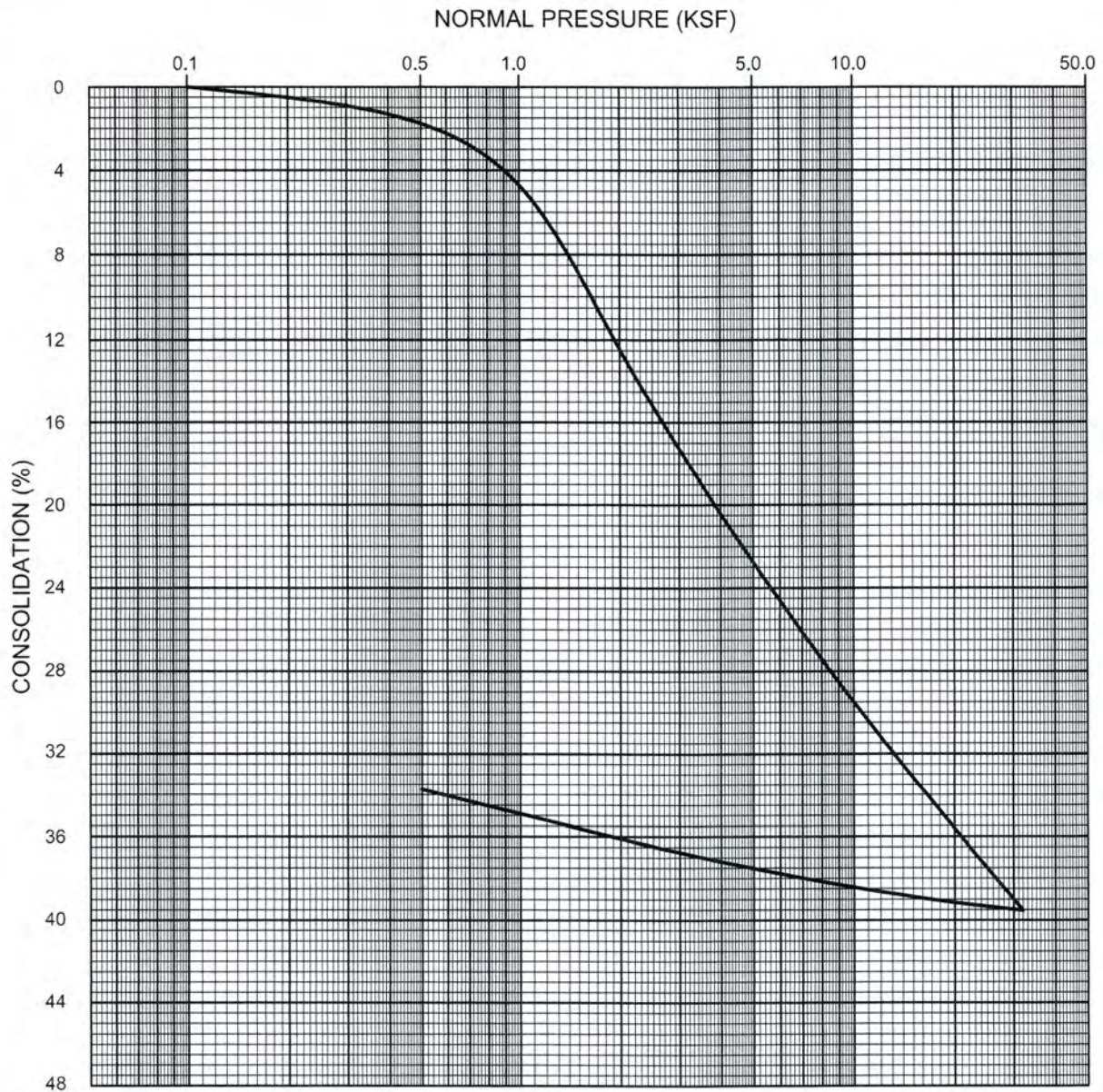
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SHEET 1 of 3

REVIEWED BY: Michael A. Ginsbach 

TECHNICAL REPORT

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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.
LABORATORY TESTING

LAB NO. 23-129

TECHNICAL REPORT

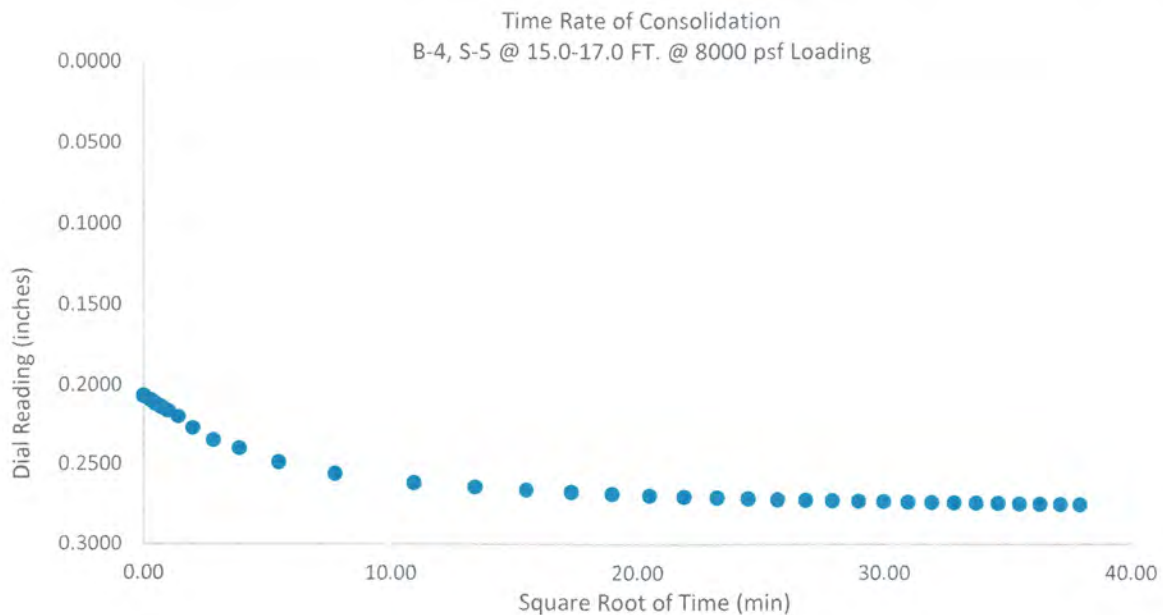
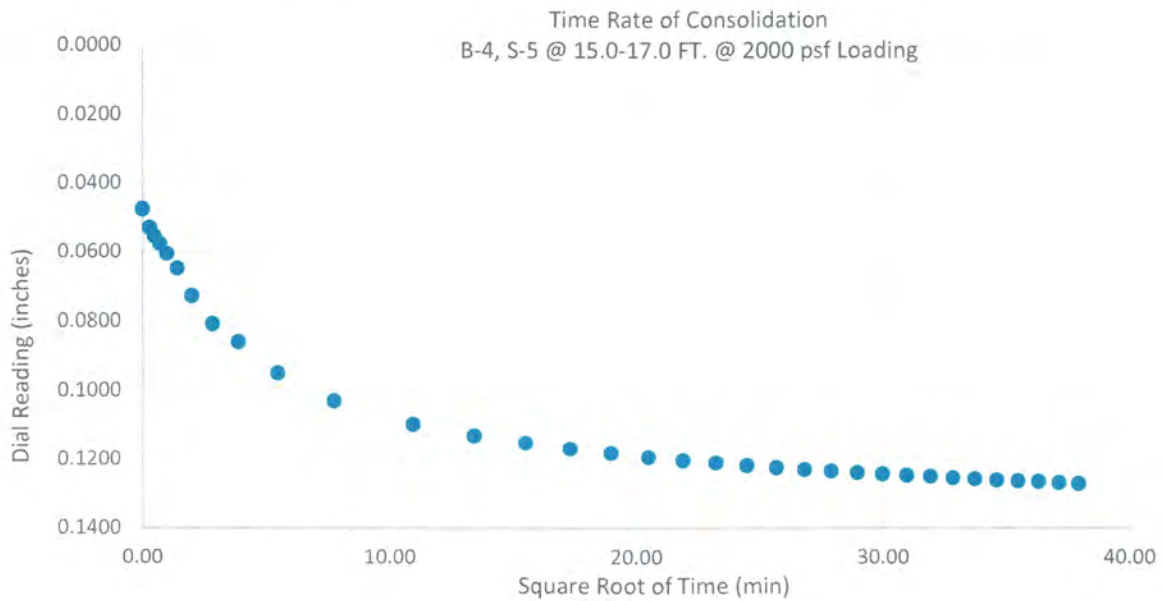
Report To: Tim Blackwood, PE, GE, CEG
Pali Consulting, Inc.
1120 SW Fifth Avenue, Suite 1302
Portland, Oregon 97204

Date: 7/10/2023

Lab No.: 23-129

Project: Noble Creek

Project No.: 00-223425-1



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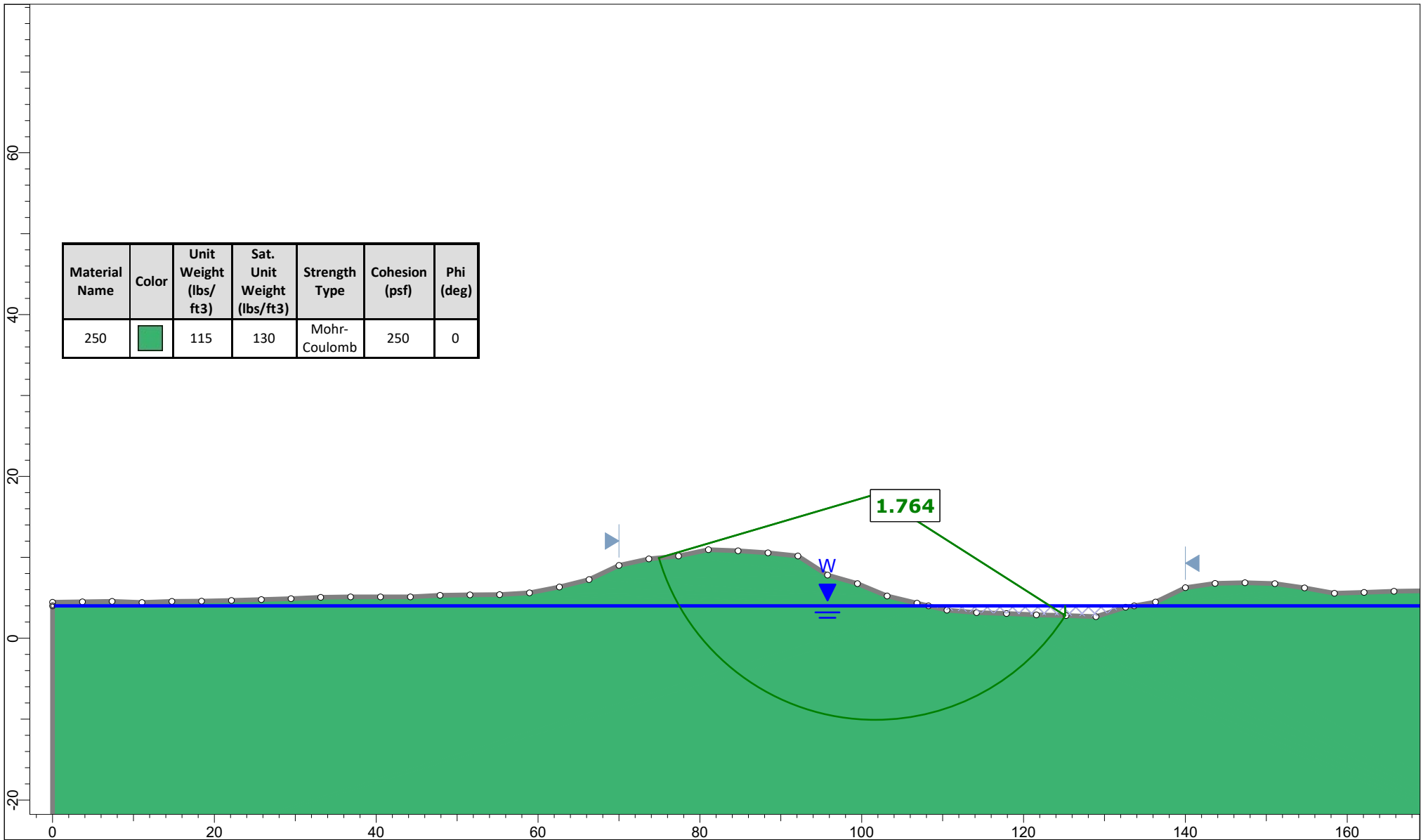
SHEET 3 of 3

REVIEWED BY: Michael A. Ginsbach

TECHNICAL REPORT

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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

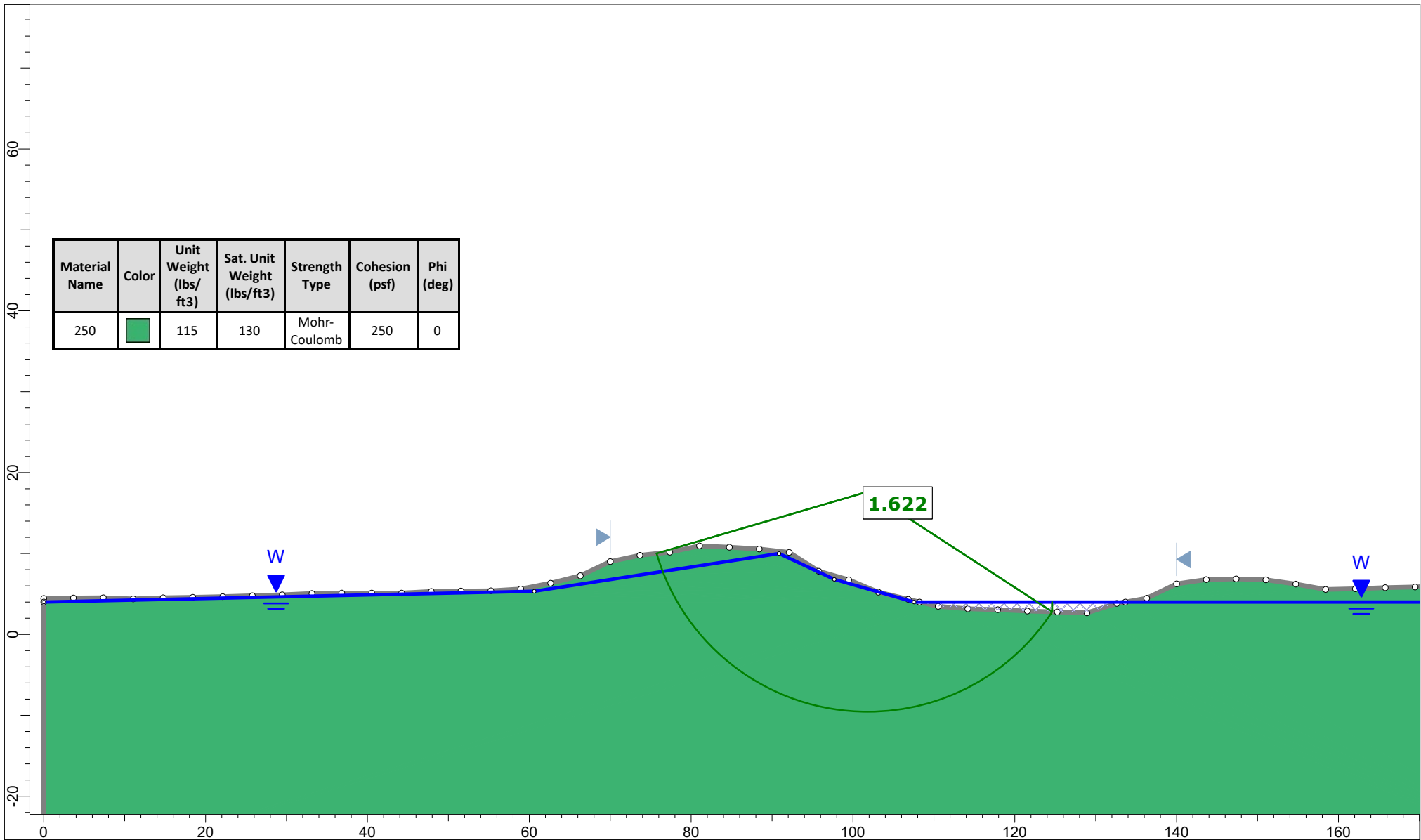
8/20/2023



Pali Consulting

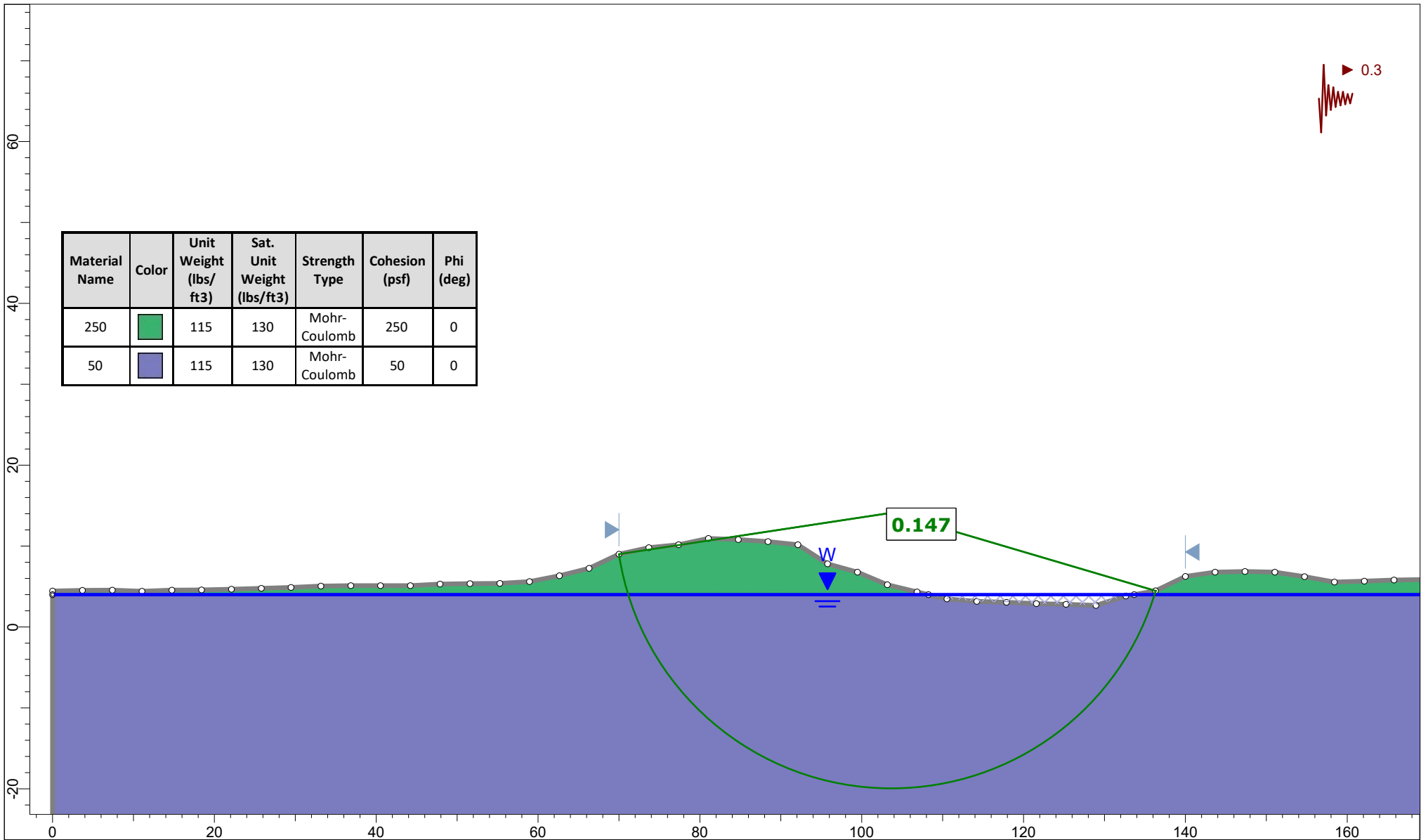
Figure

C-1C-2



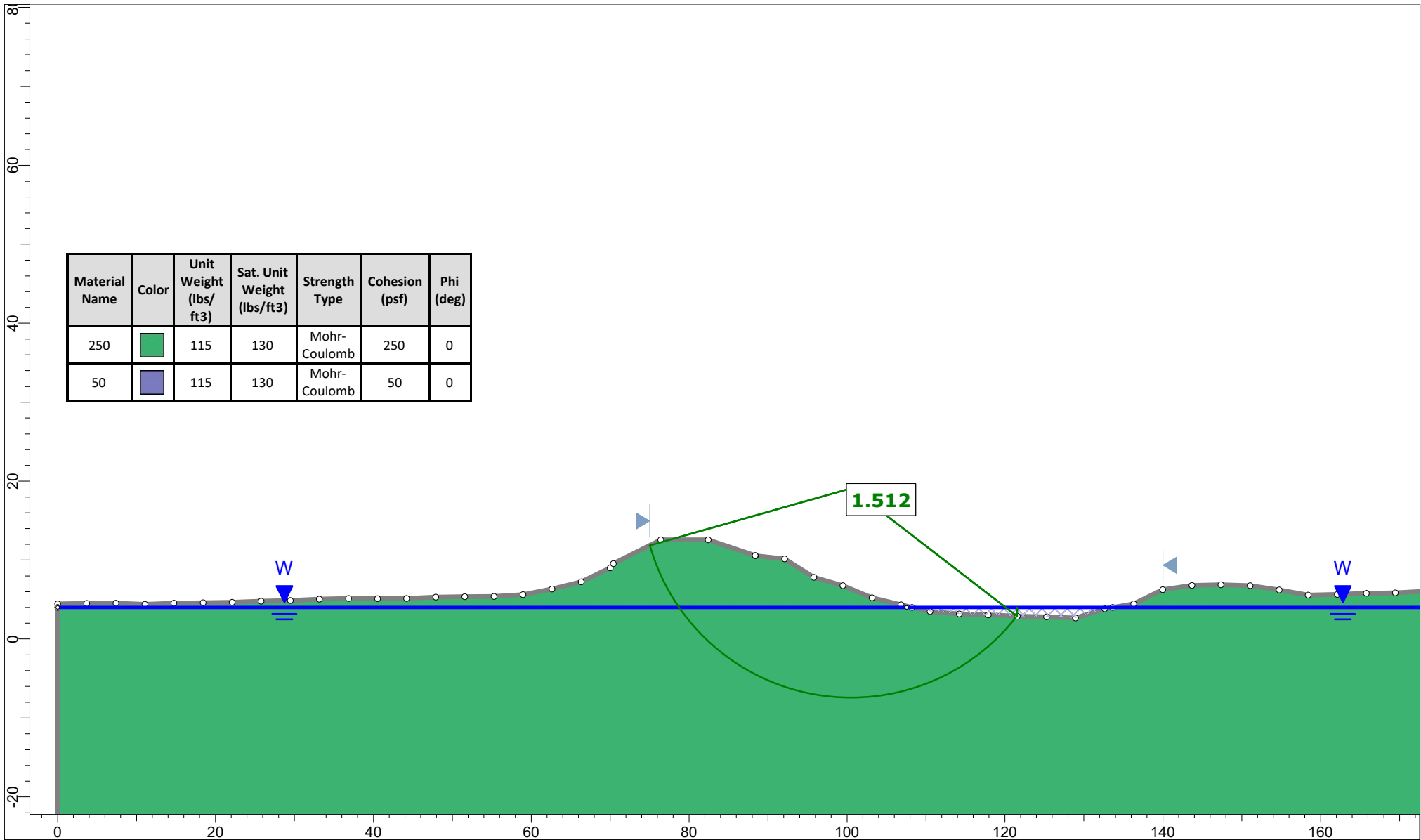
Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
250		115	130	Mohr-Coulomb	250	0

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

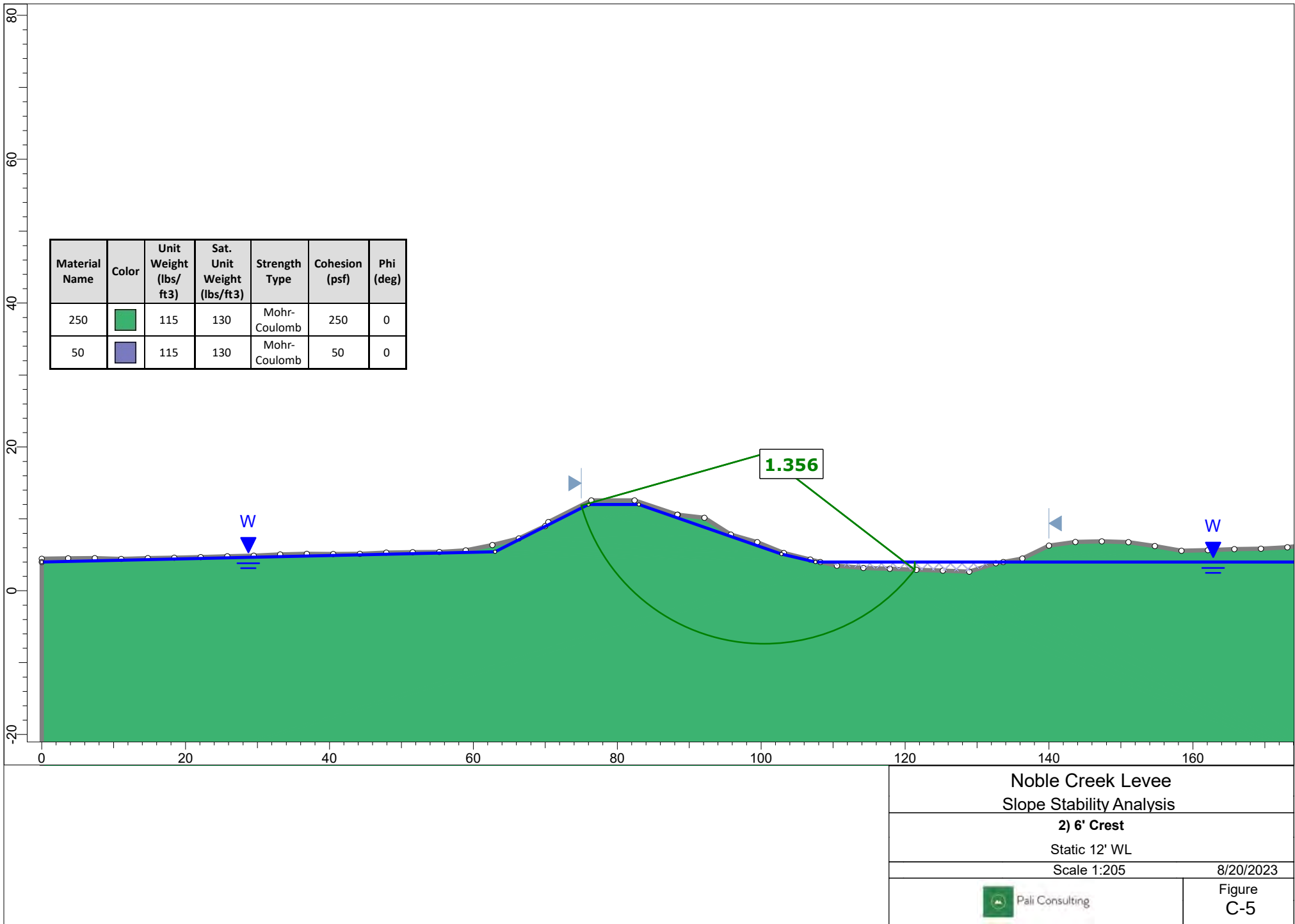
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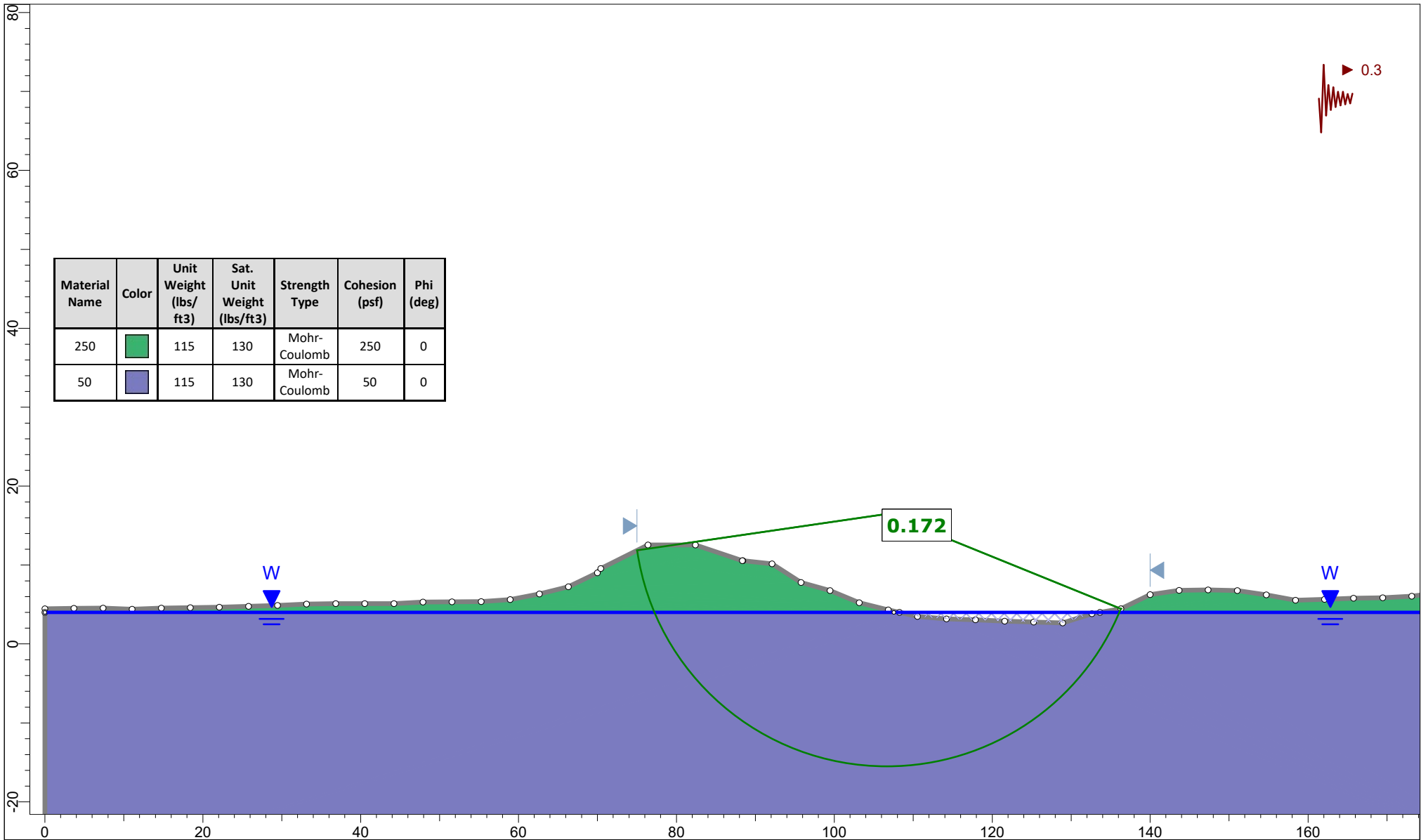
8/20/2023



Pali Consulting

Figure
C-4





Noble Creek Levee Slope Stability Analysis

2) 6' Crest

Seismic

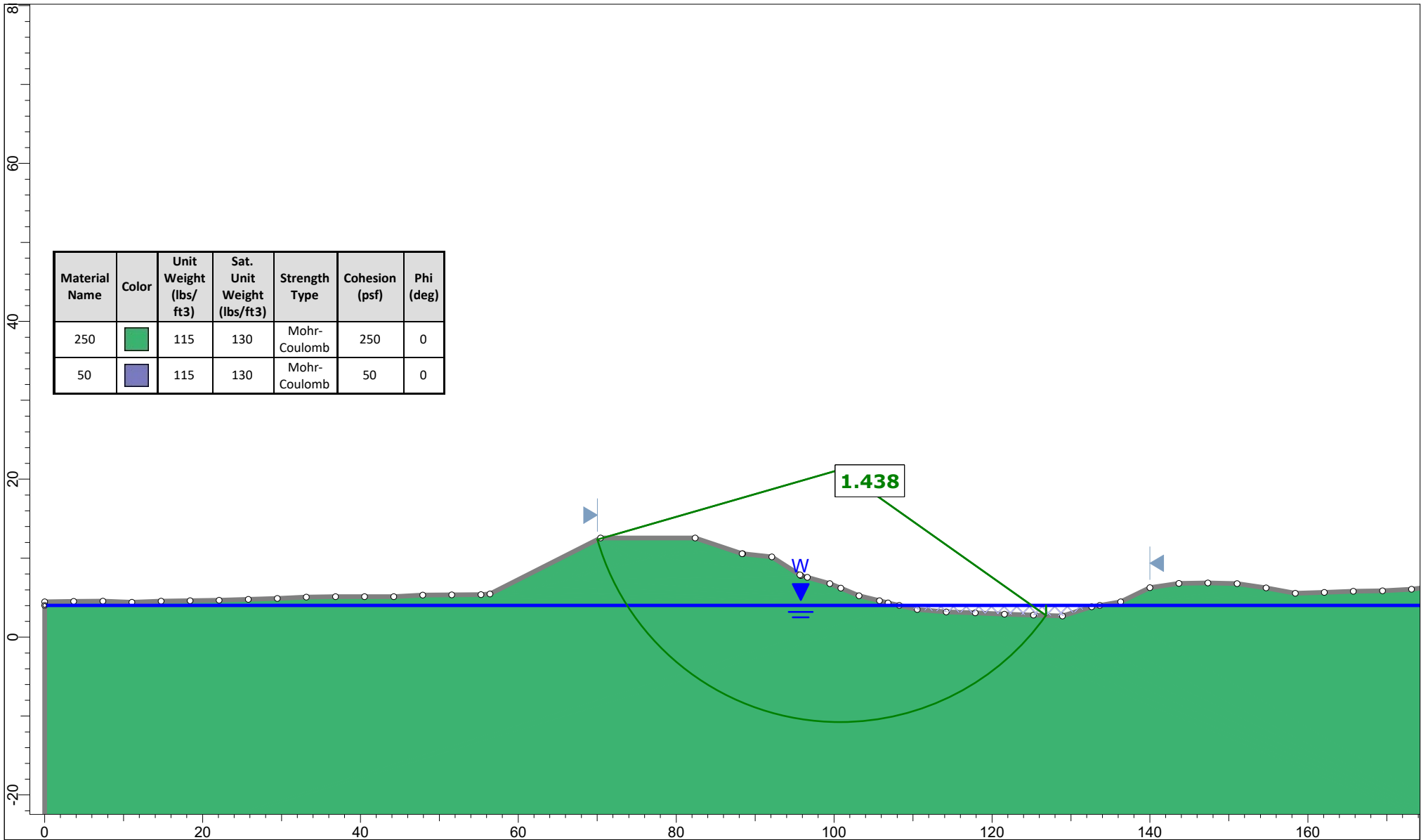
Scale 1:205

8/20/2023



Pali Consulting

Figure
C-6



**Noble Creek Levee
Slope Stability Analysis**

3) 12' Crest

Static

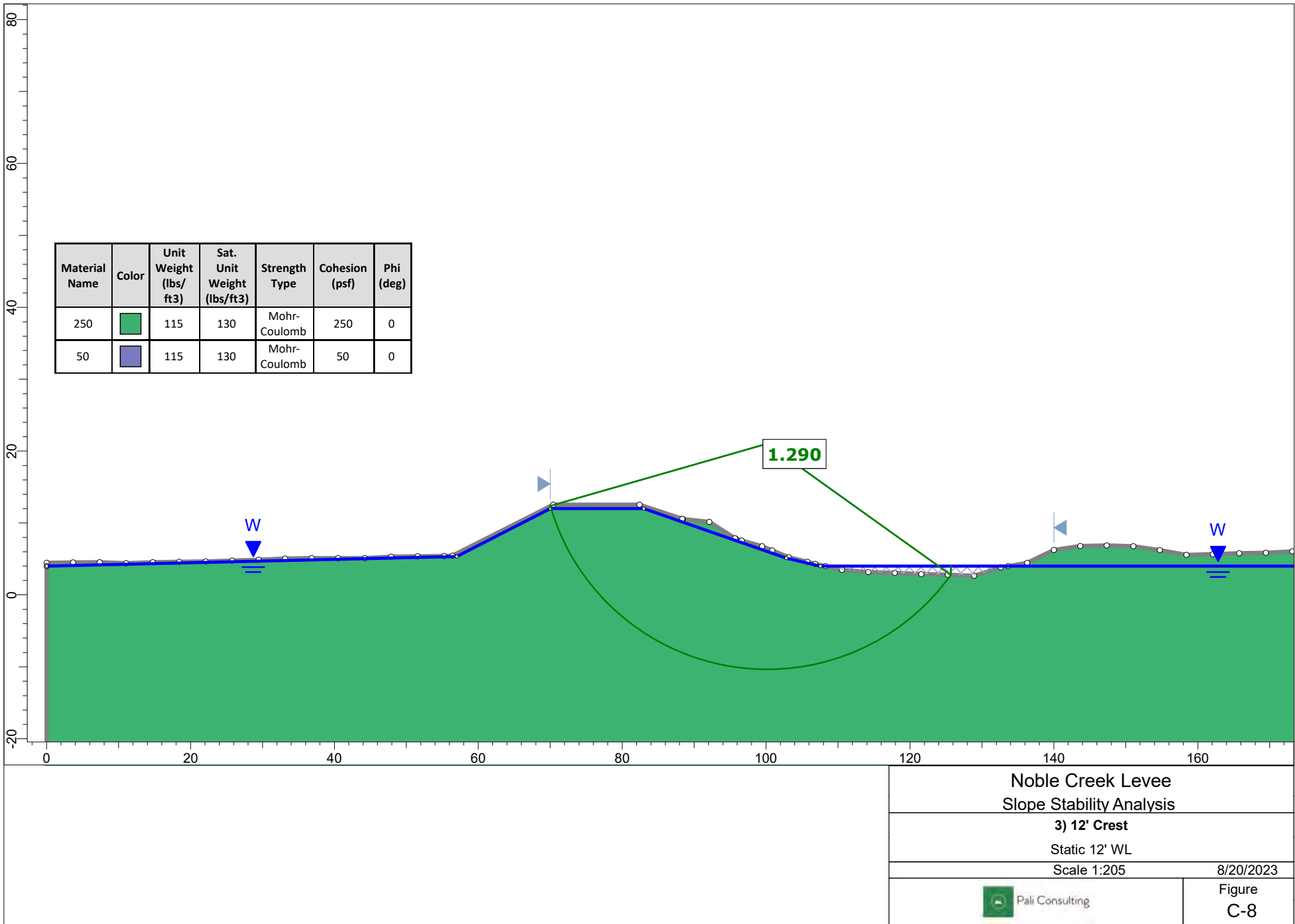
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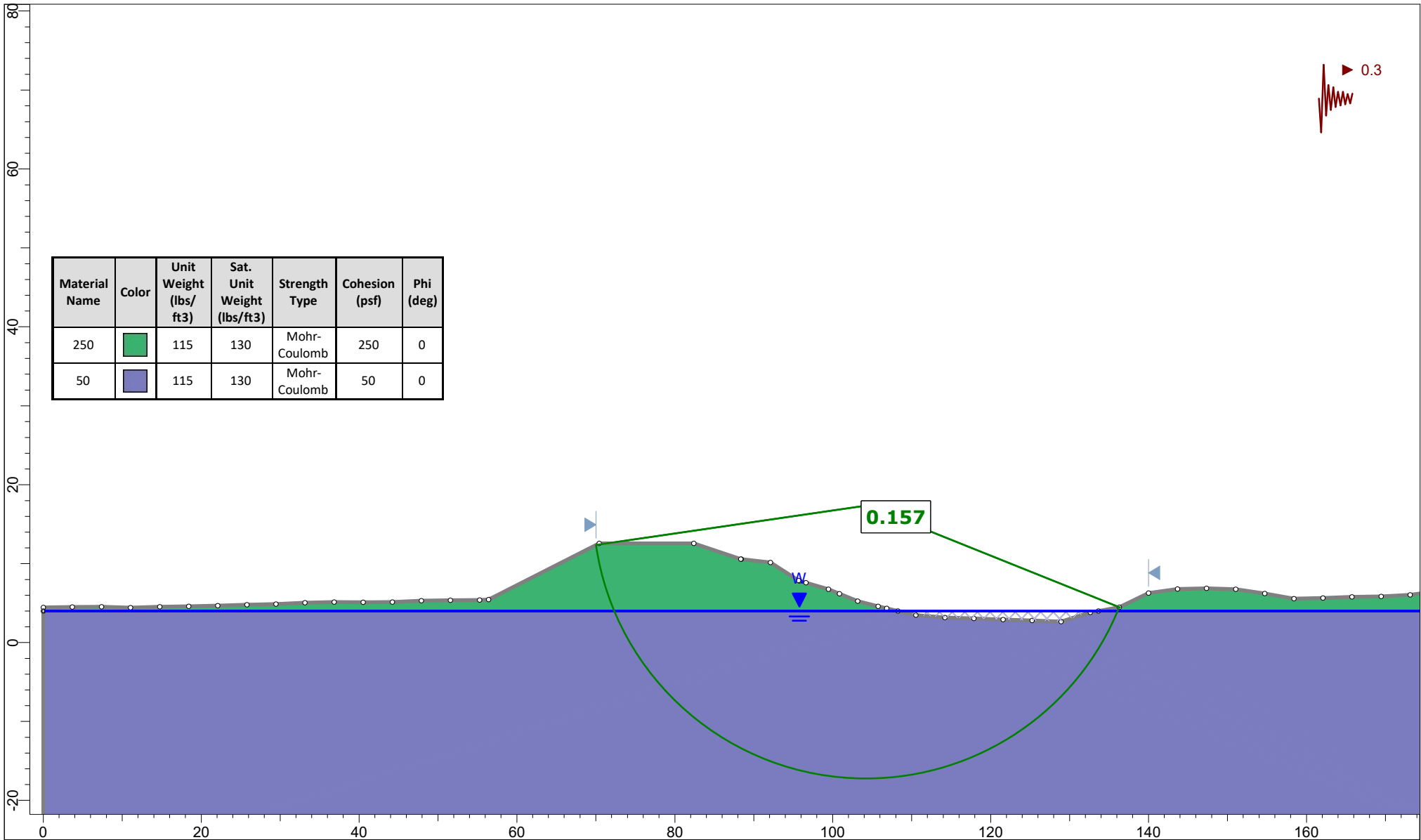
8/20/2023



Pali Consulting

Figure
C-7





**Noble Creek Levee
Slope Stability Analysis**

3) 12' Crest

Seismic

Scale 1:205

8/20/2023



Pali Consulting

Figure
C-9



Pali Consulting

July 13, 2026

Waterways Consultants
Attn: Jake D. Hofeld
1020 Taylor Street, Suite 380
Portland, Oregon 97205

Geotechnical Evaluation

Green Acres Lane Relocation Project
Coos County, Oregon
Project No. 014-25-013

1.0 INTRODUCTION

Pali Consulting, Inc. (Pali Consulting) presents this geotechnical evaluation for the Green Acres Lane Relocation Project (Project) located in Coos County, Oregon, as shown on Figure 1. Our work has been completed in general conformance with our Geotechnical Services proposal dated September 26, 2025 and our February 4, 2026 Professional Services Agreement with Waterways consulting, Inc. The project owner is the Coos County Soil and Water Conservation District (SWCD). Pali Consulting completed a previous evaluation for a related culvert replacement project at the west end of the Project, titled Noble Creek Tidelands Project, and dated August 21, 2023 (Noble Creek Report). That evaluation included deep explorations and testing, the results of which have been used and are referenced in this report.

The project includes relocating a portion of Green Acres Lane that runs parallel to Noble Creek (roughly 350 feet east of the intersection with Upper Loop Lane), away from the unstable stream bank along Noble Creek, as shown on Figure 2. The new alignment runs through adjacent pastureland, approximately 20 feet north of the existing roadway. The road will be supported on an embankment requiring up to 5 feet of fill, which is expected to result in surface settlement due to underlying soft clay.

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

- Reviewing site information available from our previous Noble Creek Report and other online sources.
- Completing subsurface explorations supplementing previous Noble Creek Report explorations. This included three shallow hand auger borings, maintaining logs of the soil and groundwater conditions encountered, and collecting soil samples for laboratory testing.
- Performing laboratory tests on select samples to evaluate engineering characteristics.
- Completing engineering analyses and an overall evaluation of the suitability of the site to support the project.



- Preparing 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

We reviewed geologic and soil mapping of the site, water well logs in the general area, and historic aerial photos of the site in developing our Noble Creek Report. A summary of our background review, including our previous site geotechnical hazard review, follows.

2.1 GEOLOGY

The geology of the site is mapped by the Oregon Department of Geology and Mineral Industries' (DOGAMI) on their Interactive Geologic Map of Oregon website (accessed April 2026). 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.

2.2 SOILS

Soils within the area are mapped by the Natural Resources Conservation Service's (USDA) on their Web Soil Survey (WSS) website (accessed April 2026). 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 0.20 in/hr).

2.3 WATER WELL LOGS

We reviewed water well logs from the surrounding area to better understand subsurface conditions. Well logs generally report clay to about 40 feet below ground surface (bgs) which was typically underlain by sandstones and siltstones. Static water levels varied widely, from 2 feet bgs to about 69 feet bgs about a quarter mile south of the nearby Bramble Lane Bridge. The closest well to the site, 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 bgs and siltstone from 43 to 80 feet bgs, the depth of the well.

2.4 GEOLOGIC HAZARDS

Geologic hazards identified in the Noble Creek Report were based on information provided on the Oregon Department of Geology and Mineral Industries' (DOGAMI's) Statewide Geohazards Viewer (HAZVU) website, which we reviewed again for this report. Geologic hazards mapped at or near the site include landslides, earthquake shaking, liquefaction, flooding, and tsunami inundation. The mapped landslide hazard is low to moderate at the west margins of the site (near the Noble Creek road crossing) and the banks of Noble Creek, associated with localized steep riverbank and levee slopes. Very strong to violent earthquake shaking from Cascadia and local earthquakes and low to moderate 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 and liquefaction potential are explained in further detail in *Section 4.1*.

3.0 SITE CONDITIONS

We evaluated site conditions by conducting a geologic reconnaissance and completing shallow hand subsurface explorations. The shallow explorations were completed to supplement our previous deep explorations which are also relied on this report. Subsurface explorations and laboratory testing from our shallow explorations are described in Appendix A. The following sections describe our findings.

3.1 SURFACE CONDITIONS

The project site is located within a floodplain northeast of Noble Creek, southeast of Upper Loop Lane. The alignment is bounded to the east by a drainage feature that ties into the Isthmus Slough. The project area consists primarily of open, vegetated ground with dense low-growth vegetation (brush) and pasture grasses, with the southwest boundary marked by a steepened bank leading down toward the adjacent marshy terrain, and some native deciduous trees, reeds, and shrubs along Noble Creek. While the existing Green Acres Lane road embankment sits at approximately elevation 10 feet above mean sea level (MSL), the realignment partially traverses a lower, floodplain area at about 5 to 6 feet MSL. Evidence of seasonal inundation is suggested by a curved drainage channel that wraps around the eastern perimeter of the site.

3.2 SUBSURFACE CONDITIONS

We completed shallow subsurface explorations at the site on February 11, 2026, using a hand auger and drive probe soundings (DPS) at the locations shown on Figure 2. The hand auger explorations extended up to 5 feet bgs, which was the maximum exploration depth due to caving under high groundwater conditions. The DPS were advanced up to 10 feet bgs. We also relied on the previous explorations described in our Noble Creek Report. A description of our shallow subsurface explorations, logs of our augers, and the results of our laboratory testing on select soil samples are included in Appendix A.

3.2.1 Near-Surface Soil Conditions

Our recent hand auger explorations along the new road alignment (labeled HA-1A through HA-3A on Figure 2) encountered about 3 inches of sod/topsoil over 3 to 6 inches of clayey sand/sandy silt fill, underlain by soft grading to very soft, moist to wet, brown-grey, elastic silt with organics, which extended to the maximum depth explored of 4 to 5 feet bgs. The elastic silt is high plasticity and generally contains varying amounts of organic material. Atterberg limits testing indicates a plasticity index (PI) of 74 to 92 and a liquid limit (LL) of 137 to 162, leading to a MH classification per ASTM. Moisture content in the elastic silt varies from 51 to 172 percent, which may reflect a varying/high amount of organic material in the soil.

3.2.2 Deep Soil Conditions

The previous subsurface explorations from our Noble Creek Report (labeled B-1, B-2, B-5, and HA-2 through HA-4 on Figure 2), indicate that the high-plasticity, elastic silt extends to a depth of 40 to 65 feet bgs. In some locations and at various depths, the deep elastic silt deposit grades to a fat clay with similar high plasticity. Atterberg limits testing on these previous exploration soil samples indicate a plasticity index (PI) of 26 to 98 and a liquid limit (LL) of 57 to 140, resulting in an ASTM classification ranging from MH to CH. Wood fragments, charcoal, and other organic materials were present in many of the samples.

SPT N-values within this deep deposit of soft to very soft, elastic silt/fat clay is on average 1 blow per foot (bpf). Moisture content ranges from 42 to 115 percent.

3.2.3 Groundwater

Depth to groundwater was observed between the ground surface and 1-foot bgs in all three recent hand augers along the new road alignment. 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

We performed various engineering analyses to evaluate the feasibility of the proposed road realignment and develop appropriate geotechnical recommendations for the design and construction of the new roadway embankment. The following sections describe our analysis procedures and results.

4.1 SEISMIC ANALYSIS

Our seismic site analysis was limited to establishing the seismic site class and design peak ground acceleration (PGA) required for slope/global stability evaluation of the proposed road embankment and evaluating the potential for soil liquefaction.

4.1.2 Seismic Site Class and Peak Ground Acceleration

We evaluated the seismicity at the site for a design earthquake hazard level corresponding to a 5 percent probability of exceedance in 50 years (975-year return period), which is common practice for roadway embankment design. Based on the USGS Earthquake Hazard Toolbox website (<https://earthquake.usgs.gov/hazards/interactive/>, USGS 2026), the bedrock (Site Class B) PGA for this design earthquake is 0.55g (mean magnitude of 8.63). Adjusted for Site Class E amplification due to the deep, soft, clay subsurface conditions, the design PGA for this project is 0.83g.

4.1.3 Potential Soil Liquefaction

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.

Because of the high PI value (greater than 18) of the high-plasticity elastic silt/fat clay deposits at this site, liquefaction or strain softening is considered very unlikely.

4.2 EMBANKMENT SETTLEMENT

We evaluated static settlement for the proposed new roadway embankment, based on the vertical and horizontal dimensions shown on Figure 2. For a 32-ft wide embankment roughly 5-ft high, we estimate that

the embankment may settle on the order of 1 foot, based on the laboratory consolidation test results from our previous Noble Creek Report and our experience with similar sites/soils.

Because of the silty and clayey nature of the soils underlying the roadway, settlements are expected to take about 3 to 12 months to reach primary consolidation, although this may take longer depending on varying soil conditions across the site. Depending on the organic content of the soils, long-term secondary consolidation could be on the order of 6 inches (half of primary consolidation as noted in Noble Creek report) and may continue for 10 to 20 years.

4.3 EMBANKMENT GLOBAL STABILITY

To analyze the global stability of the proposed roadway embankment, we performed a slope stability evaluation using the numerical computer program Slide 2 (Rocscience, version 3.034). The analysis was based on typical soil strength correlation values derived from the laboratory testing on representative soil samples from the site explorations included in this report. Our slope stability analysis methods and results are described in further detail in Appendix B.

Based on this analysis, the static factor of safety (FS) of the proposed embankment is 1.8, which is greater than the minimum typically accepted for roadways not supporting structures (1.3). However, the seismic FS is estimated to be 0.5, which is significantly lower than the minimum generally accepted (1.1) and indicates that soils underneath the embankment may fail, resulting in the embankment moving laterally toward Noble Creek in a design earthquake event. This is primarily due to the very low strength of the subsurface soils. Due to the significant depth of these weak soils, potential ground improvement or other mitigation measures will likely be cost prohibitive. We understand that the project owner is aware of and accepts this potential embankment instability in the seismic design case, so seismic instability is not further addressed in this report.

The slope stability analysis indicates that the construction of the new roadway embankment will not adversely impact the stability of the nearby creek bank/channel side slope.

5.0 CONCLUSIONS

Based on our explorations, testing, and analyses, it is our opinion that the site is suitable to support the proposed roadway realignment and new embankment construction, provided the recommendations in this report are included in design and construction. We generally conclude the following regarding the site and proposed project:

- Excavation and site grading can be performed with conventional earthwork equipment.
- The fine-grained site soils are highly moisture-sensitive and easily disturbed (e.g., rutted, pumped, etc.) by construction activities, especially during the wet season or when the moisture-content is above optimum. Therefore, earthworks should be scheduled during the dry season and care must be taken to reduce disturbance during construction. Rock haul roads and even crane mats may be necessary for construction in some cases.
- On-site soils are not suitable for use as structural fill, due to their moisture-sensitive nature. Instead, we recommend use of imported granular material for embankment construction.
- The placement of 5-feet of structural fill along the new roadway embankment is expected to cause on the order of 1 foot of settlement. Therefore, we recommend that this fill (6-foot high to include anticipated settlement) is placed as a “preload” and the settlement surveyed to confirm that primary consolidation has been completed before starting final construction of road surfacing or installing settlement-sensitive utilities. See Section 6.8 for more details.

- o Following primary consolidation, the top elevation of the road embankment may vary and will likely require additional grading to achieve final design grade and accommodate road surfacing materials.
- o If the long-term elevation of the road surface is critical, additional fill could be placed to “surcharge” the embankment alignment to reduce long term settlement from secondary consolidation.

The following sections present our specific earthwork recommendations for the proposed roadway embankment construction.

6.0 EARTHWORK RECOMMENDATIONS

6.1 GENERAL

We have assumed that site grading will be limited to minor cuts and to fills of about 5 feet for embankment construction and surcharging. Based on our site explorations and evaluations described in this report, our recommendations for earthworks components of the project are provided in the following sections. If cuts or fills will exceed these assumptions, we should be contacted to re-evaluate our recommendations.

6.2 SITE PREPARATION

Initial site preparation is expected to be minimal, as the site is largely undeveloped. However, if present, existing facilities should be demolished, followed by clearing of any vegetation, stripping surface soils, and excavating to grade in areas of improvements. Demolition should include removal of all existing structures and improvements to the full extent they occur. If piping is present, it should be removed, or grouted full if abandoned in place. Areas below final grade resulting from demolition, clearing or stripping, should be backfilled with structural fill as described later in this report.

Vegetation should be removed, including grubbing the root balls of trees and shrubs and significant roots over about ½-inch in diameter. Grubbed material should be hauled off site and disposed of properly. Stripping should remove grasses and any organic-rich soil.

Sod, topsoil, and organic-rich soil was found to extend to depths of about 3 inches deep bgs. The actual stripping depth should be determined in the field based on site conditions, but we anticipate that 4- to 8-inches should be anticipated. Stripped material should be transported off site for disposal or can be stockpiled for use as organic surface fill in non-structural areas. If the latter, Pali Consulting should review stockpile locations and configurations to be sure they don’t cause unacceptable settlement in the area of the stockpiles.

After stripping and grubbing are completed, areas should be rough graded to reach subgrade elevation. Where raised grades are required or where site preparation activities cause excessive subgrade disturbance, structural fill should be used to establish a firm and unyielding subgrade. Disturbance to the subgrade should be expected if site preparation and earthwork are conducted during periods of wet weather and/or when the moisture content of the surficial soil exceeds optimum. If this occurs, the wet-weather measures noted later in this report should be implemented. Due to high groundwater conditions expected during most of the year, a rock haul road may be needed to access the alignment during most of the year, as described in Section 6.4 of this report.

If uncontrolled fill is encountered within the embankment subgrade or other structural areas, Pali Consulting should be notified so we can evaluate whether the fill should be removed and replaced with structural fill as noted elsewhere in this report.



6.3 SUBGRADE PREPARATION AND EVALUATION

Following stripping and excavation to grade, the subgrade should be evaluated for suitability to support roadway embankment. Due to the soft native soils and high groundwater table, we recommend subgrade evaluation be conducted by Pali Consulting by probing and observing site stripping, as proof-rolling with a fully loaded dump truck (which is typical practice) will likely damage the subgrade during most of the year. If any unsuitable subgrade areas are identified during probing, these areas should be recompacted or over-excavated to the extent indicated by the Pali Consulting representative. Overexcavated areas should be replaced with structural fill per *Sections 6.6 and 6.7* of this report.

6.4 WET SOIL/WET WEATHER CONSTRUCTION

Because of the soft fine-grained soils and high groundwater, we recommend the embankment subgrade be covered with a thickened crushed rock working pad underlain/supported by a woven geotextile fabric, before the embankment fill is placed. The rock pad should consist of Stabilization Rock as specified in Section 6.6.1.4, and the woven geotextile fabric should be as specified in Section 6.6.2. The Stabilization Rock should be placed in a single 18-inch thick lift, pushing the rock ahead of equipment as it is constructed to protect the subgrade and fabric until the rock is in place. Once in place, the Stabilization Rock should be compacted with sheepsfoot or similar heavy equipment to a well-keyed condition (without vibration),

Outside of the embankment footprint, soils within the anticipated excavation depths are expected to consist of similar elastic silt, that is also highly sensitive to changes in moisture conditions and therefore easily disturbed during construction. Significant disturbance to these soils should be expected if site preparation and earthwork activities are conducted during periods of wet weather and/or when the moisture content of the surficial soils exceeds optimum. Earthwork planning should include considerations for minimizing disturbance to the subgrade. Earthwork conducted during the wet season/in wet weather conditions will likely result in significant additional effort and resulting costs/potential schedule impacts compared to dry season/weather conditions.

Care should be taken by the contractor to reduce traffic across wet subgrade areas. The contractor should be prepared to use wet-weather construction methods to reduce subgrade disturbance, as necessary. These may include additional thickened crushed rock working pads underlain/supported by geotextile fabric, using only tracked equipment, or special staging and access sequencing. It should be the contractor's responsibility to select and implement measures to limit disturbance during wet weather, and to repair subgrade damage due to wet weather operations at no cost to the owner.

6.5 EXCAVATIONS

Although significant excavation is not anticipated for roadway embankment construction, we are providing general excavation recommendations for localized areas of excavation, if needed.

6.5.1 General

As noted above, the soil encountered within anticipated excavation depths will generally be native silt. It is our opinion that conventional earthmoving equipment in proper working condition should be capable of making the necessary general earthwork excavations to the expected excavation depths. The earthwork contractor should be responsible for providing the necessary equipment and following procedures required to excavate the site soils, as described in this report and on the logs in Appendix A.

6.5.2 Trench Excavations

Because of the high groundwater and soft elastic silt subsurface conditions at this site, vertical trench excavations may not stand vertically to 4 feet deep bgs, as is commonly assumed. Trenches that are unstable or greater than 4 feet deep will require the excavation sidewalls to be sloped back or shored. Shoring is



preferable as the soft soils will be prone to collapse, even when sloped back. All trench excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. For excavation sloping and planning purposes, we recommend the site soils within anticipated excavation depths be considered OSHA Type C Soil. Because of the anticipated groundwater seepage, temporary excavation slopes may need to be flatter than the standard OSHA slope inclinations based on soil type. If shoring is required or used by the Contractor, it should be the responsibility of the Contractor to select the appropriate shoring method, meet all OSHA requirements, protect workers, and safeguard any infrastructure near trench locations.

Because of the potential for variations in soil type and properties between exploration locations, actual OSHA soil types and stable slope angles should be determined in the field during construction. The stability of temporary excavation slopes should be the responsibility of the Contractor who is in control of the construction operation and is continuously at the site to observe the nature and condition of the subsurface.

6.5.3 Temporary Dewatering

Near-surface groundwater conditions should be anticipated, based on our explorations. Therefore, perched or static groundwater seepage should be anticipated at relatively shallow depths, even if construction occurs during the dry season. In this case, the contractor should be prepared to provide dewatering systems that can adapt to varied soil and groundwater conditions and as described in this report. In general, we expect that sump pumps in the excavations will provide sufficient dewatering of the fine-grained site soils. However, increased groundwater flow within sandy soil zones may be encountered locally, which may require additional dewatering measures.

The contractor should be responsible to select and design any dewatering systems required to dewater work areas as necessary.

6.6 STRUCTURAL FILL AND BACKFILL

Although structural fill is primarily expected to be required for the embankment construction, we are also providing general fill recommendations in the event this is needed for other portions of project construction.

6.6.1 General

Structural fill includes all fill placed beneath the proposed roadways, sidewalks, and other slabs; in other areas intended to support structures; within the influence zone of any structures; and fill placed on slopes greater than 5H:1V.

Structural fill should only be placed over a subgrade that has been prepared in conformance with the prior sections of this report. A variety of soil may be used as structural fill. All material used as structural fill should be free of debris, clay balls, roots, organic matter, frozen soil, man-made contaminants, particles greater than 4 inches in size, and other deleterious materials.

6.6.1.1 On-Site Soil

The on-site, near-surface highly plastic, elastic silt/fat clay is not considered suitable for use as structural fill, due to their moisture sensitivity and high natural moisture content. Instead, we recommend using imported granular fill.

6.6.1.2 Imported Select Structural Fill

Imported select granular material may be used as a structural fill. The imported material should consist of pit or quarry run rock, crushed rock or crushed gravel and sand that is well-graded between coarse and fine



sizes and should meet the specifications provided in the Oregon Department of Transportation Standard Specifications (OSS) 00330.14 – Selected Granular Backfill or OSS 00330.15 – Selected Stone Backfill. It should meet the general structural fill recommendations provided above and should contain less than 5 percent passing the U.S. No. 200 Sieve. During dry weather, the fines content can be increased to a maximum of 12 percent.

The material should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities as recommended in *Section 6.7*.

6.6.1.3 Aggregate Base

Imported granular material used as surfacing for aggregate roads, or aggregate base (base rock) beneath pavements, sidewalks, or slabs, should be clean, crushed rock or crushed gravel and sand that is well-graded between coarse and fine particles. The base aggregate should meet the specifications provided in the (OSS) 00641 – Aggregate Subbase, Base, and Shoulder Base Aggregate, depending upon application, with the exception that the aggregate have less than 5 percent by dry weight passing a U.S. Standard No. 200 Sieve and have at least two mechanically fractured faces. The aggregate should have a maximum particle size ranging from $\frac{3}{4}$ to $1\frac{1}{2}$ inch, depending on application.

The aggregate should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities as recommended in *Section 6.7*.

6.6.1.4 Subgrade Stabilization Rock

The subgrade stabilization rock should consist of imported pit or quarry run rock, crushed rock or crushed gravel and sand that is well-graded between coarse and fine sizes, meeting the requirements of OSS 00330.14 – Selected Granular Backfill or OSS 00330.15 – Selected Stone Backfill. The rock should be angular, containing a minimum 2 fractured surfaces, and should contain less than 5 percent passing the U.S. No. 200 Sieve. The material should be placed and compacted as recommended in *Section 6.4*.

6.6.1.5 Drain Rock

Drain rock should consist of clean, angular, crushed rock or gravel with less than 3 percent passing the U.S. No. 200 Sieve and that meets the gradations provided in OSS 00430.11 – Granular Drain Backfill Material. The drain rock should be wrapped in a Type 1 drainage geotextile that meets the specifications provided in OSS Table 02320-1 – Geotextile Property Values. The geotextile should be installed in conformance with OSS 00350 – Geosynthetic Installation.

6.6.2 Geotextiles

Where imported granular material is placed over native soil subgrades during wet weather or as specified elsewhere in this report, we recommend a woven separation geotextile be placed between the subgrade and imported granular material. The subgrade geotextile should meet the specifications provided in the OSS 02320.20 – Geotextile Property Values for soil separation. The geotextile should be installed in conformance with the specifications provided in OSS 00330 – Earthwork and 00350 – Geosynthetic Installation.



6.7 FILL PLACEMENT AND COMPACTION

Structural fill should be placed and compacted in accordance with the following:

- Place fill and backfill on an approved subgrade prepared as recommended in *Sections 6.2 through 6.4*.
- Place fill or backfill in uniform horizontal lifts with a thickness appropriate for the material type and compaction equipment. Table 2 provides general guidance for lift thicknesses.

Table 2: Guidelines for Uncompacted Fill Lift Thickness

Compaction Equipment	Guidelines for Uncompacted Lift Thickness (inches)		
	Native Silt Fill	Granular and Crushed Rock Maximum Particle Size $\leq 1\frac{1}{2}$ inch	Crushed Rock Maximum Particle Size $> 1\frac{1}{2}$ inch
Plate Compactors and Jumping Jacks	4 – 8	4 – 8	Not Recommended
Rubber-tire Equipment	6 – 8	10 – 12	6 – 8
Light Roller	8 – 10	10 – 12	8 – 10
Heavy Roller	10 – 12	12 – 18	12 – 16
Hoe Pack/Sheepsfoot Equipment	12 – 16	18 – 24	12 – 16

Notes:

1. 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.

- Use appropriate operating procedures to attain uniform coverage of the area being compacted.
- Place fill at a moisture content within about 3 percent of optimum moisture content as determined in accordance with ASTM Test Method D 1557.
- Moisture condition fill to achieve uniform moisture content within the specified range before compacting. Compact fill to the percent of maximum dry densities as noted in Table 3.
- Do not place, spread, or compact fill soils during freezing or unfavorable weather conditions. Frozen or disturbed lifts should be removed or properly recompacted prior to placement of subsequent lifts of fill soil.
- During structural fill placement and compaction, a sufficient number of in-place field density tests should be completed to verify that the specified degree of compaction is being achieved.

**Table 3: 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
On-site Soil and Select Structural Fill	92	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.

6.8 PRELOAD/SURCHARGE PROGRAM

As noted in Section 4.2, the embankment fill placement will induce significant settlement that can damage overlying pavements and infrastructure. We therefore recommend that the embankment fill be placed as a preload and left to settle out before construction of final surface elements. Preload settlement should be monitored by a licensed surveyor after the fill has been placed and rough graded, and the results reviewed by Pali Consulting to evaluate when primary consolidation is complete and the risk of excessive post-construction settlement is minimal. This is expected to take several months to as much as a year. Faster settlements could be induced as described later in this section.

After a suitable length of time has passed and the underlying site soils have undergone primary consolidation under the design fill height preload, improvements can be constructed on the site and the influence of their additional weight will be lessened.

Because the preload settlement is anticipated to be significant, we recommend that one additional foot (equaling anticipated settlement) of structural fill be placed as a “surcharge” above the design fill height and then regraded/removed as necessary to meet final subgrade elevation for surface pavements. This will reduce the risk of post-construction settlement and avoid a second round of settlement monitoring for additional fill that may be required to final grade after the preload period. Greater fill heights/surcharges could be placed to accelerate the preload period and we can provide recommendations for this, if requested.

Subsequent references to surcharge fill includes the preload portion of the embankment fill.

6.8.1 Preload Placement

Preparation of the subgrade area and subsequent placement of the preload structural fill should follow the recommendations of this report. We also provide the following general recommendations for the preload/surcharge program at this site:

- The surcharge fill should be placed as structural fill in accordance with this report and the project plans and specifications.



- The surcharge fill prism should match the planned roadway embankment configuration along the entire roadway alignment.
- To minimize risk of potential subgrade failure, the 6-foot surcharge fill height should be placed in two layers (3-ft high each), spaced 2 weeks apart.
- The surcharge should be left in place for 3 to 12 months, based on settlement monitoring confirmation that adequate primary consolidation settlement has been achieved.
- Following confirmation of primary settlement, the surcharge fill can be removed and area regraded to final elevations per plan.

6.8.2 Adjacent Settlement due to Preload

The surcharge will cause gradually decreasing settlement a distance outside the delineated embankment fill area. While this will vary depending on the preload fill height and differing soil conditions, we generally anticipate potential settlements will taper out to nothing within about 50 feet of the preload edge. This may cause settlement of the adjacent ground surface, changes to drainage, and could potentially impact existing adjacent structures or buried utilities where they occur within 50 feet of the embankment edges. We therefore recommend that a comprehensive utility survey be conducted to identify potentially sensitive underground utilities near the planned preload areas to be filled, so potential adjacent ground settlement can be further evaluated in those areas, if needed. The owner of such utilities should be contacted to obtain their settlement tolerances and monitoring requirements. Changes to surface drainages from the settlement should also be considered.

6.8.3 Preload Monitoring and Removal

We recommend monitoring settlement of the fill with settlement plates placed at 75-foot intervals along the centerline of the roadway. The settlement plates should be installed following site preparation, but in advance of placing any preload fill. The settlement plates should be as depicted on the settlement plate detail provided in Figure 3.

Survey measurements should be completed by a licensed surveyor. An initial measurement should be made prior to placing any fill and then measurements should be obtained on a twice weekly basis for 4 weeks and once a week thereafter. The settlement data should be reviewed by our firm so that we can modify our preload recommendations, if needed.

As previously indicated, the total primary consolidation settlement is estimated to be achieved within approximately 3 to 12 months, although it could take longer. The final preload period should be determined based on the field settlement data. If a shorter settlement time frame is required, a surcharge can be added to the fill. Pali Consulting should be contacted to review if this is planned.

6.9 CUT AND FILL SLOPES

The following sections provide recommendations for cut and fill slopes up to 5 feet high and up to 6 feet for embankment surcharge. If cut or fill slopes greater than these heights are planned, Pali Consulting should be contacted for additional geotechnical evaluation. Cut and fill slopes should be planted with appropriate vegetation as soon as possible after grading to provide protection against erosion.



6.9.1 Cut Slopes

Permanent cut slopes should be limited to an inclination of 3 horizontal to 1 vertical (3H:1V) or flatter unless supported by retaining structures. If groundwater seepage occurs within any slope, flatter slopes or structural measures may be needed for stability. Pali Consulting should be contacted if such seepage occurs.

6.9.2 Fill Slopes

Permanent fill slopes up to 5 feet high should not exceed 2H:1V gradients, or 3H:1V if to be mowed or otherwise maintained. When placed on existing slopes steeper than 5H:1V, the fill should be keyed into the existing ground by excavating a series of stair-stepped horizontal benches before placing the fill, per the International Building Code (International Code Council, 2024). The need for keying and benching should be evaluated in the field on a case-by-case basis by the geotechnical engineer.

6.10 DRAINAGE

6.10.1 Temporary Drainage

Positive temporary drainage of surface water should be maintained during construction to prevent standing water and/or erosion of the soil within the working surface. The Contractor should keep all final subgrades free of water.

6.10.2 Surface Drainage

Site grading should be planned so as to drain water away from the roadway and other impermeable surfaces to suitable stormwater facilities or collected and tied into a storm water collection system. Stormwater should not be directed to or discharge onto site slopes.

6.10.3 Subsurface Drainage

Subsurface drains are not anticipated to be required since the proposed roadway is elevated 5 feet or more above the existing ground surface.

7.0 CONSTRUCTION OBSERVATIONS

Satisfactory earthwork performance depends 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 the subsurface exploration program. Recognition of changed conditions often requires experience; therefore, Pali Consulting staff 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 those assumed in this report, and that the intent of project plans and specifications relating to earthwork are being met. In particular, we recommend that site preparation of embankment fill/pavement subgrades be observed by Pali Consulting. Pali Consulting should also observe and perform density tests of all structural fill, base rock, and AC pavements, to confirm that the specified compaction is met. Testing of CAS should be completed by Pali Consulting, if used at the site.



8.0 LIMITATIONS

We have prepared this geotechnical evaluation and report for use by Waterways and the Coos County SWCD for the proposed Green Acres Lane Relocation project. Our report is intended to evaluate geotechnical conditions and provide geotechnical design recommendations for the proposed improvements. However, all geotechnical design involves risks, only part of which can be mitigated through geotechnical engineering and construction practices. Favorable performance of slopes 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.

9.0 REFERENCES

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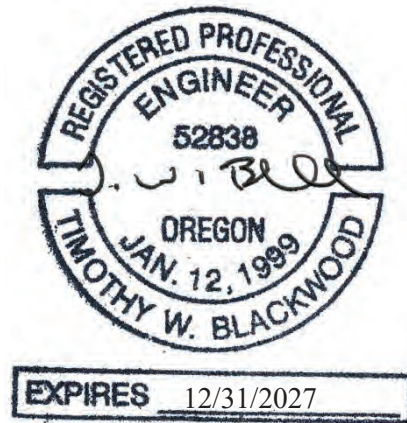
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11.0 CLOSING

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

Rolf Hyllseth, PE (Washington)
Principal Geotechnical Engineer

Attachments

- Figure 1 – Vicinity Map
- Figure 2 – Site & Exploration Plan
- Figure 3 – Monitoring Plate Detail
- Appendix A – Field Explorations and Laboratory Testing
- Appendix B – Slope Stability Analysis Methods and Results

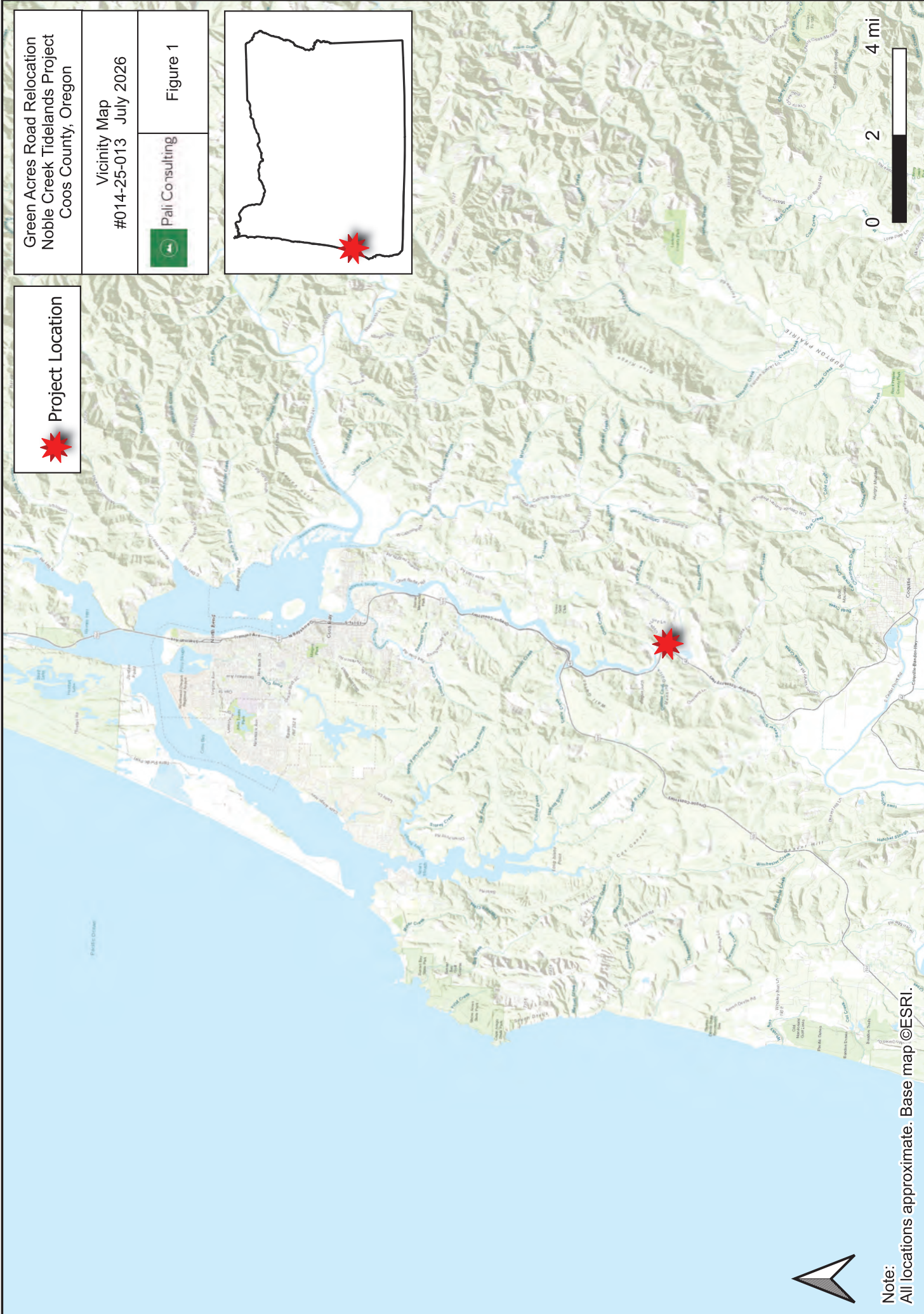
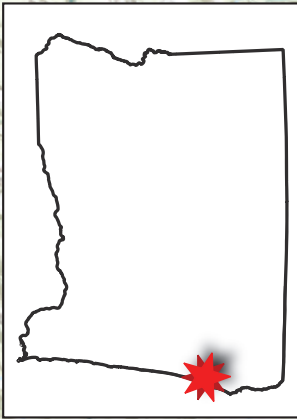
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 Project Location

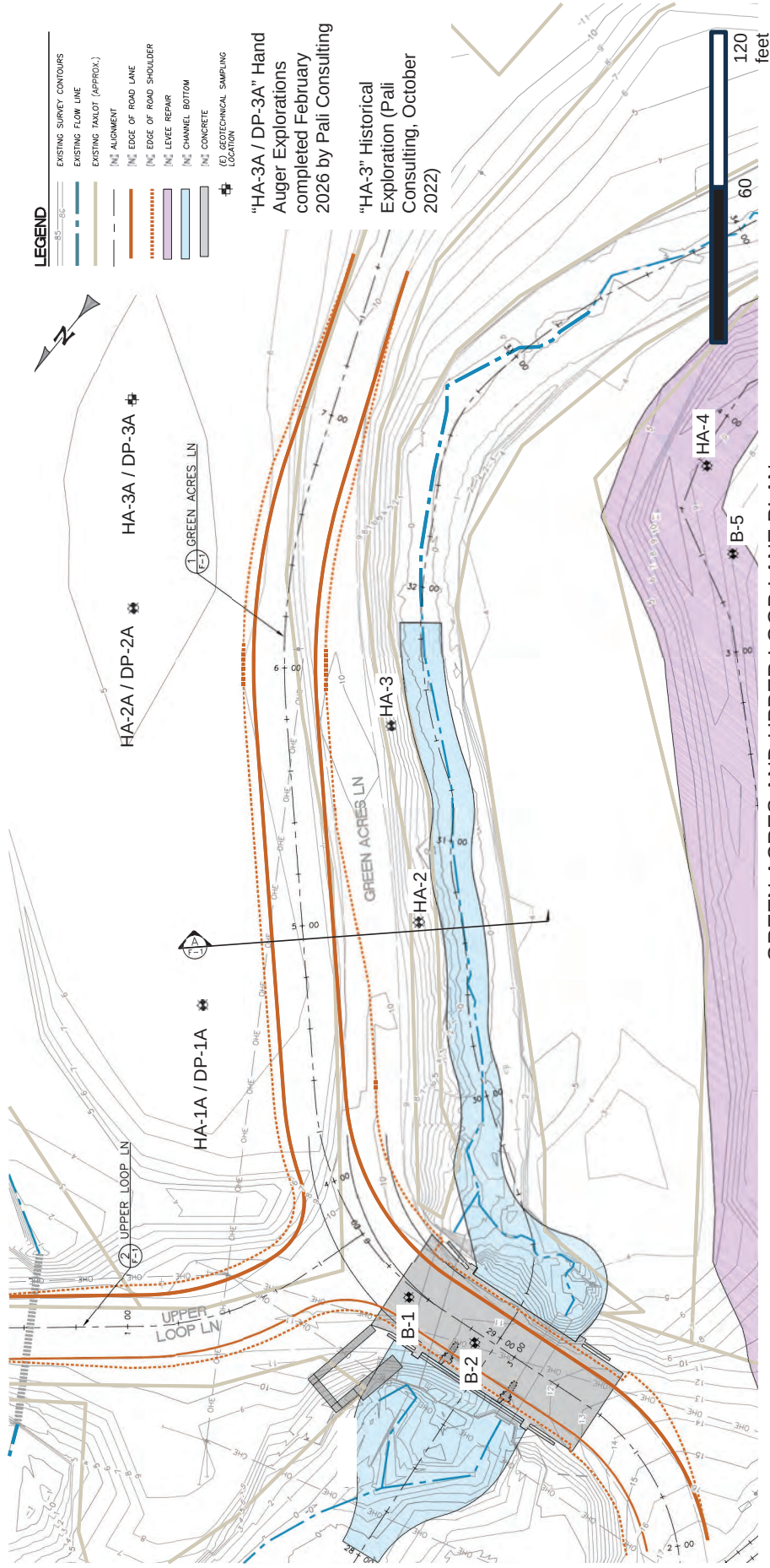
Green Acres Road Relocation
Noble Creek Tidelands Project
Coos County, Oregon

Vicinity Map
#014-25-013 July 2026

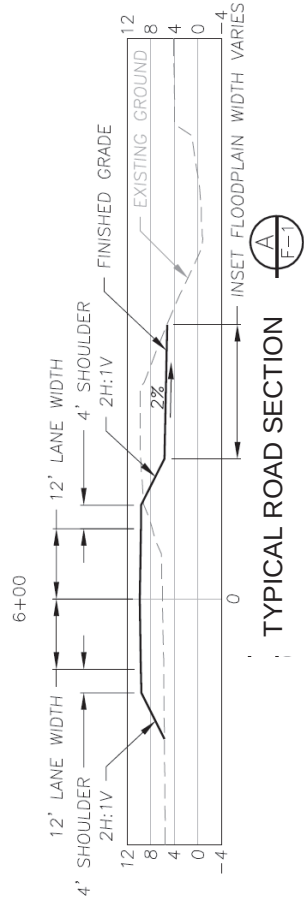
 Pali Consulting
Figure 1



Note:
All locations approximate. Base map ©ESRI.

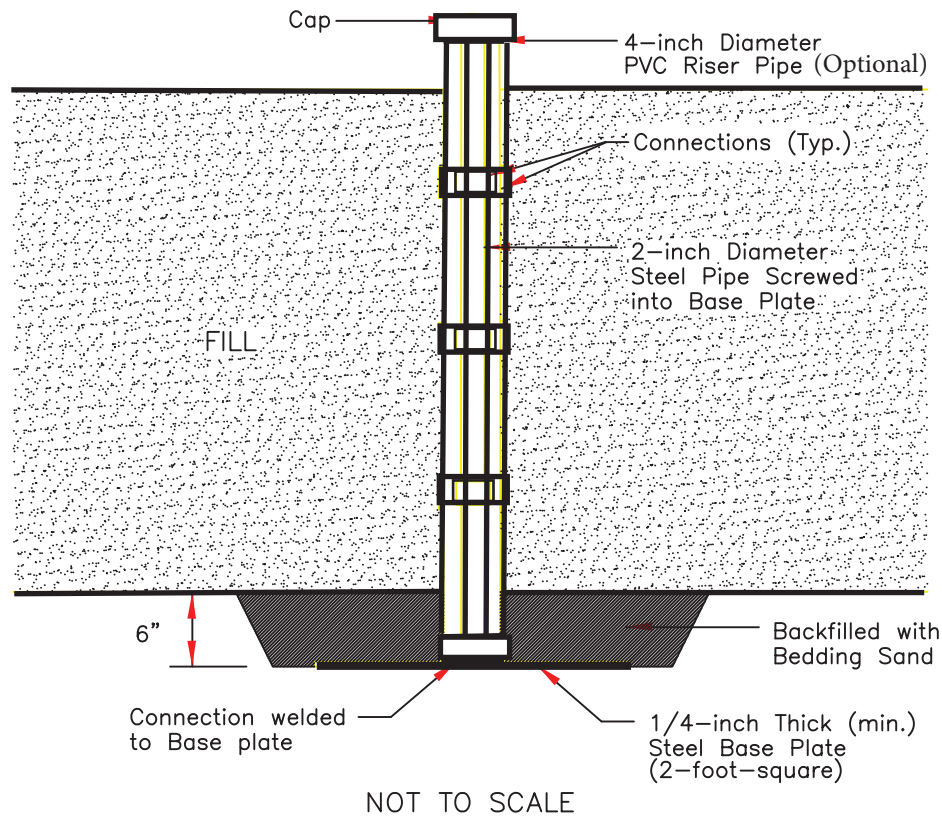


GREEN ACRES AND UPPER LOOP LANE PLAN



Note:
Figure adapted from Green Acres Ln and Upper Loop Ln Plan,
Provided by Waterways Consulting, Inc, File Date 6/1/2026

Green Acres Road Relocation Noble Creek Tidelands Project Coos County, OR	
Site & Exploration Plan #014-25-013 July 2026	
 Pali Consulting	Figure 2



NOTES

1. LOCATIONS OF SETTLEMENT PLATES SHALL BE CLEARLY MARKED AND READILY VISIBLE (RED FLAGGED) TO EQUIPMENT OPERATORS.
2. IN THE EVENT OF DAMAGE TO SETTLEMENT PLATE OR EXTENSION RESULTING FROM EQUIPMENT OPERATING WITHIN PRESCRIBED CLEARANCE AREA, CONTRACTOR SHALL IMMEDIATELY NOTIFY SOILS ENGINEER AND SHALL BE RESPONSIBLE FOR RESTORING THE SETTLEMENT PLATES TO WORKING ORDER.
3. INSTALL PLATES ON FIRM GROUND OR ON SAND PADS IF NEEDED FOR STABILITY. TAKE INITIAL READING ON TOP OF PIPE PRIOR TO PLACEMENT OF FILL.
4. FOR EASE IN HANDLING, ROD AND CASING ARE USUALLY INSTALLED IN 5-FOOT SECTIONS. AS FILL PROGRESSES, COUPLINGS ARE USED TO INSTALL ADDITIONAL LENGTHS. CONTINUITY IS MAINTAINED BY READING THE TOP OF THE MEASUREMENT ROD, THEN IMMEDIATELY ADDING THE NEW SECTION AND READING THE TOP OF THE ADDED ROD. BOTH READINGS ARE RECORDED.
5. RECORD THE ELEVATION OF THE TOP OF THE MEASUREMENT ROD AT THE RECOMMENDED TIME INTERVALS. EACH TIME, NOTE THE ELEVATION OF THE ADJACENT FILL SURFACE.
6. READ THE MEASUREMENT ROD TO THE NEAREST 0.005 FOOT. NOTE THE FILL ELEVATION TO THE NEAREST 0.1 FOOT.
7. THE ELEVATIONS SHOULD BE REFERENCED TO A TEMPORARY BENCHMARK LOCATED ON STABLE GROUND AT LEAST 200 FEET FROM THE EMBANKMENT.

Green Acres Road Relocation, Coos County, OR	
Settlement Plate Detail	
Project# 014-25-013	July 2026
Pali Consulting, Inc.	Figure 3



Pali Consulting

APPENDIX A - FIELD EXPLORATIONS AND LABORATORY TESTING



FIELD EXPLORATIONS

GENERAL

We evaluated subsurface soil and groundwater conditions at the site by completing three hand augers and four drive probes on February 11, 2026. The hand augers and drive probes were completed by a staff engineer from Pali Consulting. We have also included subsurface explorations from our previous Noble Creek Report. The approximate locations of the explorations are shown on Figure 2 of the report. Exploration and drive probe logs are included on Figures A-1 through A-4 of this Appendix.

SOIL 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.

SAMPLING AND LOGGING

Soil samples were collected from the test pits at the intervals noted on the exploration logs in this Appendix. Sampling was completed by collecting “grab” samples from the spoils at target depths. All disturbed samples obtained were sealed in watertight containers and transported to our laboratory for subsequent classification and testing.

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.



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. We have also included laboratory test results from our previous Noble Creek Report. The tests were performed in general accordance with the test methods of the ASTM or other applicable procedures. Test results are indicated on the exploration logs and on Figure A-5 of this Appendix.

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.

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 and Figure A-1 in this Appendix.

SOIL CONSOLIDATION

Consolidation testing was completed on two samples by Northwest Testing, Inc. in general accordance with ASTM Test Method D 2435, and included in our Noble Creek Report. The test results are presented 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	TYPICAL DESCRIPTIONS
			LETTER	
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL-GRADED GRAVELS, GRAVEL - SAND - MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM	SILTY GRAVELS, GRAVELS - SAND - SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL- SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN 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 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

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

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

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-1A

Surface Elevation:
Boring Date: 2/11/2026
Boring Location:
Drilling Method: Hand Auger

File: C:\Users\Tech\Pali Consulting\Dropbox\1-Projects\Active-Projects\014-Waterways\014-25-013-GreenAcresLnRelocation\Analysis\HA_Logs.log Date: 7/8/2026

Depth	Remarks	Moisture (%)	Dry Density	Blow Counts	Sample Type	Water Table
0					TP	Top Soil
S1					SM	Very soft, moist, reddish brown clayey SAND, with organics (rootlets) [FILL]
S2	51				MH	Soft, moist, grey to brown with red mottling ELASTIC SILT with organics (rootlets) [NATIVE]
S3						
S4						Grades to wet
S5	85					Increase in organics (peat-like)
S6						Grades to tan color lensed with minor charcoal (strong organic scent)
S7	102					Grades to very soft, wet dark brown with grey to red mottling (peat + strong organic scent)
S8	117					Grades to dark grey with less organics (rootlets)
S9						
5					END	HA-1 terminated at 5 ft bgs due to caving. Groundwater observed in auger hole during excavation.

LOG OF BORING



Pali Consulting

Green Acres Ln Relocation
Project No. 014-25-013

A-1

HA-2A

Surface Elevation:
Boring Date: 2/11/2026
Boring Location:
Drilling Method: Hand Auger

File: C:\Users\Tech\Pali Consulting Dropbox\1-Projects\Active-Projects\014-Waterways\014-25-013-GreenAcresLnRelocation\Analysis\HA_Logs.log Date: 7/8/2026

Depth	Remarks	Moisture (%)	Dry Density	Blow Counts	Sample Type	Water Table
0					TP	Top Soil
					SM	Soft, wet, brown sandy SILT with organics (rootlets) [FILL]
S1	86				MH	Very soft, wet, brown to grey ELASTIC SILT with organics (rootlets) [NATIVE] Grades to grey with reddish-brown mottling
S2	91					
S3						
S4						
2						
S5	104					Grades to grey, increase in organics (rootlets and minor wood chips)
S6	140					Increase in organics (peat-like)
3						
S7						As above with less organics
4						
S8						
5					END	HA-2 terminated at 4.6 ft bgs due to caving. Groundwater observed in auger hole during excavation.
6						
7						

LOG OF BORING



Pali Consulting

Green Acres Ln Relocation
Project No. 014-25-013

A-2

HA-3A

Surface Elevation:
Boring Date: 2/11/2026
Boring Location:
Drilling Method: Hand Auger

File: C:\Users\Tech\Pali Consulting\Dropbox\1-Projects\Active-Projects\014-Waterways\014-25-013-GreenAcresLnRelocation\Analysis\HA_Logs.log Date: 7/8/2026

Depth	Remarks	Moisture (%)	Dry Density	Blow Counts	Sample Type	Water Table
0					TP	Top Soil
					MH	Soft, moist to wet, brown to grey ELASTIC SILT with much organics (rootlets) [NATIVE]
S1	70					Grades to grey with reddish brown mottling with less organics
S2						
S3						
S4	115					Grades to dark brown
S5	103					Increase in organics (peat-like)
S6	172					Grades to very soft, wet, dark grey, with much organics (wood)
					END	HA-3 terminated at 4.2 ft bgs due to caving. Groundwater observed in auger hole during excavation.

LOG OF BORING

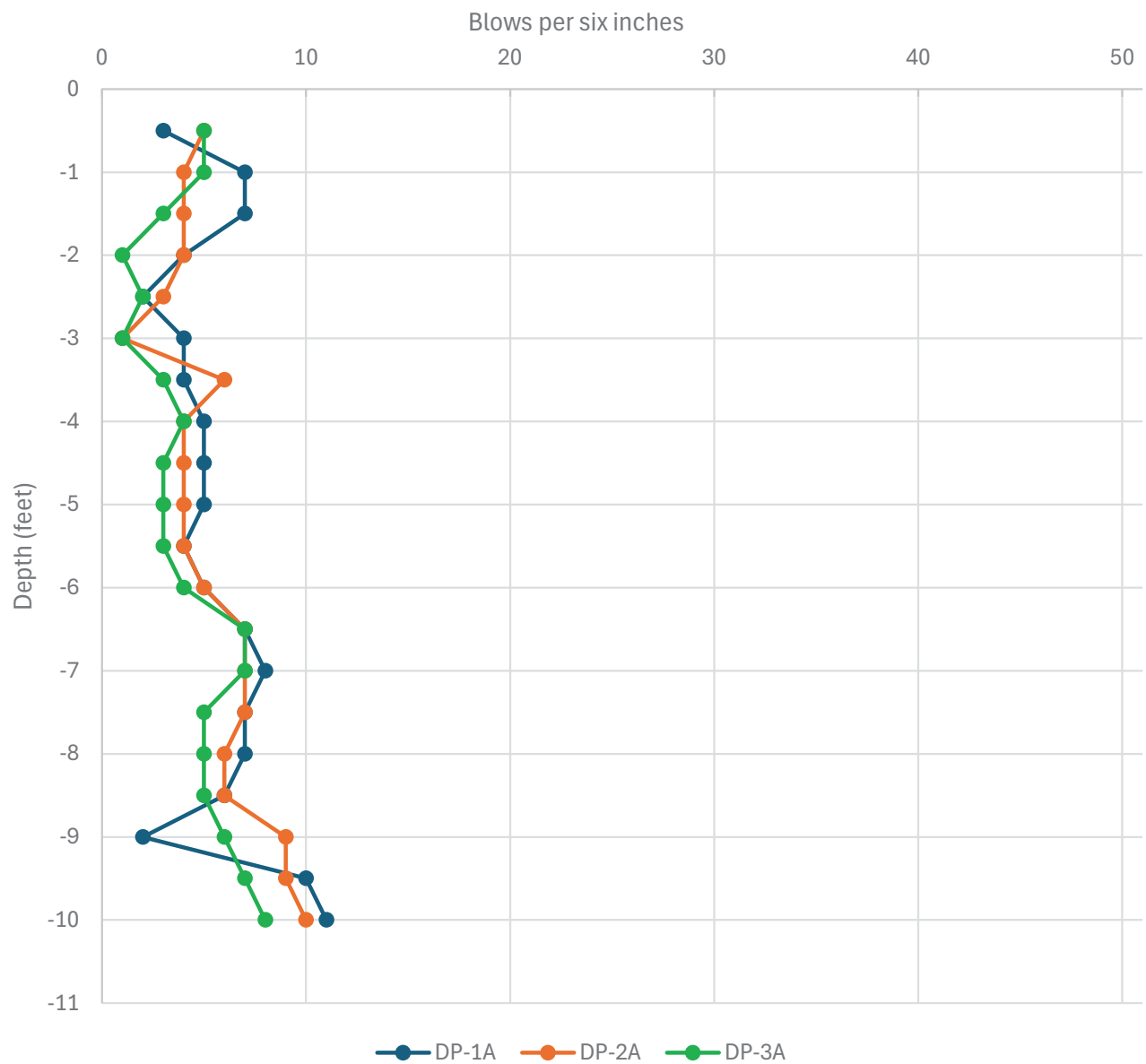


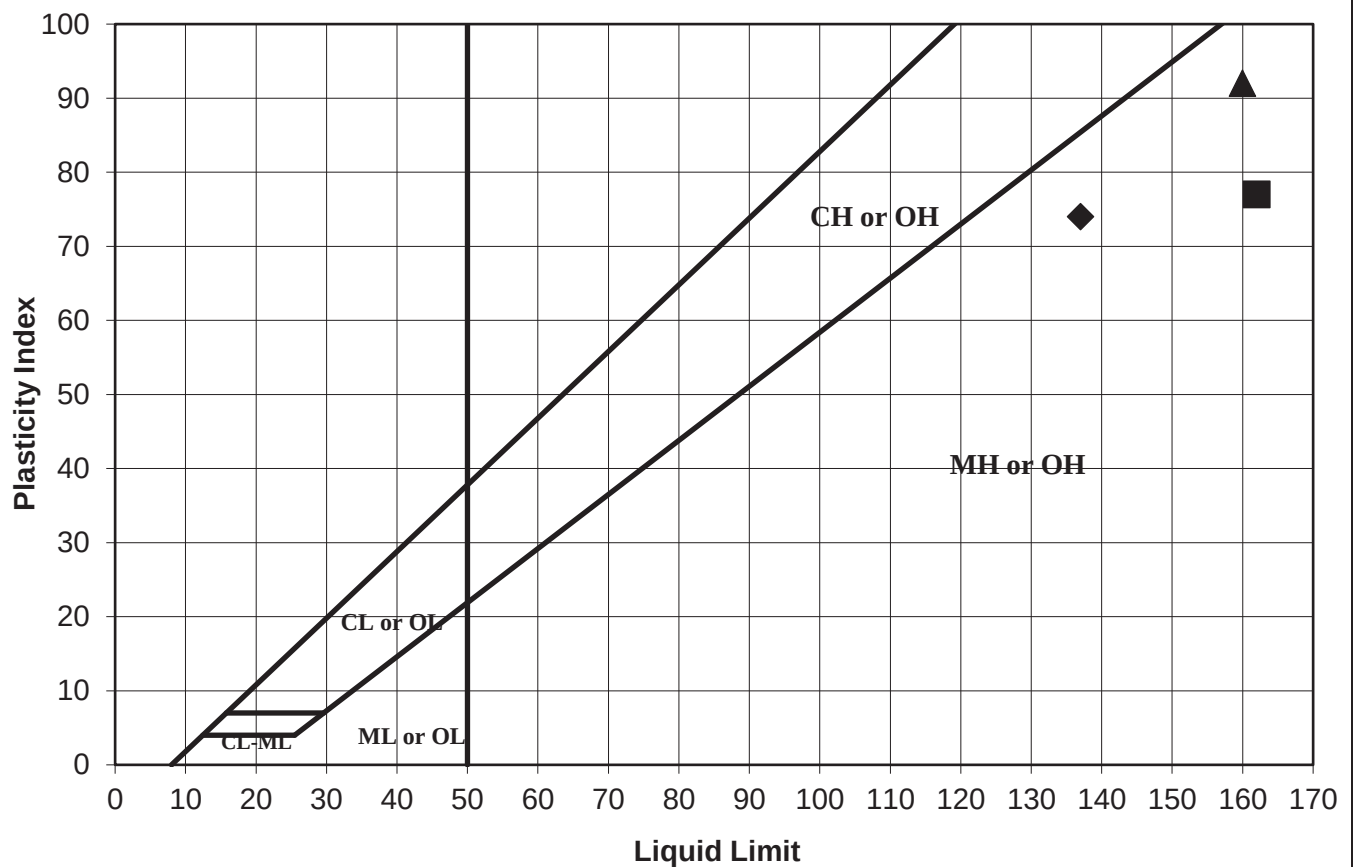
Pali Consulting

Green Acres Ln Relocation
Project No. 014-25-013

A-3

Drive Probes, Green Acres Lane





Atterberg Limits Determination

Symbol	Boring	Sample	Depth (ft)	Liquid Limit	Plastic Limit	PI	Classification
◆	HA-1A	S-8	4	137	63	74	MH
■	HA-2A	S-6	2.5	162	85	77	MH
▲	HA-3A	S-6	3.8	160	68	92	MH

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ATTERBERG LIMITS - ASTM D4318

Green Acres Ln Relocation
Project No.: 014-25-013



Pali Consulting

A-5

NOBLE CREEK 2022 REPORT
FIELD EXPLORATIONS AND LABORATORY TESTING

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
0					CH	Soft, moist, dark blue-grey FAT CLAY with much organics
1	120					Grades to wet
2	132					Grades to mottled light and dark grey
3					END	HA-2 terminated at 2 ft bgs due to caving
4						
5						
6						
7						

LOG OF BORING



Pali Consulting

Noble Creek
Project No. 014-21-006

A-5

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

HA-3

Surface Elevation: 4'

Boring Date: 8/26/22

Boring Location: Green Acres Ln Embankment

Drilling Method: Hand Auger

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

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
3					END	HA-3 terminated at 2.67 ft bgs due to caving
4						
5						
6						
7						

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

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

Depth	Remarks	Moisture (%)	Dry Density	Blow Counts	Sample Type	Water Table
0						
1	%F=71	27				
2						
3						
4		71				
5		89				
6						
7						

GWT not encountered

SM

Stiff, dry, tan, SANDY SILT with occasional small gravels, roots (levee fill)

Grades to tan with minor light grey mottling

Grades to medium stiff, tan and grey, decreasing sand

With minor rusty red mottles

Grades to moist, decreasing sand

CH

Grades to soft, mottled, brown and grey, sandy FAT CLAY with charcoal (native)

Grades to grey with rusty red mottles, decreasing sand, increasing moisture, with much charcoal and organics

Grades to wet, blue-grey

HA-4 terminated at 6.25 ft bgs due to caving

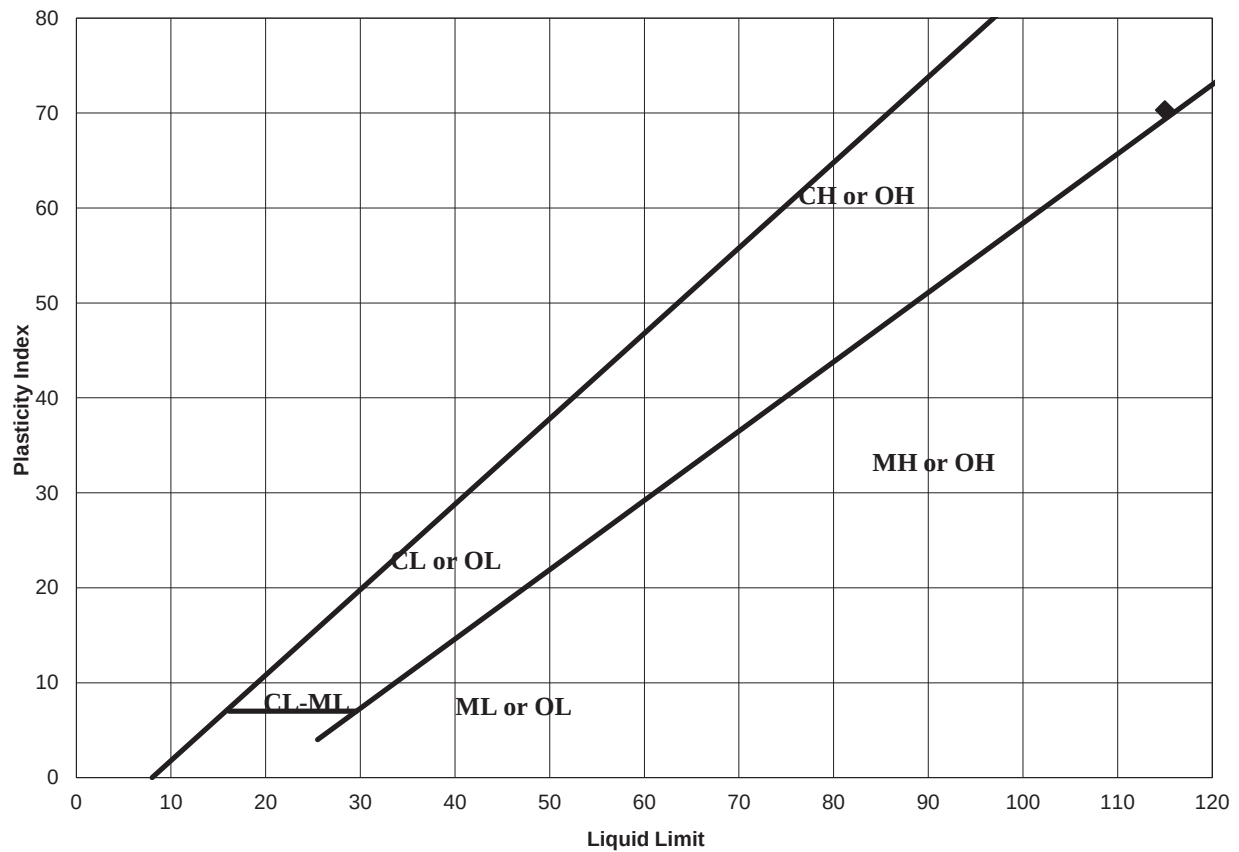
LOG OF BORING



Pali Consulting

Noble Creek
Project No. 014-21-006

A-7



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
 Plastic Limit = 45
 Plasticity Index = 70

Date: 10/13/2022
 Tested By: JLE
 Checked By: TWB

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ATTERBERG LIMITS - ASTM D4318

Page A-8

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
S1		20		2-2-2	4	GWT not encountered	0		AC ML Asphalt roadway surface underlain by 6" well graded gravel (Roadway Base Rock) Soft to medium stiff, moist, brown, SANDY SILT (FILL)	43	AL, PI=60, LL=100, PL=40 DD=27 pcf AL, PI=71, LL=120, PL=49 CS %F=64	
S2		50		0-1-1	2		5		CH Soft, wet, grey, FAT CLAY with minor charcoal (NATIVE) Grades to dark grey	66		
S3		75										
S4		100		0-0-0	0		10		MH Very soft, wet, dark grey ELASTIC SILT with variable grey sand	83		
S5		100										
S6		100		0-0-1	1		15		With interbedded very loose, wet, dark grey SILTY SAND, with minor charcoal and wood	83		
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 Grades to light grey	39		
S9		100					30					

 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

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

B-2

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
S1		35		4-4-4	8		0		AC MH Asphalt roadway surface underlain by 6" well graded gravel (Roadway Base Rock)		
N/A		0		0-0-0	0		5		SM Very loose to loose, moist, dark grey SILTY SAND (NATIVE)	33	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'

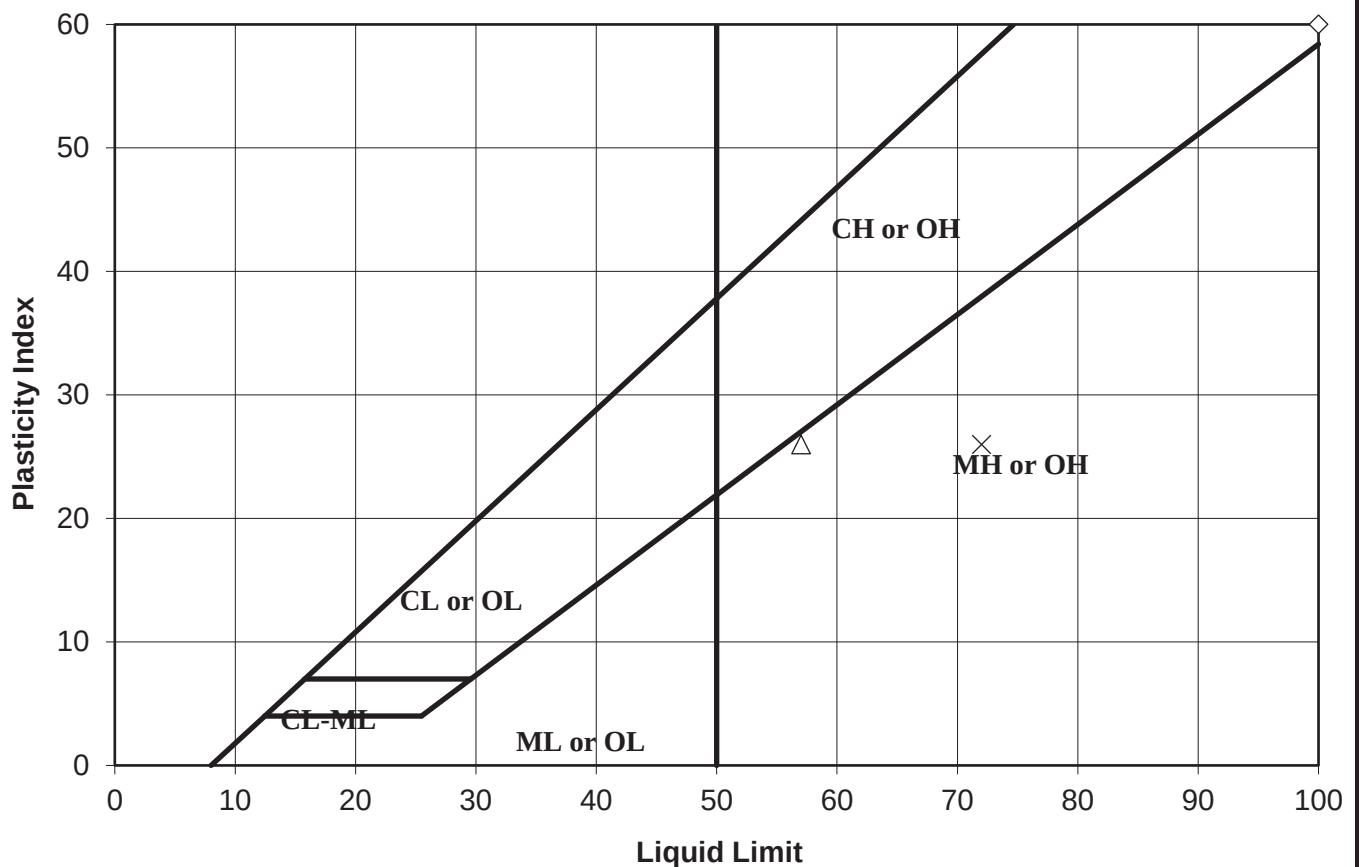
Date: 8/18/2023

File: C:\Users\june.pali\Consulting\Dropbox\1-Projects\Active-Projects\014-21-006\NobleCreek\02-GeotechnicalDesign\Analysis\NobleCreekLogs.log

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			

Pali Consulting							Noble Creek Tidelands Project Coos County, OR			B-5		
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: 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
S1		100		0-0-0	0	GWT not encountered	0		CL	Very soft, moist, varicolored (yellow/orange/gray/brown) SANDY CLAY with minor charcoal and organics (LEVEE FILL) Grades to brown-grey	53	DD=66 pcf
S2		100					5					
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)			
S4		0					10			Grades to dark grey, moist to wet, with increasing sand	103	AL, PI=88, LL=140, PL=52
S5		100										
S6		100		0-0-1	1		15			76		
S7		100					20			Grades to grey. decreasing sand	115	
S8		100		0-0-0	0		25			Grades with increasing sand, wood, organics, and charcoal		
S9		100		0-0-0	0		30			Grades to brown-grey, with minor sand		

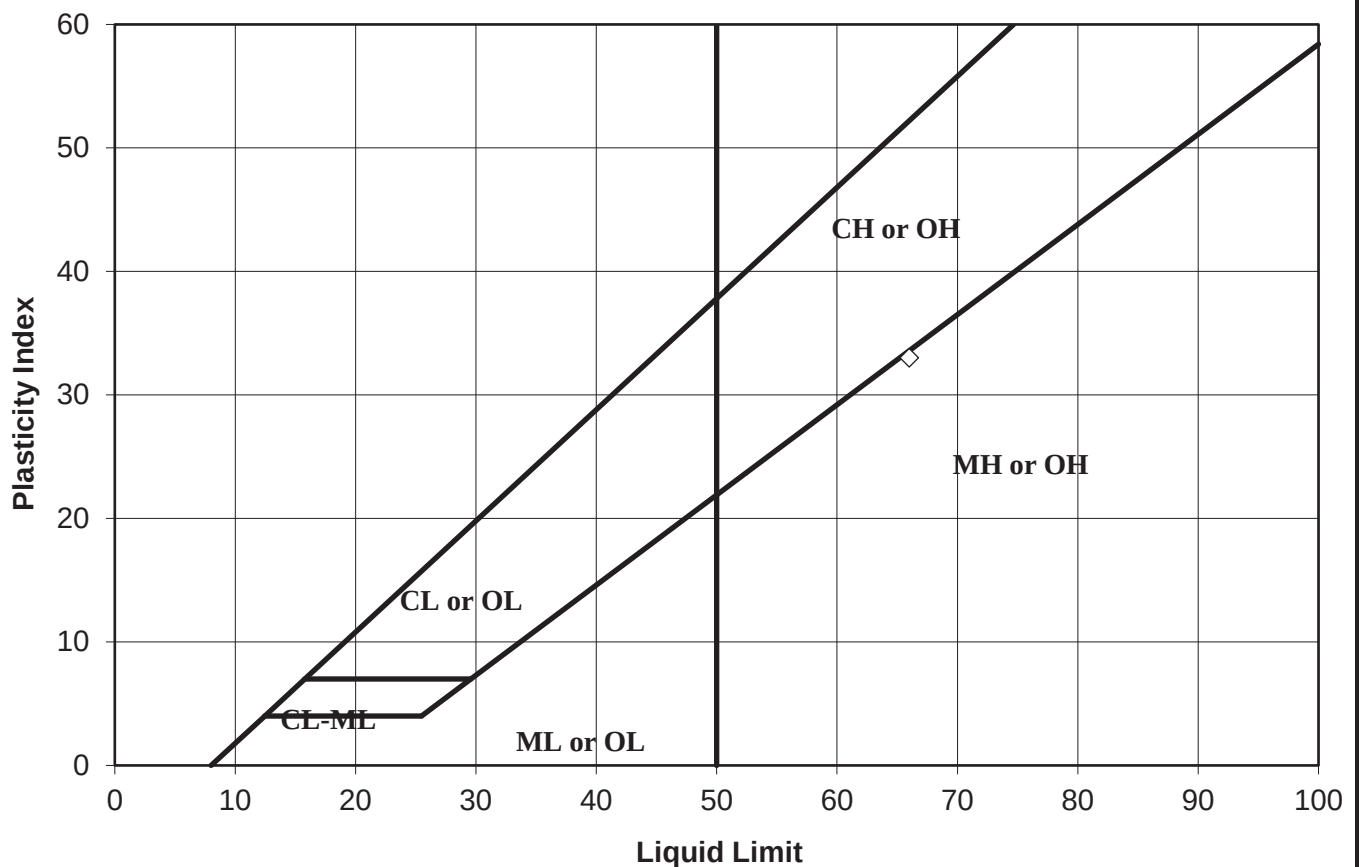
 Pali Consulting							Noble Creek Tidelands Project Coos County, OR			B-5		
Project: Noble Creek Tidelands Project							Driller: Western States Soil Conservation, Inc					
Proj No. 014-21-006-2							Date: 5/9/23					
Drilling Method: Mud Rotary							Elevation: 9'					
Diameter: 4"			Water Table: N/A				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			
S11		100					40					DD=47 pcf
S12		100		0-0-0	0		45		Grades to grey, with increasing sand	48		%F=75
S13		100		0-0-1	1		55		With minor sand	42		
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			



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

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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
×							

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ATTERBERG LIMITS - ASTM D4318	
Noble Creek Tidelands Project Project No.: 014-21-006-2	
 Pali Consulting	Figure B-1b

TECHNICAL REPORT**Report To:** Tim Blackwood, PE, GE, CEG
Pali Consulting, Inc.
1120 SW Fifth Avenue, Suite 1302
Portland, Oregon 97204**Date:** 6/16/2023**Lab No.:** 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	
Dry Unit Weight, (pcf)		51.4	
Height of Specimen, (inches)		1.0000	
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.

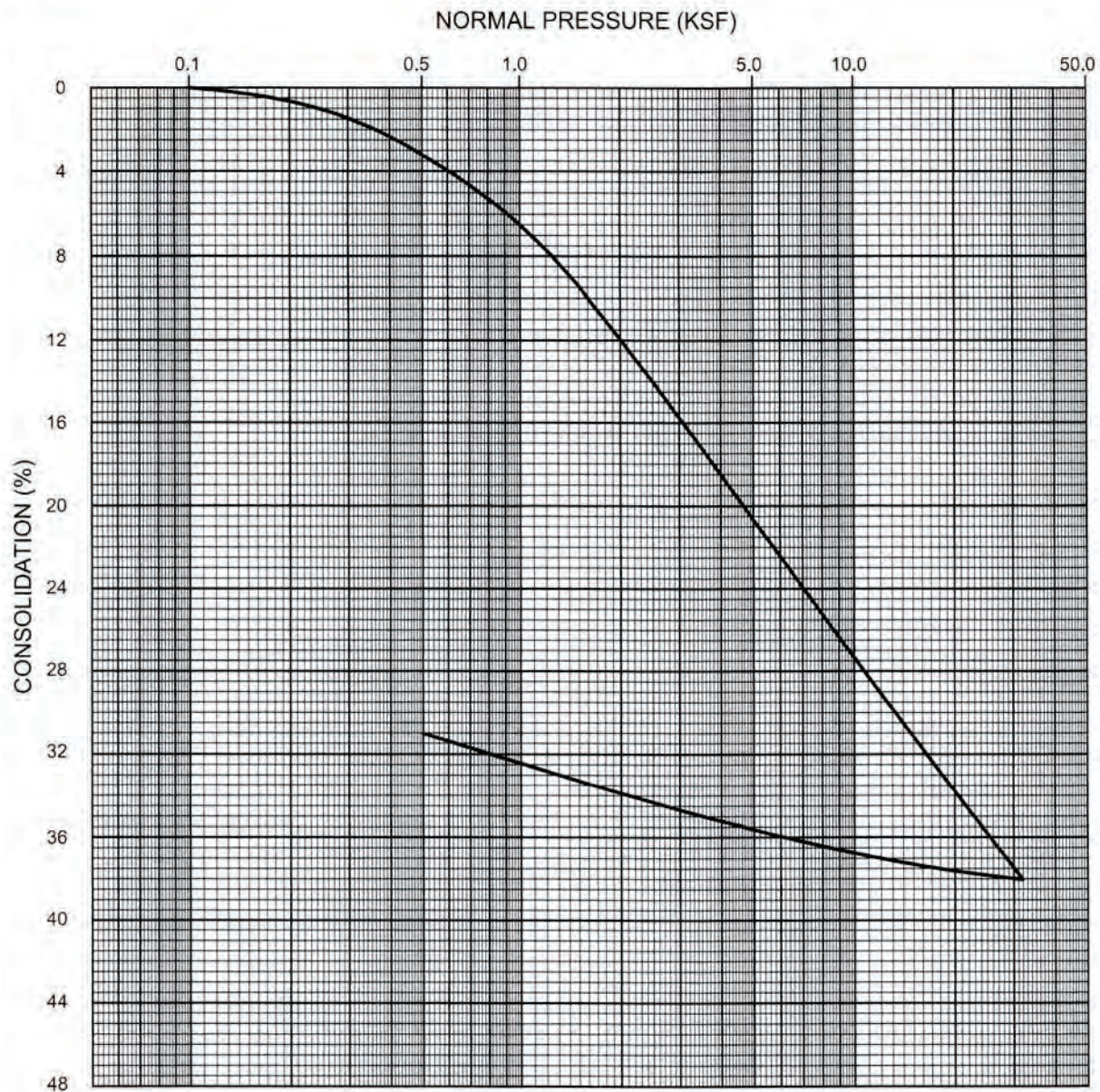
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SHEET 1 of 3

REVIEWED BY: Michael A. Ginsbach 

TECHNICAL REPORT

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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.
NOBLE CREEK

LAB NO. 23-095

TECHNICAL REPORT

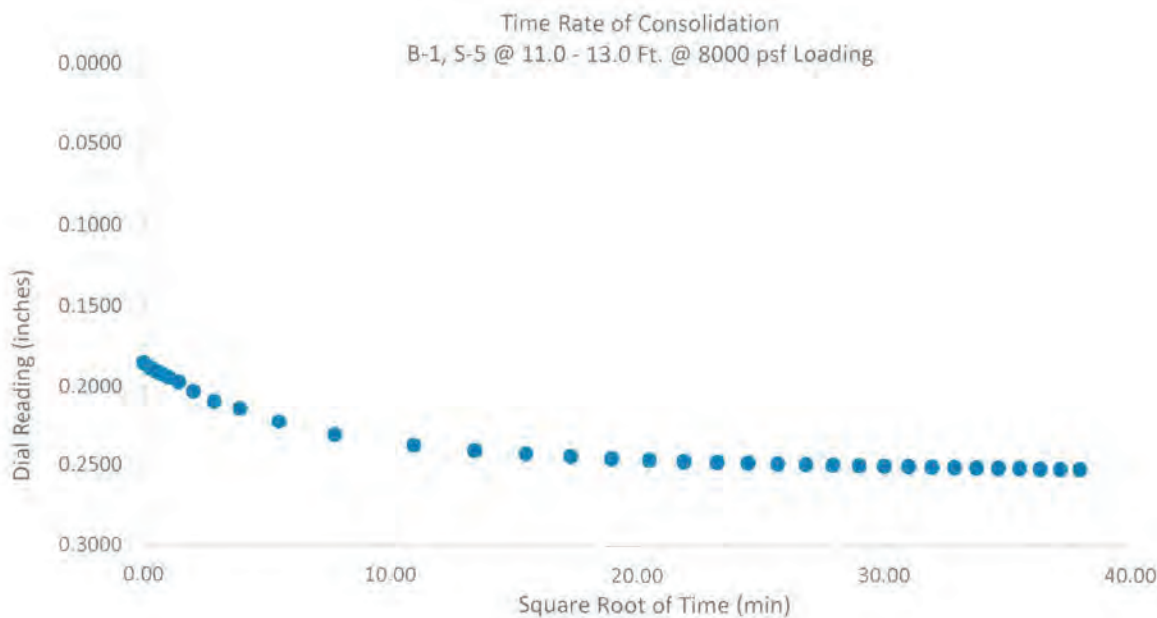
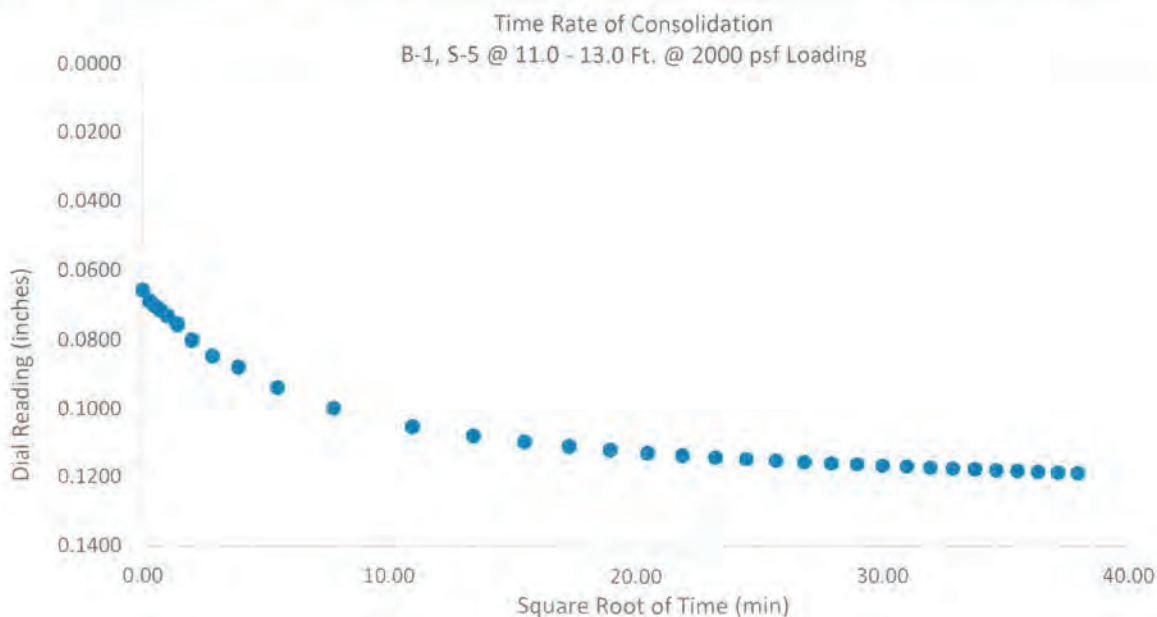
Report To: Tim Blackwood, PE, GE, CEG
Pali Consulting, Inc.
1120 SW Fifth Avenue, Suite 1302
Portland, Oregon 97204

Date: 6/16/2023

Lab No.: 23-095

Project: Noble Creek

Project No.: 00-223425-1



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SHEET 3 of 3

REVIEWED BY: Michael A. Ginsbach

TECHNICAL REPORT

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**ATTACHMENT B -
SLOPE STABILITY ANALYSIS METHODS AND
RESULTS**

SLOPE STABILITY ANALYSIS METHODS AND RESULTS

General

To evaluate the global stability of the proposed roadway embankment, we completed numerical slope stability analyses of the planned embankment configuration fill slopes at the site using the commercially available, computer analysis software Slide2 (Rocscience; version 3.034). Slide2 uses a two-dimensional, limit equilibrium analysis method to evaluate slope stability by determining the global factor of safety (FS) against slope instability under specific design cases with varying input parameters. The FS can be generalized as the ratio of forces resisting slope movement (soil strength, weight of soil mass, etc.) to forces driving slope movement (gravity, earth pressure, seismic loading, etc.). A FS less than 1.0 indicates a condition when the available resisting forces are less than the driving forces, resulting in slope failure. The program also predicts the location and geometry of “critical” analysis failure surfaces, which represent the zones within the soil that exhibit the lowest FS. Our analyses were completed using the Bishop Simplified Method, which satisfies moment and force equilibrium.

Our analyses were completed along the typical road section subsurface soil profile A-A’ located near the central portion of the site, as indicated on Figure 2. Slope stability was analyzed under both static and seismic (pseudo-static) conditions. Following conventional practice, the pseudo-static loading was assumed to be half of the site-adjusted, peak ground acceleration (PGA) associated with the design earthquake (975-year event).

Our assessment analyzed the following cases:

- o Case 1 – Static;
- o Case 2 – Seismic (pseudo-static, 1/2 PGA);

Design Material Properties

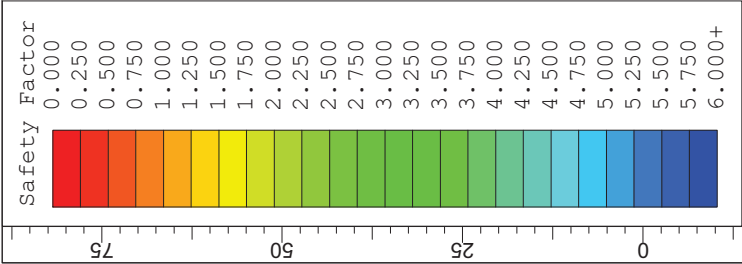
Our stability analyses were completed using soil properties estimated based on the strength/behavior characteristics derived from laboratory testing, published literature values, the values used in our Noble Creek Report (where applicable), and our experience. The specific Slide2 material properties used in our stability analyses are presented in a summary table on each analysis case computer printout included in this appendix.

Stability Analysis Results

The minimum global FS results from our slope stability analyses are summarized below in Table A-1. Graphical results for the cases analyzed are included in Figures A-1 and A-2 of this appendix. These results are further discussed within the text of this report.

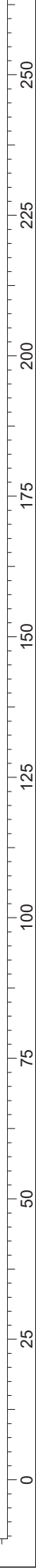
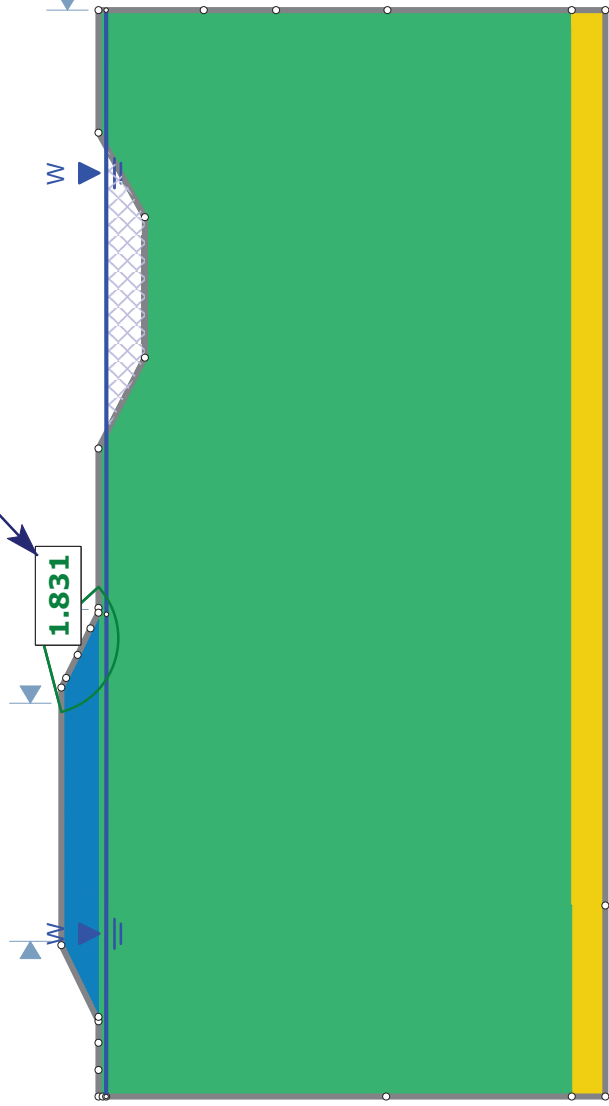
Table A-1 – Global Stability FS Summary

Figure	Design Case	Global Minimum FS
A-1	1 – Static	1.831
A-2	2 – Seismic (pseudo-static, 0.5 PGA)	0.476



Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu
Clayey Sand		110		Mohr-Coulomb	50	26			Water Table	Custom	1
FAT CLAY		90	95	Vertical Stress Ratio			0.4	150	Water Table	Custom	1
Granular Fill		125	130	Mohr-Coulomb	0	36			Water Table	Custom	1

Results
Bishop simplified method showing all
Surfaces with a factor of safety below 1.300
1.831



Green Acres Road Relocation

Slope Stability Analysis

32 ft Wide Roadway (5 ft High)

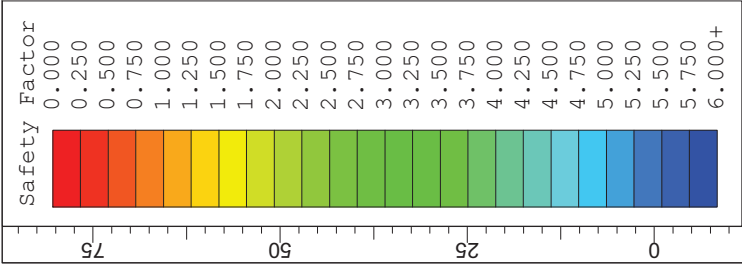
Static

Scale 1:319

7/8/2026

Pali Consulting

Figure A-1

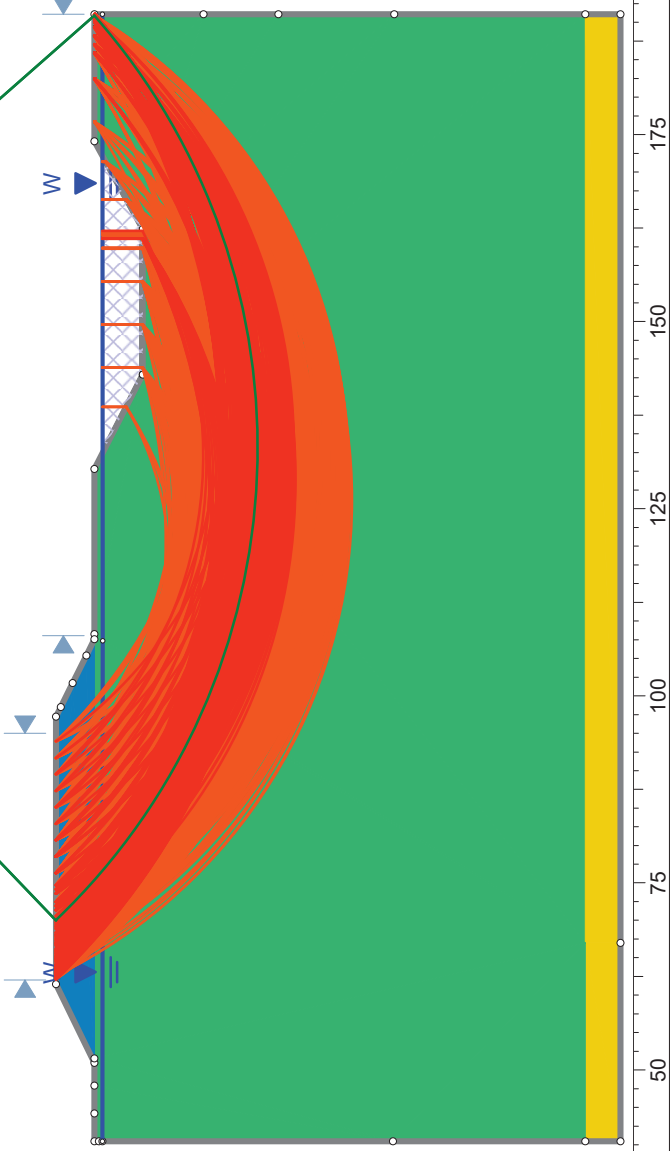


0.476



Material Name	Color	Unit Weight (lbs/ft3)	Saturated U.W. (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Hu Type	Hu
Clayey Sand		110		Mohr-Coulomb	50	26			Water Table	Custom	1
FAT CLAY		90	95	Vertical Stress Ratio			0.4	150	Water Table	Custom	1
Granular Fill		125	130	Mohr-Coulomb	0	36			Water Table	Custom	1

Results
Bishop simplified method showing all
Surfaces with a factor of safety below 0.550
0.476



APPENDIX C

ALTERNATIVES PRESENTATION TO STAKEHOLDERS AND LANDOWNERS (FILES ATTACHED SEPARATELY)

APPENDIX D

SUMMARY OF EXISTING AND PROPOSED CONDITIONS MODELING RESULTS (FILES ATTACHED SEPARATELY)