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Windsor Engineers
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Report of Geotechnical Services
Manzanita – Classic Street and Necarney City Road,
Stormwater Improvements and Water Main Extension Project
Manzanita, Oregon
Project #074-24-015

1.0 INTRODUCTION

This report provides Pali Consulting Inc.'s (Pali Consulting's) geotechnical evaluation and recommendations for the Manzanita – Classic St Road and Necarney City Road Stormwater Improvements and Water Main Extension Project (Project) in Manzanita, Oregon. The Project consists of geotechnical design recommendations for water system improvements to Classic Street and Necarney City Road. Improvements will include installation of a water main line, stormwater facilities as necessary, pavement widening, and installation of a shared use path along Classic Street. The Project area is shown on Figure 1.

Windsor Engineers requested that Pali Consulting provide geotechnical services for the Project. Our scope of work included a review of existing information, site reconnaissance, subsurface explorations, geotechnical analyses, and design recommendations for the Project. Our work was completed in general accordance with Task Order 09 of our master services agreement with Windsor Engineers, dated November 12, 2024, and subsequent modifications.

2.0 BACKGROUND REVIEW

2.1 PROJECT DESCRIPTION

The Project includes installation of a water mainline and appurtenant facilities in an undeveloped area known as the Highlands. The water system improvements will allow development of the area and will connect existing branches of the City of Manzanita's water system.

The new water mainline will consist of 12-inch diameter HDPE pipe that will be installed with open trench and/or trenchless methods at depths of between 3 and 5 feet below ground surface (bgs) for much of the alignment. The current proposed alignment will follow existing roadways between the intersection of Classic Street and Laneda Avenue and the junction of Highlands Drive and Meadows Drive, then run overland to

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Necarney City Road, where it will tie in with the existing water line near the junction of Necarney City Road and Clipper Court. The approximate alignment is shown on Figure 1.

2.2 BACKGROUND INFORMATION

2.2.1 Topography and Geomorphic Features

We reviewed U.S. Geological Survey (USGS) topographic maps, satellite imagery, as well as LiDAR data downloaded from the Oregon Department of Geology and Mineral Industries' online mapping portal (DOGAMI, online mapping accessed November 2024), for analysis of topographic and geomorphic features in the Project area along Classic Street and the Highlands area.

Topography along the alignment is mostly flat to gently sloping where it traverses a dune complex which forms the Highlands development area. About 60 feet of total relief separates the highest and lowest points in the Project area. Although originally a dune complex, the topography has been highly altered by housing, road, and infrastructure development, and little of the pre-development topography remains. The low relief hills are relict sand dunes, of which the general shape still remains. Classic Street mostly occupies a cut bench across a former dune slope so is characterized by locally steeper slopes above and below the portion within the cut. A housing development lies above Classic Street through this segment and vacant land which is slated for development below. Other roadways generally follow gentler modified dune surfaces with little relief.

2.2.2 Geologic, Landslide, and Soils Mapping

The geology within the Project area is mapped by DOGAMI. The geology consists of ancient sand dunes which were deposited in the last several hundred thousand years and have since stabilized from vegetation growth and development. These deposits are composed of eolian, or wind-deposited, fine sand.

Landslide mapping from DOGAMI's SLIDO database maps one earth slide-rotational landslide about 450 feet west of Classic Street, outside of the Project area. It is about 140 feet wide and is mapped with low confidence. No other information regarding the landslide is available.

Faults are mapped by the United States Geologic Survey (USGS) in their online Quaternary Faults and Folds Database (https://www.usgs.gov/tools/interactive-us-fault-map, accessed November 2024). The nearest mapped fault to the Project area is the Tillamook Bay fault zone, 9-10 miles south near Garibaldi. Little is known about this fault zone, but its geomorphology suggests that it is active, though at a low slip rate less than 0.2 mm/yr. Other nearby faults include unnamed faults offshore that are related to the Cascadia Subduction Zone, but little is known about these faults other than that they are likely active in the last 12,000 years with slip rates of 2.0-5.0 mm/yr.

Geologic hazards were reviewed using DOGAMI's Statewide Geohazards Viewer (HAZVU). Geologic hazards mapped along the alignment include landslides, earthquake shaking, liquefaction, coastal erosion, and tsunami inundation. Mapped landslide hazard is moderate to high along the length of Classic Street, where the steep sand cutbanks are susceptible to shallow landsliding. No hazard from deep-seated landsliding is mapped along Classic Street or in the Highlands area. DOGAMI assigns a 10-20% probability of damaging earthquake shaking in the next 50 years throughout Manzanita, including the Project area. A Cascadia Subduction Zone earthquake is expected to generate severe local shaking of 8 on the Modified Mercalli Scale, indicating widespread severe damage to structures. Earthquake shaking strong enough to be damaging would also produce liquefaction in areas of loose sediments saturated with water. The entirety of the Project area's susceptibility to liquefaction is rated as High. About half of Classic Street is within the evacuation zone for an expected Cascadia Subduction Zone tsunami, between approximately Dorcas Lane and Jackson Way.

The area's soils are mapped by the Natural Resources Conservation Service (NRCS, Web Soil Survey accessed November 2024). The Project area is entirely underlain by the Netarts fine sandy loam, present on slopes of 5 to 30 percent. It is derived from eolian dune sands, and has a typical profile of slightly decomposed plant material from 0-2 inches depth, an A horizon of fine sandy loam from 2-5 inches, an E horizon of loamy fine sand from 5-9 inches, an AB horizon of loamy fine sand from 9-15 inches, a B horizon of fine sand from 15-54 inches, and a C horizon of fine sand from 54 to 67 inches depth. It is considered well-drained with infiltration rates of 1.98 – 5.95 inches/hr. The depth to both a restrictive feature and the water table is greater than 80 inches, according to NRCS.

2.2.3 LiDAR and Aerial Imagery Review

LiDAR-generated bare earth hillshade mapping of the Project area was obtained from DOGAMI. Aerial photos from USGS Earth Explorer and Google Earth Pro for the years 1953, 1980, 1994, 2000, 2005, 2012 and 2021 were reviewed for evidence of instability or other changes. A discussion of the LiDAR imagery is provided below, and a summary of pertinent geomorphic and slope stability observations made from aerial imagery is given in Table 1.

Table 1 – Review of Aerial Imagery

Image Number	Date	Image Source	Notes
1	1953	USGS Earth Explorer	B/W photo shows that the area of Classic Street is forested and undeveloped, south of Dorcas Lane. No signs of slope instability are interpreted.
2	1986	USGS Earth Explorer	False-color photo shows that Classic Street is still undeveloped south of Dorcas Lane. No signs of slope instability are interpreted.
3	1994	Google Earth	B/W photo shows Classic Street present as a narrow unpaved road. Grading for housing developments is in progress to the southwest and southeast. Highlands Drive is not present, but the Highlands area is clear of timber. No signs of slope instability are interpreted.
4	Dec. 2005	Google Earth	Color photo shows Classic Street apparently paved. Terracing for adjacent housing developments appears complete. Highlands Drive is not present. Large areas of bare sand are visible, likely due to recent earthmoving activity. No signs of slope instability are interpreted.
5	2021	Google Earth	Color photo shows the project area much as it appears presently. All grading adjacent Classic Street is complete, and the Highlands development is partly graded as well, although Highlands Drive appears only about half constructed. No signs of slope instability are interpreted.
6	2024	Google Earth	Color photo shows project area as it appears presently. No signs of slope instability are interpreted.

Notes:

1. B/W = black and white

In summary, the air photo record shows stable dune slopes and the time history of existing development. No indications of instability were interpreted in the air photos over the approximately 70-year record.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

Classic Street is a narrow asphalt-paved road which is surrounded by development except for the central portion between Highlands Drive and Dorcas Lane where there are housing developments upslope (east) of the road and vacant land downslope (west) of the road. Where not covered by landscaping soil or vegetation, fine sand is visible throughout the Project area. Vegetation is limited to blackberries, some shrubs, occasional conifer trees, and grasses. The nature of the sandy subsoil means that there are no areas of persistent ponding or standing water.

The Project area also includes Highlands Drive, a residential connector extending northeast from Classic Street, and Necarney City Road, a paved county road which extends roughly east-west through Manzanita, as well as the undeveloped area between these two roads. The area is dominated by rolling hills of sand covered with primarily grassy vegetation.

Pavement cracking is prevalent along Classic Street between Dorcas Avenue and Highlands Drive, where the road is built on a bench cut into an old dune. Cracks are generally arcuate in shape, and areas of cracking are often noticeably subsided compared to surrounding pavement. Cracks range in width from hairline up to about ¾”, with slight vertical offset generally too small to be measured individually but adding up to about two inches across the damaged zone in some cases. Areas of cracking are mostly restricted to the westernmost (downhill) few feet of the roadway, although some areas extend to the approximate centerline of the roadway. The areas of cracking are variable in length, but up to about 70 feet long in the roadway direction.

3.2 SUBSURFACE CONDITIONS

We completed eleven machine-drilled borings and four hand auger explorations within the Project area. Machine-drilled borings are designated B-1 through B-11 and were completed to depths between 11.5 feet and 51.5 feet bgs. Borings were completed using hollow-stem auger methods, except B-6 which used mud rotary methods. Hand auger explorations were completed to between 6 and 10 feet bgs and are designated HA-1 through HA-4. Additionally, we completed four drive probe soundings to evaluate subsurface conditions to a depth of 10 feet, designated DP-1 to DP-4. The approximate locations of our explorations are shown on Figure 2. Explorations were completed between November 12th and 15th, 2024. Descriptions and logs of our subsurface explorations are included in Appendix A.

Our site explorations generally encountered native eolian sands to 51.5 feet bgs, the maximum depth of exploration. The native sands were generally overlain by roadway gravels and variable depths of gravelly and sandy fill where in roadway areas, or by a thin layer of organic material where outside of roadway areas. We completed interpretive cross sections at the boring locations shown on Figure 3. Our interpretation of subsurface conditions at each cross section are provided in Figures 4A through 4G. The geologic units we encountered are described in more detail below.

3.2.1 ROADWAY ASPHALT AND FILL

We encountered well-graded roadway gravel and/or sand fill in all of our borings. The gravel fill extended from the ground surface to variable depths of up to 2.5 feet bgs, but generally ranged in thickness from 0.5 to 2 feet. In all borings except B-1, B-2, and B-7, the gravel was overlain by approximately 2 inches of asphaltic concrete (AC) pavement. The gravels generally contained clasts measuring ¾-inch to 2 inches in diameter with varying amounts of sand and silt. Two sieve gradation tests were completed near the base of the fill which determined the fill to consist of predominately fine sand, with 5 to 42 percent gravel and 11 to 7 percent fines (B-10 S1 and B-6 S1, respectively). In some borings, sand without gravel was interpreted as fill, but the similarity of the native and fill sand made distinguishing between the two uncertain.

No laboratory testing was completed on the Roadway Asphalt and Fill.

3.2.2 Eolian Sands

Below the Roadway Asphalt and Gravels, we encountered orange to gray sands. These sands were poorly graded and fine-grained in size. They were generally uniform apart from color changes and occasional thin beds containing small amounts of organic material. The sand ranged from very loose to dense, with N-values of 2 to 65 bpf. Higher blow counts, those above about 25, were often, but not always at depths of about 30 feet bgs or greater.

Laboratory testing on samples of the eolian sands found moisture contents ranging from 1 to 25 percent. Most sands encountered were dry to moist, but samples retrieved during mud rotary drilling (Boring B-6) and/or recovered from below the water table were moist to wet, with measured moisture contents of up to 25 percent. Fines content tests measured 2 to 3 percent fines in three samples from B2 and B7.

3.2.3 Groundwater

We encountered groundwater in Boring B-5 at 38 feet bgs. We did not encounter groundwater in any of our other borings (depths of between 11.5 and 31.5 feet bgs). Groundwater could not be confirmed in Boring B-6, which extended to 51.5 feet bgs, due to the mud rotary drilling method used, but is presumed to occur at a similar depth as B-5.

Although groundwater was encountered at the depth and location noted above, groundwater conditions vary temporally due to seasons, precipitation, development and other factors. Perched (transient) groundwater could be encountered anywhere within the Project area during periods of heavy or prolonged precipitation, but the relatively clean sands throughout the Project area, suggest perched groundwater will not be common.

4.0 EVALUATION

Our background review and subsurface explorations found the primary geotechnical factors affecting the Project are the extensive loose dry eolian sand, the stability of site slopes along Classic Street, and seismic hazards overall. Based on our analysis, retaining walls as proposed will need to be designed to stabilize the roadway in general, currently failing areas in particular, and to support the widened roadway sections. These key geotechnical factors affecting the Project and geotechnical design of retaining walls are further evaluated in the following sections.

4.1 SLOPE STABILITY

We completed numerical slope stability analyses (SSA) at representative sections of Classic Street where indications of road instability are visible (arcuate cracking as noted above) and significant grading is proposed. The locations of our SSA analyses are shown on Figure 2, as Sections A, C, and E. The SSA were completed using the two-dimensional commercial software SLIDE by RocScience. SLIDE uses two-dimensional limit equilibrium methods to analyze slope stability by determining a factory of safety (FS) against slope instability. The FS against slope instability can be generalized as the ratio of forces resisting slope movement (soil strength, soil mass, etc.) to forces driving slope movement (gravity, earth pressure, etc.). A FS equal to or less than 1.0 indicates a condition when the available soil shear resistance decreases below the shear stresses required to maintain stability of the slope and the slope will theoretically fail. FS above 1.0 indicates the slope is stable with increasing FS indicating increasing stability. The program also predicts the location and geometry of "critical slip surfaces." Critical slip surfaces are the zones with the lowest FS. Our SSA was completed using the Spencer and Morgenstern-Price Methods, which both satisfy moment and force equilibrium. The lowest calculated FS from these two search methods is reported.

4.1.1 Cases Analyzed

The existing and proposed surface geometry of our models was developed from data provided by Windsor Engineers. As noted above, we analyzed three of our eleven boring locations: A-A' (B-10), C-C' (B-5), and E-E' (B-11), which approximately correspond to Stations 19+00, 15+50, and 12+00, respectively. The locations of the cross sections are illustrated on Figure 3. Our subsurface interpretations were based on the findings of our borings and laboratory testing at the cross sections analyzed. We estimated soil properties under existing static conditions by back-analysis at the analyzed locations. Back-analyzed conditions were developed by iteratively varying soil properties until achieving a FS of approximately 1.0 with failure surface locations similar to those observed in the field (the extent of pavement cracking). This method provides soil shear strength (average) in their current conditions to be used in analyses. Using these properties, we then analyzed the following scenarios:

- Existing static and seismic conditions.
- Developed conditions with retaining walls at the locations proposed by Windsor Engineers, and
    - Designed to meet a static FS of 1.25, with the resulting seismic FS calculated (not meting ODOT requirements).
    - Designed to meet both a static FS of 1.25 and seismic FS of 1.0 (meeting ODOT requirements).

A traffic surcharge was included within the roadway area as indicated by Windsor Engineers which was modeled as 250 pounds per square feet (psf) across the full traffic lanes.

Our seismic analyses utilized a horizontal acceleration of 0.254g, based on a peak ground acceleration of 0.5081 for the 975-year event, per Section 4.2.2.

Although not presented in this report, we evaluated both gravity and mechanically stabilized earth (MSE) walls. Our analyses found that the walls had different height and embedment requirements to meet the same required FS, but the differences were not large. Due to the additional excavation needed for placement of geogrid elements for the MSE walls and the resulting undermining of the slope (unless shored), we opined that the gravity wall was preferred for the Project and only present those results in our tables and in Appendix B. We also note that at some wall height/depth, a pile-wall may become preferable from a constructability and cost standpoint.

Other methods to stabilize the slopes were considered but not evaluated in detail. Buttresses require slopes that would not fit within the City ROW, groundwater is too low for drainage to be an option, and concrete cantilever walls would be more expensive and still require large excavations. So gravity walls were used in our analyses. Cantilever and/or tie-back pile walls are generally expected to be less cost effective, too, but at taller heights may become the preferred wall type as noted above.

4.1.2 Design Parameters

Based on the back analysis described above and on laboratory testing completed for this Project, we developed the soil properties for soil units used in the analyses, as summarized in Table 2. We note that the laboratory direct shear test completed for the Project (Appendix A) measured a significantly higher strength than the back-calculated values. Because the back-calculated values were consistent across the cross sections and they estimate average values (not at a single location like a test sample), we used the back-calculated, not laboratory-derived, strength.

Table 2 – Stability Soil Properties For All Cross Sections

Soil Unit Description	Material Color in SLIDE	Total Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Native sand, loose		110	26	0
Road Fill		125	40	0
Concrete Blocks		150	Infinite Strength	-

4.1.3 Stability Analyses Results

Results from our SSA are summarized in Tables 3a through 3c. Graphical results are included in Appendix B.

For analysis of slope mitigation options, a global minimum FS of 1.25 was used under static loading conditions for roadway embankment sideslopes per ODOT GDM, Table 7.3. Table 7.3 recommends FS of 1.25 for roadway sideslopes not supporting structures and for landslide remediation, FS of 1.3 for slopes adjacent to but not supporting structures, and 1.5 for slopes supporting structures. Since mitigation structures will support the slopes (not vice-versa) and are designed to stabilize the slopes, it is our opinion that 1.25 is an appropriate FS for static design. If an FS higher than 1.25 is preferred, we can provide additional design information.

For seismic conditions, FS of 1.1 to as low as 1.0 are the minimum required by ODOT, depending on acceptable ground deformations at the site (ODOT GDM, Section 13.5.3.1). To design to ODOT seismic requirements is expected to increase mitigation costs substantially. Due to the cost and because the roadway is a City-owned road so not subject to ODOT jurisdiction, we understand that ODOT requirements will generally be considered but may not be implemented. We, therefore, provide the results for mitigation configurations to meet a seismic FS of 1.0, as well as the resulting seismic FS under the minimum static design requirements (1.25) for consideration by the City and design team. Under each scenario in Tables 3a through 3c, we note the wall heights needed to meet the design FS, for consideration in estimating the additional cost.

Based on the results in Tables 3a through 3c, we note the following:

- Stabilization of the roadway slopes utilizing gravity retaining walls meet the design FS (are stable) under static conditions (FS of 1.25 or greater) with the wall heights noted in the tables.

- Under seismic conditions, stabilization using the same wall heights do not achieve a minimum FS of 1.0.

- To achieve minimum FS of 1.0 for seismic conditions, the MSE walls would have to be increased in height by approximately 25 to 30 percent, as noted in the tables.

Table 3a – Cross Section A-A' Stability Factor of Safety Summary

Figure	Case	Global Minimum FS	Wall Height* (feet)
B-1	A-A' – Back Analysis (Existing Conditions) – Static	1.64	NA
B-2	A-A' – Back Analysis (Existing Conditions) – Seismic	0.83	NA
B-3	A-A' – Proposed with Retaining Wall – Static	1.49	10
B-4	A-A' – Proposed with Retaining Wall – Seismic	0.92	10
B-5	A-A' – Proposed with Retaining Wall – Seismic FS 1.0	1.16	12.5

*Wall heights include exposed and embedded portions of wall and are approximate

Table 3b – Cross Section C-C' Stability Factor of Safety Summary

Figure	Case	Global Minimum FS	Wall Height* (feet)
B-6	C-C' – Back Analysis (Existing Conditions) – Static	1.03	NA
B-7	C-C' – Back Analysis (Existing Conditions) – Seismic	0.69	NA
B-8	C-C' – Proposed with Retaining Wall – Static	1.25	22.5
B-9	C-C' – Proposed with Retaining Wall – Seismic	0.82	22.5
B-10	C-C' – Proposed with Retaining Wall – Seismic FS 1.0	1.09	30

*Wall heights include exposed and embedded portions of wall and are approximate

Table 3c – Cross Section E-E' Stability Factor of Safety Summary

Figure	Case	Global Minimum FS	Wall Height* (feet)
B-11	E-E' – Back Analysis (Existing Conditions) – Static	0.94	NA
B-12	E-E' – Back Analysis (Existing Conditions) – Seismic	0.53	NA
B-13	E-E' – Proposed with Retaining Wall – Static	1.27	30
B-14	E-E' – Proposed with Retaining Wall – Seismic	0.82	30
B-15	E-E' – Proposed with Retaining Wall – Seismic FS 1.0	1.18	40

*Wall heights include exposed and embedded portions of wall and are approximate

We also analyzed the stability of the slope above Classic Street under seismic conditions. Our analysis shows that the slope east of Classic Street is prone to failure if not mitigated. Table 4 summarizes the results.

Table 4 – Global Stability FS Summary

Figure	Case	Global Minimum FS
B-16	Cross Section A-A' – Cut Slope Seismic	0.70
B-17	Cross Section C-C'– Cut Slope Seismic	0.73
B-18	Cross Section E-E'– Cut Slope Seismic	0.67

As noted in Table 4, the cutslope above the roadway is not stable under seismic conditions. Under the design seismic event the cutslope would be expected to fail above the road, and would likely deposit significant debris onto the roadway surface.

4.2 SEISMIC HAZARDS

The Project site is in a seismically active area. In this section, we describe seismic sources at the site, identify the seismic site class, provide seismic response spectra, and outline our interpretation of other seismic hazards at the site.

4.2.1 Seismic Sources

The Project site is in a seismically active area. The seismicity of the region is controlled 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: interface, intraslab, and crustal earthquakes, as described below.

Interface Seismic Sources – Subduction zones are typically characterized by interactions between the oceanic Juan de Fuca Plate and the continental North American Plate. As the oceanic plate subducts beneath the continental plate, the two lock together. As they lock together, stresses build in the overlying continental plate. When the stresses become too large, the plate can rupture resulting in an interface earthquake. An example of an interface earthquake is the moment magnitude 9.0 (M9.0) event which occurred in 2011 in Tohoku, Japan. Interface earthquakes are some of the largest magnitude and most destructive earthquakes recorded across the globe.

Intraslab Seismic Sources – Intraslab earthquakes originate from a deeper zone of seismicity that is associated with bending and breaking of the subducting oceanic plate. Intraslab earthquakes occur at depths of 40 to 70 kilometers (km) and can produce earthquakes with magnitudes up to and greater than magnitude M7.0. An example of an intraslab earthquake is the 2001 M7.0 Nisqually earthquake which occurred in west-central Washington.

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. Based on our review of available geologic maps (through DOGAMI HazVu), the closest mapped active fault to the site is approximately 10 miles to the south as described in section 2.2.2.

Details of these sources and their contributions to seismic hazard at the Project site are provided below.

4.2.2 Seismic Shaking

We evaluated potential seismic shaking at the site in accordance with the ODOT GDM and AASHTO based on 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).

We evaluated potential seismic shaking at the site using 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 0.4369g based on the ODOT, 2016 maps. 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

(As) is determined by applying a site class factor to the PGA noted above and is presented in Section 4.2.3. Refer to Section 4.2.4 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 32 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.

4.2.3 Seismic Site Class

The "site class" is a classification used by the 2022 Oregon Structural Specialty Code (OSSC) and by ODOT to quantify ground motion amplification. The classification is based on the properties of the upper 100 feet of the soil and bedrock materials at a site.

The deepest exploration performed at the site was approximately 51.5 feet bgs. The SPT N-value obtained at the bottom of this exploration was extrapolated down to 100 feet in order to obtain a site class designation. The weighted average N-values in the upper 100 feet of this boring were 19 blows per foot (bpf). As a result, we consider **Site Class D** to be an appropriate designation for the Project area.

However, we note this site class designation does not consider potential liquefaction of site soils, as discussed in Section 4.2.5.

4.2.4 Design Response Spectrum

We obtained seismic design parameters for the 975-year AASHTO design event (AASHTO, 2020) at Latitude 45.715399 and Longitude -123.929722. The parameters provided in Table 1 were developed using the ODOT ARS Spreadsheet (ODOT, V.2014.16). The values provided in Table 5 are considered generally appropriate for AASHTO and ODOT code-based seismic design, except for liquefaction, as noted above.

Table 5 - Seismic Design Parameters for 975-year Event

Parameter	Value
Site Class	D
Mapped Spectral Response Acceleration (Short Period), S_S	0.9041
Mapped Spectral Response Acceleration (1-Second Period), S_1	0.3743
Peak Ground Acceleration Coefficient, F_{pga}	1.1631
Short Period Spectral Acceleration Coefficient, F_a	1.1383
Long Period Spectral Acceleration Coefficient, F_v	1.9257
A_s (F_{pga} x PGA)	0.5081
Spectral Response Acceleration (Short Period), S_{DS}	1.0292
Spectral Response Acceleration (1-Second Period), S_{D1}	0.7208
Peak Ground Acceleration (PGA)	0.4369

4.2.5 Liquefaction Hazards

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 (gravels and sands), 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. Although the loose to medium dense sands at the site are subject to liquefaction where saturated, due to the depth of groundwater at the site (38 feet bgs, where encountered), the potential for soil liquefaction to affect the Project area is very low.

4.2.6 Other Seismic Hazards

4.2.6.1 Surface Fault Rupture

As noted previously, the nearest mapped active fault is approximately 10 miles south of the Project site. Therefore, we consider the hazard from ground surface rupture on mapped active faults to be relatively low. Unmapped or inactive faults may still exist that could increase the risk of ground fault rupture at the site.

4.2.6.2 Tsunami and Seiche

The proposed alignment is generally outside of the tsunami hazard area, but on its north end (from about the intersection of Classic St and Dorcas Ln to about the intersection of Classic St and Jackson Way) is within the local tsunami evacuation zone. As a result, tsunami hazards are likely to impact surface structures within the north end of the alignment if a tsunami occurs. The potential damage to buried structures, such as pipelines, is judged to remain low, however, as scour and erosion from tsunamis are not likely to reach them.

4.2.6.3 Seismic Subsidence or Uplift

Given the proximity of the site to the coastline, it is likely that the site will experience considerable coseismic subsidence associated with a rupture on the CSZ. Based on mapping by DOGAMI (Madin and Burns, 2013), between 3 and 4 feet of subsidence is anticipated following the design subduction zone earthquake. Generally, such subsidence is expected to be a widespread areal event which is not likely to have a significant effect on the alignment as differential displacement would be minimal.

4.2.6.4 Earthquake-Induced Landsliding

As described in Section 4.1.3 of this report, the steep slopes along Classic Sreet between Dorcas Lane and Highlands Drive will undergo earthquake-induced landsliding within this portion of the alignment. Outside of this area of steep slopes, the potential for earthquake-induced landsliding is low as slopes are generally flat.

4.2.6.5 Earthquake-Induced Settlement

It is well-known that seismically induced settlement of sand soils occur, even absent liquefaction (ODOT GDM, Section 13.5.4). We estimated sand settlement at the site and found that up to several inches of settlement is possible following a design earthquake.

4.3 RETAINING WALLS

We evaluated different methods to stabilize the slopes and to support the roadway along Classic Street, as discussed in Section 4.1.1. We determined that retaining walls are needed for this purpose.

We considered both gravity and MSE walls, but did not further consider concrete cantilever or pile walls for the reasons discussed in Section 4.1.1. For our slope stability analyses we determined the configurations (heights/embedments) needed to achieve global stability under static and seismic conditions as detailed in Section 4.1. We found that there was little difference in heights between the two wall types, however, MSE walls likely require more excavation and potential undermining of the roadway and eastern slope, compared to gravity walls. Therefore, we completed our slope stability analyses with gravity walls as described in Section 4.1. Wall heights/embedments necessary to meet design FS are noted in Tables 3a through 3c.

Final wall design will be completed by the Contractor and our recommendations for design of the walls are included in Section 6.2 of this report.

4.4 INFILTRATION TESTING

We completed two infiltration tests, IT-1 and IT-2, on each side of the intersection of Classic and Necarney City Road. The tests were completed on November 14, 2024, at the approximate locations shown on Figure 2. The tests were completed in general accordance with the encased falling head test in general accordance with the US Bureau of Reclamation (USBOR), as described in Attachment A of this report. We measured the following results in our infiltration tests:

Table 6. Field-Measured Infiltration Rates

Test #	Unfactored Rate Max/Min (in/hr)	Notes
IT-1	52/39	West side of Classic St
IT-2	85/67	East side of Classic St

As indicated in Table 6, the measured field (unfactored) minimum infiltration rate varies from 67 to 39 inches per hour with the slowest of the two measurements in IT-1 and IT-2 averaging 53 inches per hour. Given the depth of the water table in the area and consistent occurrence of sands within the area that are similar to those at the test locations, we anticipate an unfactored infiltration rate of 53 inches per hour is reasonable for the locations where testing was completed. Appropriate factors should be applied to this rate in design.

5.0 CONCLUSIONS

Based on our explorations, testing, and analyses, it is our opinion that the proposed Project is feasible from a geotechnical perspective, provided the recommendations in this report are included in design and construction. We offer the following general summary of our conclusions:

- Soils at the site are loose sands within anticipated excavation depths and generally all depths of interest.
- Groundwater is several tens of feet below ground surface and not anticipated to have an effect on the Project.

- Pavement cracking along the edge of Classic Street is interpreted as due to ongoing sliding and creep of loose sands beneath the roadway.
- Gravity or MSE retaining walls are recommended to stabilize the downslope edge of Classic Street, with gravity walls preferred. Final design of the walls will be completed by the Contractor and should consider the information provided in this report, meet a static FS of 1.25, and meet a seismic FS as determined by the City.
- The loose sand soils will not hold steep slope angles or have stable trench walls at significant excavation depths. They will also be prone to raveling. Temporary and permanent excavations should consider the loose nature of the soils and take appropriate measures to protect structures and avoid excessive overexcavation from slope raveling and collapse.
- Excavations into steep slopes below the houses adjacent Classic Street should be avoided due to the potential for upslope raveling and damage to house foundations. If such excavations are planned, we should be contacted to provide recommendations and review grading plans.
- Additional measures to protect upslopes homes from construction-related damage should be considered, including a pre-construction survey and the use of non-vibratory compaction for roadway subgrades and base rock.
- The site is conducive to on-site stormwater infiltration per the recommendations in this report.
- Pavement design, based on the traffic data provided by the City, should follow the recommendations in this report.
- On-site soils are suitable for use as structural fill.
- Subsurface conditions will make shallow trenchless methods difficult to complete, due to mud loss and heave at the surface. However, we understand that local contractors have been able to successfully advance utilities in the site soils with specific mud mixtures. Completion of trenchless utilities may require reliance on local contractors experienced in such soils.

Our geotechnical recommendations for the Project, which address the above, are provided in the following sections.

6.0 RECOMMENDATIONS

Our Earthworks and Retaining Wall Recommendations for the Project are provided in the following sections.

6.1 EARTHWORKS RECOMMENDATIONS

6.1.1 Site Preparation

Site preparation will depend on final selection of a pavement section. Where pavements remain in place, no significant site preparation is anticipated. However, where pavements are to be removed, they should be removed to the full depth they occur. The underlying base rock can generally be left in place, unless removal is necessary to reach site grades, or the rock is contaminated. Removed AC and base rock can be stockpiled and re-used later as structural fill as described later in this report.

Where retaining walls are to be constructed, site preparation should also include clearing of trees, grubbing stumps and other vegetation, and stripping any organics and duff within structural and work areas. We estimate that stripping will generally be less than 6 inches deep. Cleared, stripped, and grubbed materials should be hauled off-site and properly disposed of.

Any utilities to be abandoned within the Project area should be fully removed or grouted full if left in place. Areas disturbed by their removal should be repaired as recommended elsewhere in this report.

The exposed subgrade should be evaluated after site preparation activities are complete. Evaluation should be completed by proofrolling the subgrade with a fully-loaded dump truck or similar heavy rubber-tired construction equipment to identify remaining soft, loose or unsuitable areas. The proofroll should be conducted prior to placing any other fill. The proofrolling should be observed by a member of Pali Consulting's staff who should evaluate the suitability of the subgrade and identify any areas of yielding that are indicative of soft or loose soil. If soft or loose zones are identified during proofrolling, these areas should be excavated to the extent indicated by the engineer and replaced with structural fill.

6.1.2 Wet Weather Construction

The sandy soils at the site are not very susceptible to wet conditions, except during periods of high precipitation or when saturated.

However, it is good practice to schedule earthwork for drier summer months, if possible. If earthwork is scheduled for the wet season or significant precipitation occurs during construction, the contractor should be prepared to employ wet weather measures to minimize disturbance to the subgrade from construction traffic. Such measures might include:

- Constructing a temporary working pad of 12 to 24 inches or more of crushed rock over a geotextile fabric,
- Using tracked equipment and smooth-edge buckets to minimize subgrade disturbance,
- Covering soil stockpiles or subgrade areas with plastic to prevent erosion and saturation,
- Protecting footing subgrades with four or more inches of lightly compacted crushed rock.
- Other measures as needed to protect structural areas of the site and structural materials.

Bearing soils that are disturbed during construction should be recompacted in place, if practical, or removed and replaced with structural fill.

6.1.3 Excavation

Site soils within expected excavation depths will generally consist of loose sand that is dry to moist. It is our opinion that conventional earthmoving equipment in proper working condition should be capable of making Project excavations in expected soil types. The earthwork contractor should be responsible to provide the equipment and procedures to excavate the site soils described in the exploration logs and text of this report.

6.1.4 Dewatering

Regional groundwater was encountered at over 30 feet deep, so is not expected to occur within anticipated excavation depths. During periods of high precipitation, perched groundwater may occur within planned excavation depths, but given the very uniform well-drained sandy soils at the site, such perched conditions are unlikely to be persistent for long periods of time. In addition to perched groundwater, surface water inflow to the excavations during the wet season could be problematic, especially adjacent to areas where AC pavements remain. Provisions for temporary ground and surface water control should be the responsibility of the contractor to select the means and methods best suited to the schedule and their equipment.

6.1.5 Excavation Stability

Trench sidewalls throughout the Project will be prone to raveling and collapse at all depths. We recommend that all excavations be shored or laid back. All trench excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. Site soils are expected to be OSHA Type C throughout the Project area, but the soil type should be confirmed by a "competent person" under the direction of the contractor in the field based on actual conditions encountered.

Trenches should not be excavated adjacent the toe of any slope below a line projected from the toe of the slope at a 3H:1V gradient, unless evaluated by qualified personnel.

While this report describes certain approaches to excavation and shoring, the contractor should be responsible for selecting and designing the specific methods, monitoring the excavations for safety, and providing shoring required to protect personnel and adjacent structural elements.

6.1.6 Permanent Cut and Fill Slopes

Permanent cut slopes in the loose native aeolian sands should not exceed 3H:1V. Fill slopes can be constructed at maximum gradients of 2H:1V, if completed per Section 6.1.7 of this report. Slopes that will be maintained by mowing or adjacent to surface water should be 3H:1V or flatter. Footings, access roads and pavements should be located at least 5 feet horizontally from any slope face. If steeper slopes or closer setbacks are necessary, we should be contacted to provide additional recommendations, and additional explorations may be necessary.

Slopes should be planted with appropriate vegetation as soon as possible after grading to provide protection against erosion. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

6.1.7 Structural Fill and Backfill

All fill associated with roadways, retaining walls, and slopes over 5H:1V should be considered structural fill for this Project.

Structural fill soils should be free of debris, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 4 inches, and other deleterious materials. The suitability of soil for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines in the soil matrix increases, the soil becomes increasingly more sensitive to small changes in moisture content and achieving the required degree of compaction becomes more difficult or impossible.

Recommendations for suitable fill are provided in the following sections.

6.1.7.1 Pipe Bedding and Pipe Zone Material

Utility trench backfill for pipe bedding and in the pipe zone should consist of well-graded granular material with a maximum particle size of 3/4-inch and less than 10 percent fines. The pipe bedding and pipe zone material should meet the pipe manufacturer's recommendations, as well, including placement of the bedding and pipe zone material so that the pipe is evenly supported and backfilled.

6.1.7.2 Trench Backfill

Backfill above the pipe zone should consist of materials suitable for the overlying use of the area. Our recommendations for backfill within and outside of roadway areas follow, separately:

6.1.7.2.1 Backfill in Roadway Areas

Within roadway areas we recommend that imported granular material be used as backfill. The material should be pit or quarry run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in the Oregon Department of Transportation (ODOT) Standard Specifications for Construction (SSC) 00330.14 – Selected Granular Backfill or SSC 00330.15 – Selected Stone Backfill. The imported granular material should also be angular, fairly-well graded between coarse and fine material, have less than 10 percent by dry weight passing the U.S. Standard No. 200 Sieve, and have at least two mechanically fractured faces. During dry weather, the fines content may be increased to a maximum of 20 percent.

6.1.7.2.2 Backfill in Non-roadway Areas

Outside of roadway areas where no surcharge loads or traffic will occur, on-site granular material (sand) can be used provided the material meets the general requirements for structural fill. If the use of on-site soil as structural fill is problematic, imported granular material such as that specified for roadway areas or Imported Structural Fill can be used.

6.1.7.3 Roadway Base Rock

Imported granular material used as aggregate base (base rock) in roadway areas 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 of SSC 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 based on the minus 3/4-inch fraction and have at least two mechanically fractured faces. The aggregate base should have a maximum particle size of 1 to 1-1/2 inch, depending on future performance preference. Smaller aggregate material generally has more favorable drivability characteristics but shorter lifespan, while larger aggregates have the opposite characteristics where AC will not be placed over the base rock.

6.1.7.4 Haul Road Rock

If haul roads are constructed, rock to construct haul roads should consist of crushed rock that is well-graded between coarse and fine particle sizes, contains no unsuitable materials or particles larger than 4 inches, and has less than 5 percent by weight passing the U.S. Standard No. 200 sieve. It should be placed in a single lift, typically over a separation geotextile fabric, and compacted to a well-keyed state using a heavy non-vibratory roller.

6.1.7.5 Imported Select Structural Fill

Select imported granular material may be used as structural fill. The imported material should consist of pit or quarry run rock, crushed rock or crushed gravel and sand that is fairly well graded between coarse and fine sizes. It should have a maximum particle size of 4 inches and 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 the tables that follow.

6.1.7.6 Crushed Rock Fill

Crushed rock fill for aggregate base located under footings or other structures, should consist of imported clean, durable, crushed angular rock. Such rock should be well-graded, have a maximum particle size of 1½ inch, and have less than 5 percent passing the U.S. No. 200 Sieve. The material should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities as recommended in the tables that follow.

6.1.7.7 Drainage Rock

Rock for drainage purposes should consist of open-graded crushed granular rock with a maximum particle size of 1 ½ -inch and less than 2 percent passing the U.S. No. 200 Sieve (washed analysis). The material should be free of organic matter and other deleterious materials. Crushed rock of ¾- to 1½- gradation drain rock is suitable for this purpose. The drain rock should be nominally compacted to a well-keyed state unless specified otherwise.

6.1.8 Fill Placement and Compaction

Fill should be placed and compacted in accordance with the following guidelines.

- Place fill and backfill on a firm subgrade, in uniform horizontal lifts with a thickness appropriate for the material type and compaction equipment. Table 7 provides general guidance for lift thicknesses.
- 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 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 8.
- 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.

Table 7. Guidelines for Uncompacted Lift Thickness

Compaction Equipment	Guidelines for Uncompacted Lift Thickness (inches)		
	Native Soil	Granular and Crushed Rock (Maximum Particle Size < 1½")	Crushed Rock (Maximum Particle Size > 1½")
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 Equipment	12 – 16	18 – 24	12 – 16

Note:

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.

Table 8. Fill Compaction Criteria

Fill Type	Percent of Maximum Dry Density Determined in Accordance with ASTM D 1557		
	0 – 2 Feet Below Subgrade	>2 Feet Below Subgrade	Pipe Bedding and Pipe Zone
Pipe Bedding and Pipe Zone			90
Trench Roadway Backfill	95	92	----
Trench Non-roadway Backfill and Non-roadway Areas	88	88	----
Aggregate Base[1]	95	----	----
Nonstructural Zones	88	88	----

Notes:

1. Structural fill with more than 30 percent retained on the ¾-inch sieve should be compacted to a well-keyed dense state at near optimum moisture content and performance tested to evaluate compaction.

During structural fill placement and compaction, 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.

6.1.9 Surface Drainage

Surface runoff in unpaved areas can be controlled during construction by planning and grading practices. Surface drainage should be planned to promote drainage away from open trenches and excavations, slopes, and roadways. Such measures should be completed daily at the end of each shift. Design and implementation of such measures should be the responsibility of the Contractor.

6.1.10 Trenchless Methods

Trenchless methods are expected to be an allowed option for the Contractor to use in addition to open cut. Given the shallow depth of the utilities and loose nature of the aeolian sands that cover the Project area, trenchless methods could be subject to much mud loss and frac-outs. However, we understand that local contractors have developed methods that work well in the materials at the site. If allowed for the Project, we recommend the following related to the use of trenchless methods:

- The Contractor be responsible for the design and execution of trenchless construction methods.
- The Contractor submit a written plan of their proposed methods, including their experience in the area and with such methods, the equipment to be used, methods to prevent frac-outs and contain drilling fluids if frac-outs occur, and measures to protect existing utilities from damage due to their methods.

6.2 Retaining Walls

Retaining walls are necessary to support the west side of Classic Street. As presented in Section 4.1, Our SSA analyses found that gravity and MSE walls require similar heights/embedments to achieve similar design FS. Due to construction considerations, we opine that gravity walls will be most cost effective, however, provide preliminary recommendations for gravity and MSE walls below.

6.2.1 Concrete Gravity Walls

Concrete gravity walls constructed of Ecology Blocks©, or their equivalent, are suitable for the site. The walls should be designed per the manufacturer's recommendations and using the parameters in Table 9. These parameters are based on the following assumptions:

- Wall heights/embedments, per Tables 3a – 3c, are approximate and final designs will be per the results of slope stability analyses.
- Global stability should meet a minimum FS of 1.25 under static conditions. Seismic FS shall be as directed by the City, but not be less than the seismic FS under existing conditions.
- Internal stability shall meet the minimum FS of the OSSC and wall manufacturer.
- Wall and slope geometries shall be consistent with the design drawings.
- The walls will not be restrained against rotation when the backfill is placed.
- The backfill will consist of structural fill which extends behind the walls for a minimum distance equal to the wall height.
- Backfill within 2 feet of walls consists of free-draining granular materials.
- Hydrostatic pressures do not develop, and drainage will be provided behind walls.

Traffic or other surcharge loads should be appropriately accounted for in wall design. The blocks should be placed on a pad consisting of a minimum of 6 inches of compacted crushed rock. Backfill should be placed and compacted as recommended for structural fill in Section 6.1.7.

Table 9 – Retaining Wall Design Parameters for Gravity Walls

Parameter	Results/Units	Notes
At-rest earth pressure	60 pcf	Triangular load at 1/3H*
Seismic earth pressure increase	12.5H	Rectangular load at 1/2H
Active earth pressure	35 pcf	Triangular load at 1/3H
Passive earth pressure	0 pcf	Not considered due to slope
Backfill soil density	110/125 pcf	Native Sand/Granular rock backfill
Downward drag coefficient	0.4	Based on 2/3 phi
Vehicle load on backfill	2 feet equivalent fill above grade	Where within a distance from the wall = wall height or less

*H = wall height

6.2.2 MSE Walls

MSE walls supplied by various manufactures, are suitable for the site. The walls should be designed per the manufacturer's recommendations and using the parameters in Table 10. These parameters are based on the following assumptions:

- Wall heights/embedments, per Tables 3a through 3c, are approximate and final designs will be per the results of slope stability analyses.
- Global stability should meet a minimum FS of 1.25 under static conditions. Seismic FS shall be as directed by the City, but not be less than the seismic FS under existing conditions.
- Internal stability shall meet the minimum FS of the OSSC and wall manufacturer.
- Wall and slope geometries shall be consistent with the design drawings.
- Backfill within the reinforced zone will consist of granular structural fill meeting the

compaction requirements of Section 6.1.8 and the strength requirements in Table 10.

- Hydrostatic pressures do not develop, and drainage will be provided behind walls.

Traffic or other surcharge loads should be appropriately accounted for in wall design. The blocks should be placed on a pad consisting of a minimum of 6 inches of compacted crushed rock. Backfill should be placed and compacted as recommended for structural fill.

Passive pressures in front of the wall should be assumed zero for design purposes. Wall sections greater than 2½ feet in height or subject to surcharge loads (such as from slopes or traffic) should include reinforcing elements.

Many MSE walls are available as proprietary wall systems. If proprietary wall systems are used, the wall supplier is responsible to design the wall for adequate internal stability, i.e., pullout and yield of reinforcing elements and overturning. However, we recommend that proprietary wall system designs be reviewed by Pali Consulting to verify that design is consistent with material properties recommendations of this report.

Table 10. Recommended Design Parameters for Reinforced Soil Walls

Soil Properties	BACKFILL SOIL Compacted Structural Fill[1]	RETAINED SOIL Native	FOUNDATION BEARING SOIL Native
Unit Weight (pcf)	125	110	110
Friction Angle (degrees)	34	26	26
Cohesion (psf)	0	0	0
Allowable Bearing Pressure (psf)	N/A	N/A	1,500

Note:

1. Backfill soils should be properly compacted, imported granular soils, as described above in *Section 6.1*.

7.0 PAVEMENT RECOMMENDATIONS

Pavement design was completed using the AASHTO Guide for the Design of Pavement Structures (AASHTO, 1993). We considered both new pavement and rehabilitation of the existing pavement. We considered the following key constraints in evaluating suitable pavement methods:

1. A wall will be constructed along the west side of the existing roadway, requiring removal of much of the pavement along at least the west side,
2. Loose sands underly the roadway adjacent the areas to be excavated for roadway retaining walls, which may undergo raveling and undermining of pavements on the east side of the roadway if left in place,
3. Existing pavement damage is structural-related so would require full depth rehabilitation at a minimum,
4. Pavement data for design includes infrequent information at boring locations, not more continuous data from non-destructive testing (NDT), making reclamation strategies less reliable than preferred.

Based on the above constraints, we recommend the existing pavements along Classic Street be replaced with new pavement sections. Our recommendations for new pavements are provided in the following sections.

7.1 Assumptions and Design Parameters

We made the following assumptions regarding the design of the pavement:

- Construction occurs during a period of dry weather.
- The subgrade will consist of suitable sand fill or native sand that has been compacted to at least 92 percent of the maximum dry density per ASTM D-1557, and proofrolled as noted in Section 6.1.1 of this report.
- Equivalent single-axle loads (ESALs) were estimated using traffic studies completed by Mackenzie at the intersections of Classic Street with Laneda Street and Dorcas Lane. Traffic counts were only completed for two 2-hour peak periods at the respective intersections and included all traffic, buses and heavy trucks. Traffic on Classic Street at Laneda and Dorcas converted to daily (24-hour) counts are summarized below. The traffic counts were the maximum of the sum of Left, Right and Thru traffic through the intersections on Classic Street.
    - Classic @ Dorcas: 1326 cars, 120 trucks
    - Classic @ Laneda: 1152 cars, 24 trucks
- A 20-year design life was computed with equivalent single-axle loads (ESALs) and heavy truck traffic, from the above traffic which results in the following ESAL's:
    - Classic Avenue at Dorcas 2.358 x 10^6 ESAL's
    - Classic Avenue at Laneda 4.94 x 10^5 ESAL's
- A California Bearing Ratio (CBR) of 5 for recompacted fine sand soil subgrade that has been prepared in conformance with the recommendations of this report.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability and standard deviation of 85 percent and 0.45, respectively.
- Structural coefficients of 0.42 and 0.14 for the flexible asphalt and base rock layers, respectively.

Significant construction traffic should not be allowed on new pavements. If construction traffic is to be allowed on new pavements, an allowance for additional traffic will need to be made in the design pavement section.

As discussed elsewhere in this report, the near-surface site soils are fine sands that may be difficult to properly compact during periods of wet weather. Therefore, alternatives, such as thickened rock sections may be needed if construction will occur during wet weather. Thickened rock sections are described in the following section of this report.

7.2 Pavement Sections

Where the soil subgrade has been prepared as described in Section 6.1, and above, the pavement sections shown in Table 11 may be utilized.

Table 11. Pavement Sections with Compacted Subgrade

Pavement Designation	AC (inches)	Aggregate Base (inches)
Classic Street at Dorcas	4.0	14
Classic Street Laneda	4.0	8.0

If compaction of the subgrade cannot be attained during periods of wet weather, the aggregate base thicknesses listed in Table 10 can be increased by 6 inches to account for the decreased subgrade modulus. The subgrade should be at least medium dense and approved by Pali Consulting before placing the base rock.

7.3 Pavement Materials

AC pavements should consist of Level 2, 12.5-mm, dense hot mixed asphalt concrete according to OSS 00744 – Minor Hot Mixed Asphalt Concrete Pavement. The asphalt cement binder should be PG 64-22 Performance Grade Asphalt Cement. The AC should be placed in two lifts with a minimum lift thickness of 2 inches. The AC should be compacted to 91 percent of Rice Density of the mix, as determined in accordance with ASTM D 2041.

Imported granular material used as base aggregate (base rock) should meet the criteria specified in Section 6.1.7.3 of this report. The base aggregate should be compacted to not less than 95 percent of the maximum dry density as determined by ASTM D 1557.

7.4 Pavement Construction

Construction should be completed in general accordance with the Oregon Department of Transportation (ODOT) Standard Specifications for Construction (SSC) and the recommendations in *Section 6.0*. Construction traffic should not be allowed on new pavements. If construction traffic is to be allowed on newly constructed pavements, an allowance for additional traffic will need to be made in the design pavement section.

8.0 LIMITATIONS

Our evaluation was based on surface reconnaissance and limited subsurface explorations. Our report is intended to evaluate geotechnical conditions within the Project area and make recommendations for design of the Project. However, all development on slopes involves risks, only part of which can be mitigated through qualified geotechnical evaluation and 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.

The conclusions and recommendations contained within this report are professional opinions based on our evaluation of limited information and should not be construed as a warranty of slope performance. Soil conditions can differ during different seasons, from earth processes, storms, or other factors that occur after our work has been completed. Although we evaluated areas of anticipated instability, some locations may have been overlooked. If additional unstable areas are encountered, site conditions change, or significant time passes after our work is completed, we should be given an opportunity to review our work and provide additional input if we believe it to be warranted.

Within the limitations of scope, schedule, and budget, our services were 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.

9.0 REFERENCES

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10.0 CLOSING

We appreciate this opportunity to submit this report. If we may provide any additional information or clarification, please contact us.

Sincerely,

PALI CONSULTING INC.

REGISTERED PROFESSIONAL ENGINEER
52838
OREGON
JAN. 12, 1999
TIMOTHY W. BLACKWOOD
EXPIRES 12/31/2025

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TIMOTHY W. BLACKWOOD, PE, GE, CEG
President/Principal Engineer

Attachments:

- Figures 1 through 4
- Appendix A – Field Explorations and Laboratory Testing
- Appendix B – Slope Stability Analysis

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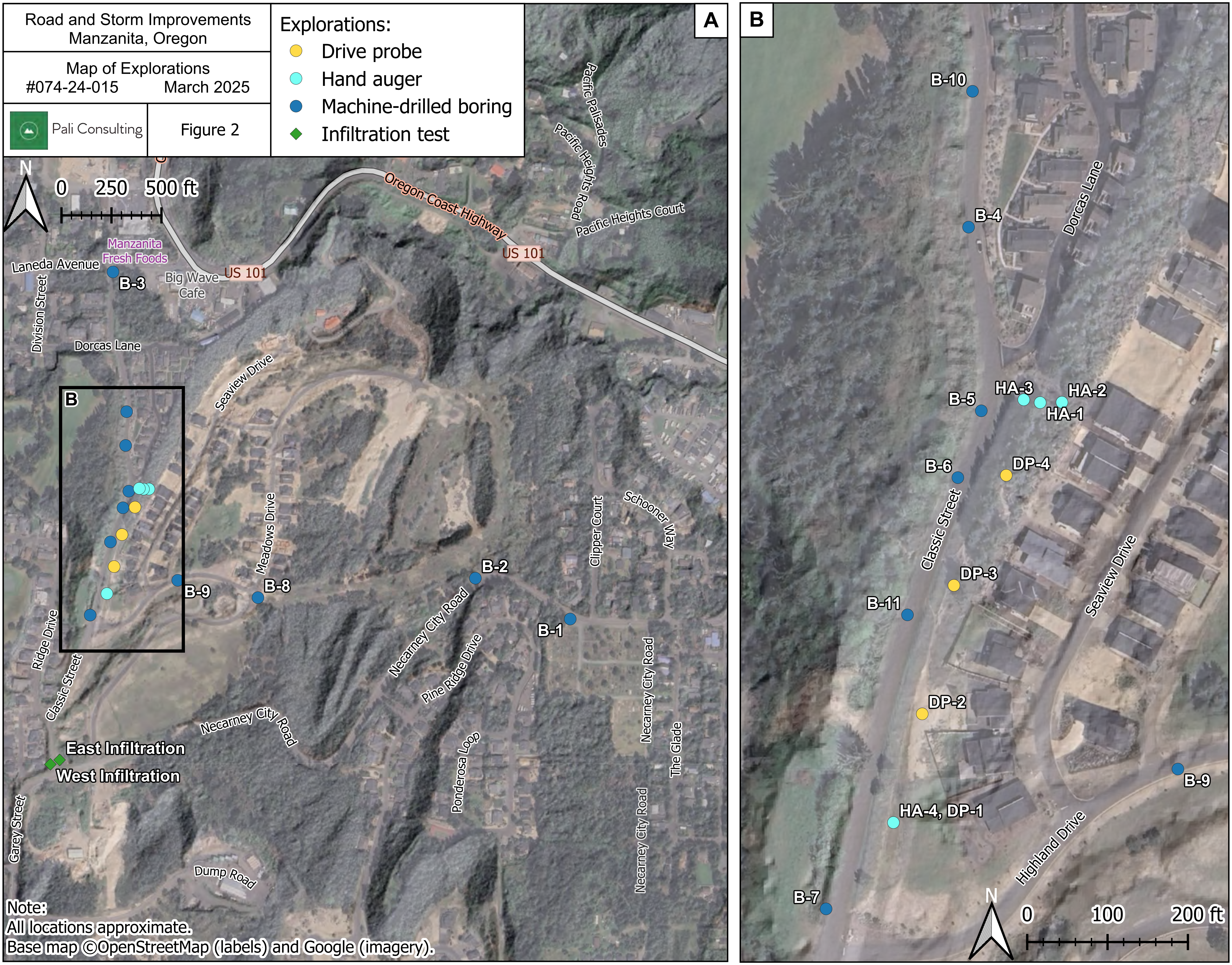
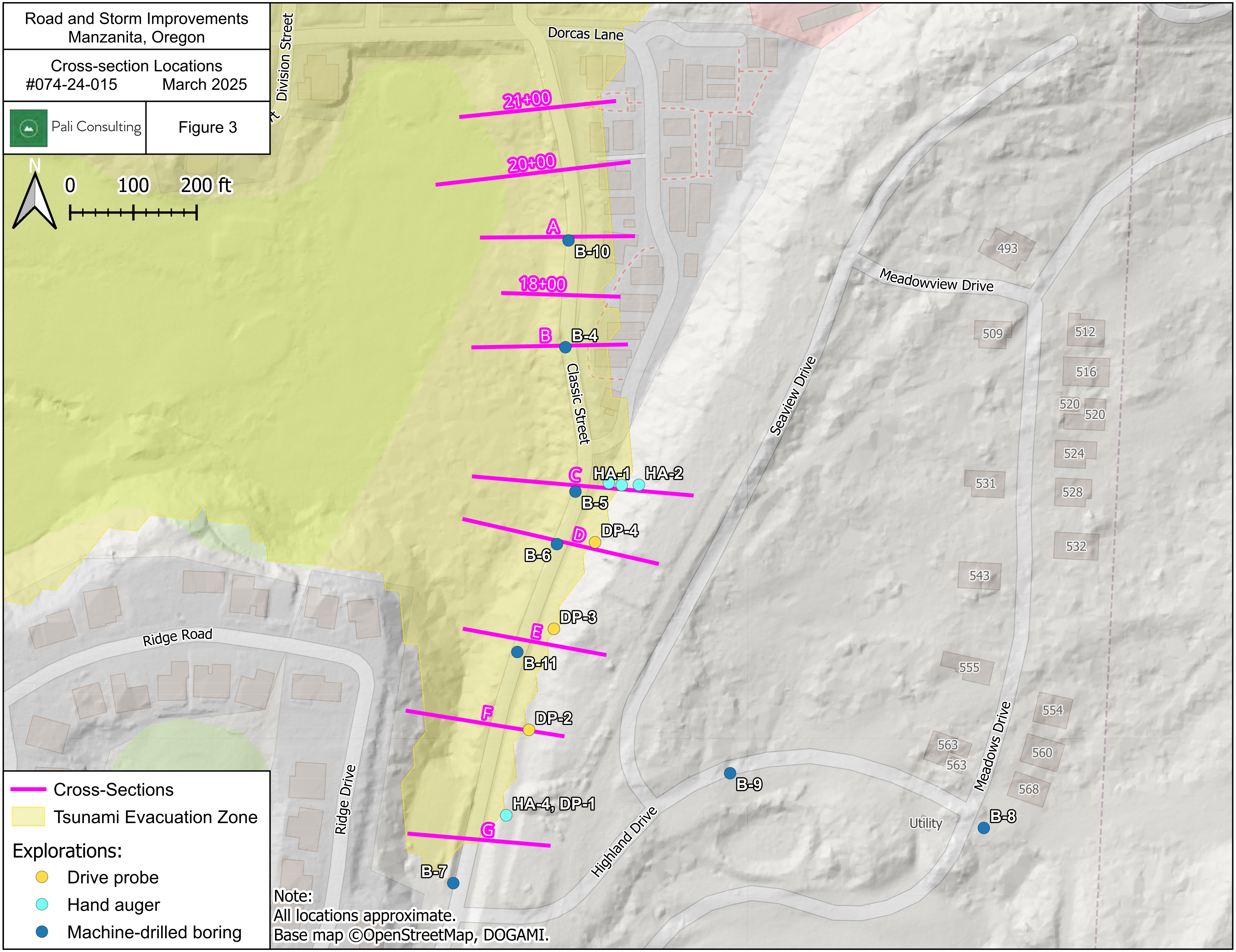
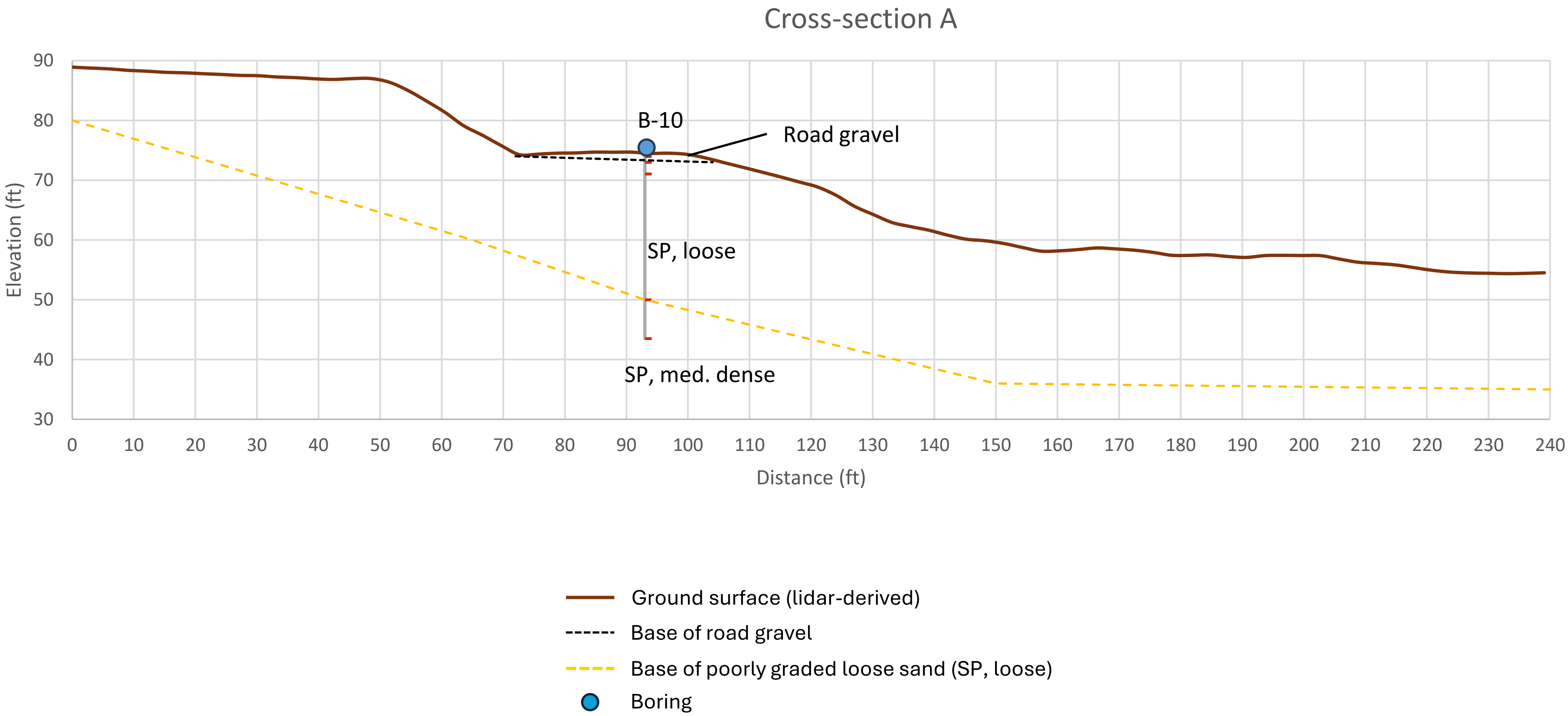
Road and Storm Improvements
Manzanita, Oregon
Vicinity Map
#074-24-015
March 2025
Pali Consulting
Figure 1
Project area
College Avenue
Nehalem Road
Cherry Street
Elm Street
Greenridge Street
Madrona Street
Poysky Avenue
Oak Street
Epoh Avenue
Ocean Road
Ocean Avenue
US 101
North Avenue
Neahkahnie Lake
2nd Street North
North 3rd Street
4th Street
Pine Avenue
Fir Avenue
Manzanita Avenue
Division Street
Oregon Coast Highway
Laurel Avenue
1st Avenue
Calvary Bible Church
Laneda Avenue
Merton Lane
Hallie Lane
Edmund Lane
Treasure Cove Lane
Beeswax Lane
Pacific Lane
South 3rd Street
South 4th Street
South 4th Place
Dorcas Lane
Beach Street
Ridge Road
Classic Street
Seaview Drive
Upland Road
Ridge Drive
Meadows Drive
Manzanita City Park
Necarney City Road
Manzanita Golf Course
Sunset Lane
Lakeview Drive
Fairway Court
Garey Street
Chinook Lane
Puffin Lane
Sandpiper Lane
Pelican Lane
5th Street
Sitka Lane
Pacific Palisades
Rex Champ Field
Alder Creek
Schollmeyer Road
Underhill Lane
Clipper Court
Schooner Way
American Legion Cemetery
Bayside Gardens
Corder Lane
Bay Place
Ernst Court
Evergreen Way
Fern Way
Pine Ridge Drive
The Glade
The Promenade
Bayside Gardens Road
Seamont Way
Ocean Way
Tohl Avenue
Pintail Avenue
Pinyon Drive
Lodgepole Drive
Neptune Way
Elk Meadows Park
N
Note:
All locations approximate.
Base map ©Google (imagery),
Open Street Map (labels)
0
0.5
1 mi



Road and Storm Improvements
Manzanita, Oregon
Map of Explorations
#074-24-015
March 2025
Pali Consulting
Figure 2
Explorations:
Drive probe
Hand auger
Machine-drilled boring
Infiltration test
A
N
0
250
500 ft
Oregon Coast Highway
US 101
Manzanita Fresh Foods
Big Wave Cafe
Laneda Avenue
Division Street
Dorcas Lane
Seaview Drive
Pacific Palisades
Pacific Heights Road
Pacific Heights Court
Meadows Drive
Clipper Court
Schooner Way
Necarney City Road
Pine Ridge Drive
Ponderosa Loop
The Glade
Ridge Drive
Classic Street
Garey Street
Dump Road
East Infiltration
West Infiltration
B-1
B-2
B-3
B-8
B-9
Note:
All locations approximate.
Base map ©OpenStreetMap (labels) and Google (imagery).
B
B-10
B-4
Dorcas Lane
B-5
HA-3
HA-2
HA-1
B-6
DP-4
Classic Street
Seaview Drive
DP-3
B-11
DP-2
B-9
HA-4, DP-1
Highland Drive
B-7
N
0
100
200 ft


Road and Storm Improvements
Manzanita, Oregon
Cross-section Locations
#074-24-015 March 2025
Pali Consulting
Figure 3
N
0 100 200 ft
Cross-Sections
Tsunami Evacuation Zone
Explorations:
Drive probe
Hand auger
Machine-drilled boring
Note:
All locations approximate.
Base map ©OpenStreetMap, DOGAMI.
Dorcas Lane
Division Street
Classic Street
Seaview Drive
Meadowview Drive
Meadows Drive
Highland Drive
Ridge Road
Ridge Drive
Utility
21+00
20+00
18+00
A
B
C
D
E
F
G
B-10
B-4
B-5
B-6
B-11
B-7
B-9
B-8
HA-1
HA-2
HA-4, DP-1
DP-4
DP-3
DP-2
493
509
512
516
520
520
524
531
528
532
543
555
554
563
563
560
568

Road and Storm Improvements Manzanita, OR
Cross-Section A-A' #074-24-015 March 2025
Pali Consulting | Figure 4A



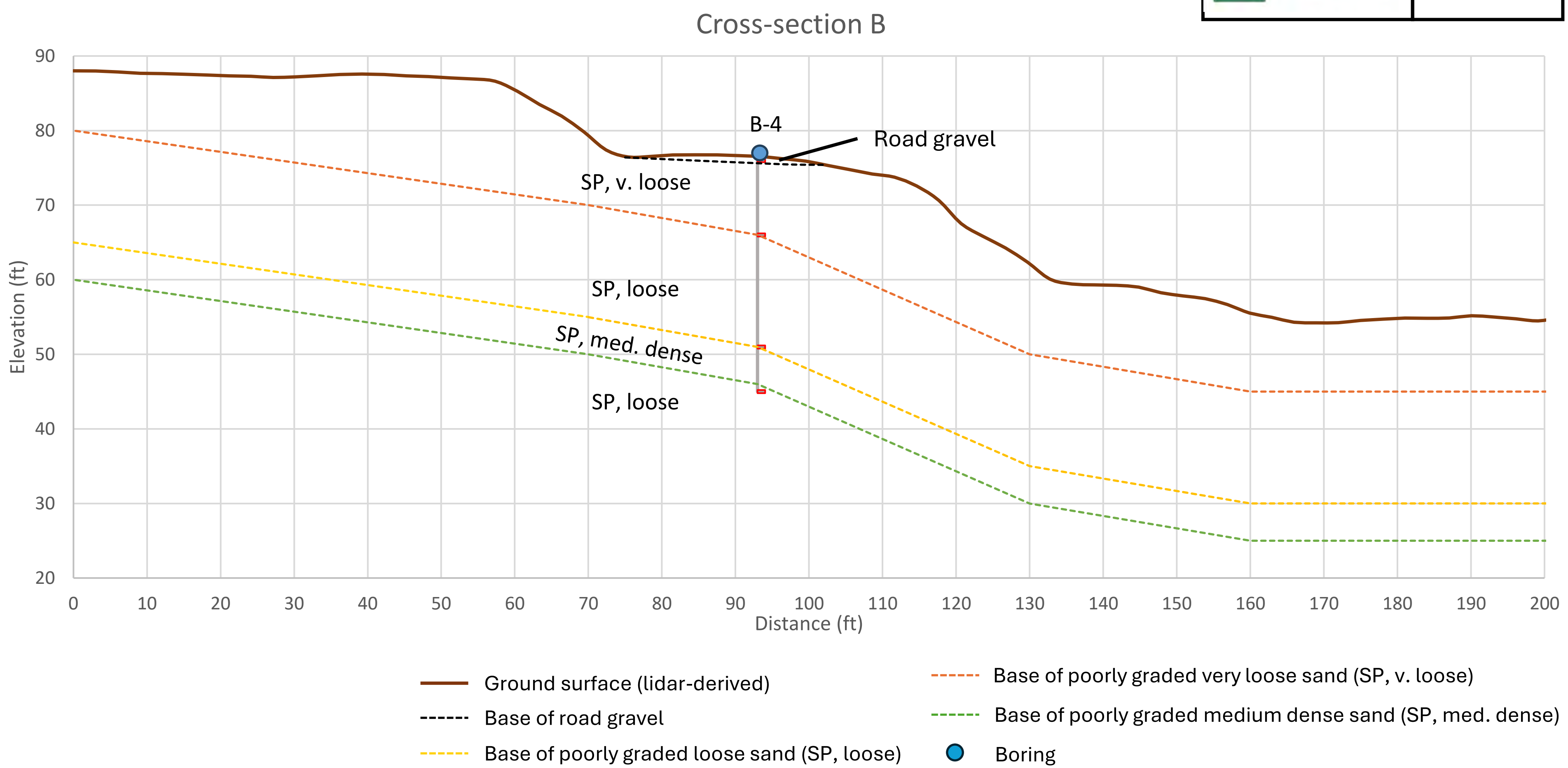
Notes:
No vertical exaggeration.
Looking south through site.

Road and Storm Improvements
Manzanita, OR

Cross-Section B-B’
#074-24-015 March 2025

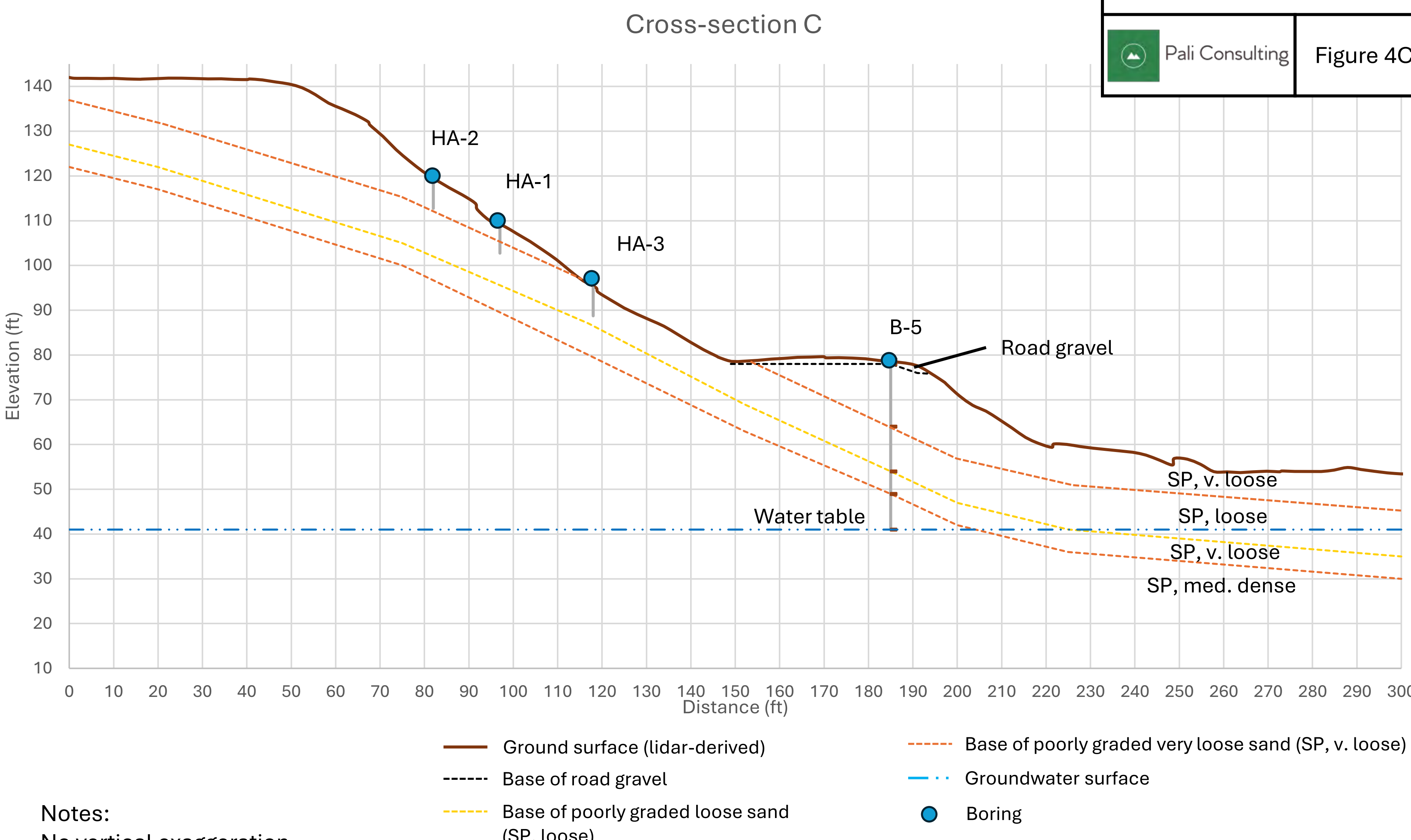
Pali Consulting

Figure 4B



Notes:
No vertical exaggeration.
Looking south through site.

Road and Storm Improvements Manzanita, OR
Cross-Section C-C’ #074-24-015 March 2025
Pali Consulting — Figure 4C



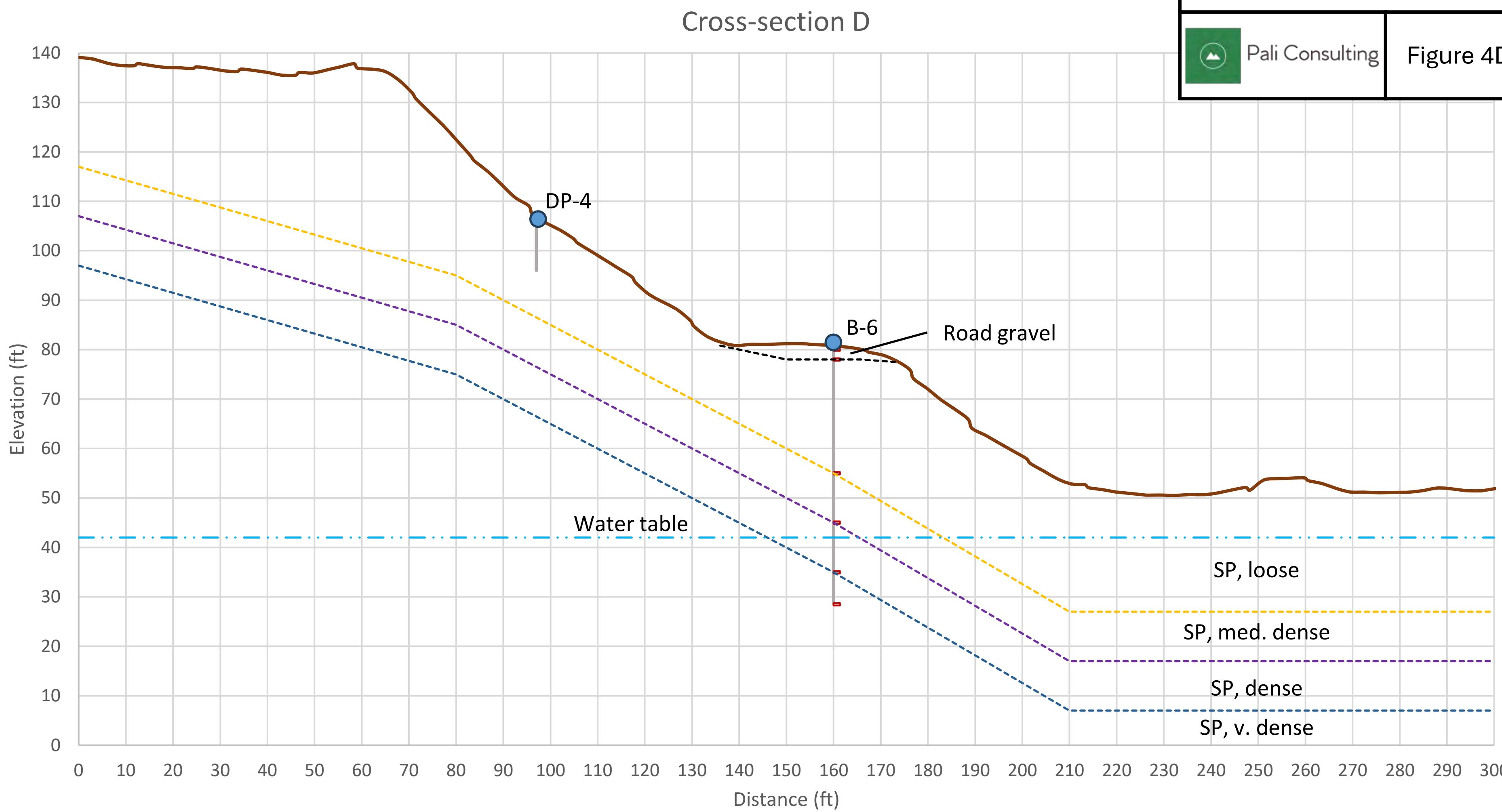
Notes:
No vertical exaggeration.
Looking south through site.

Road and Storm Improvements
Manzanita, OR

Cross-Section D-D’
#074-24-015 March 2025

Pali Consulting

Figure 4D



Ground surface (lidar-derived)
Base of road gravel
Base of poorly graded loose sand (SP, loose)
Boring
Base of poorly graded medium dense sand (SP, med. dense)
Base of poorly graded dense sand (SP, dense)
Groundwater surface

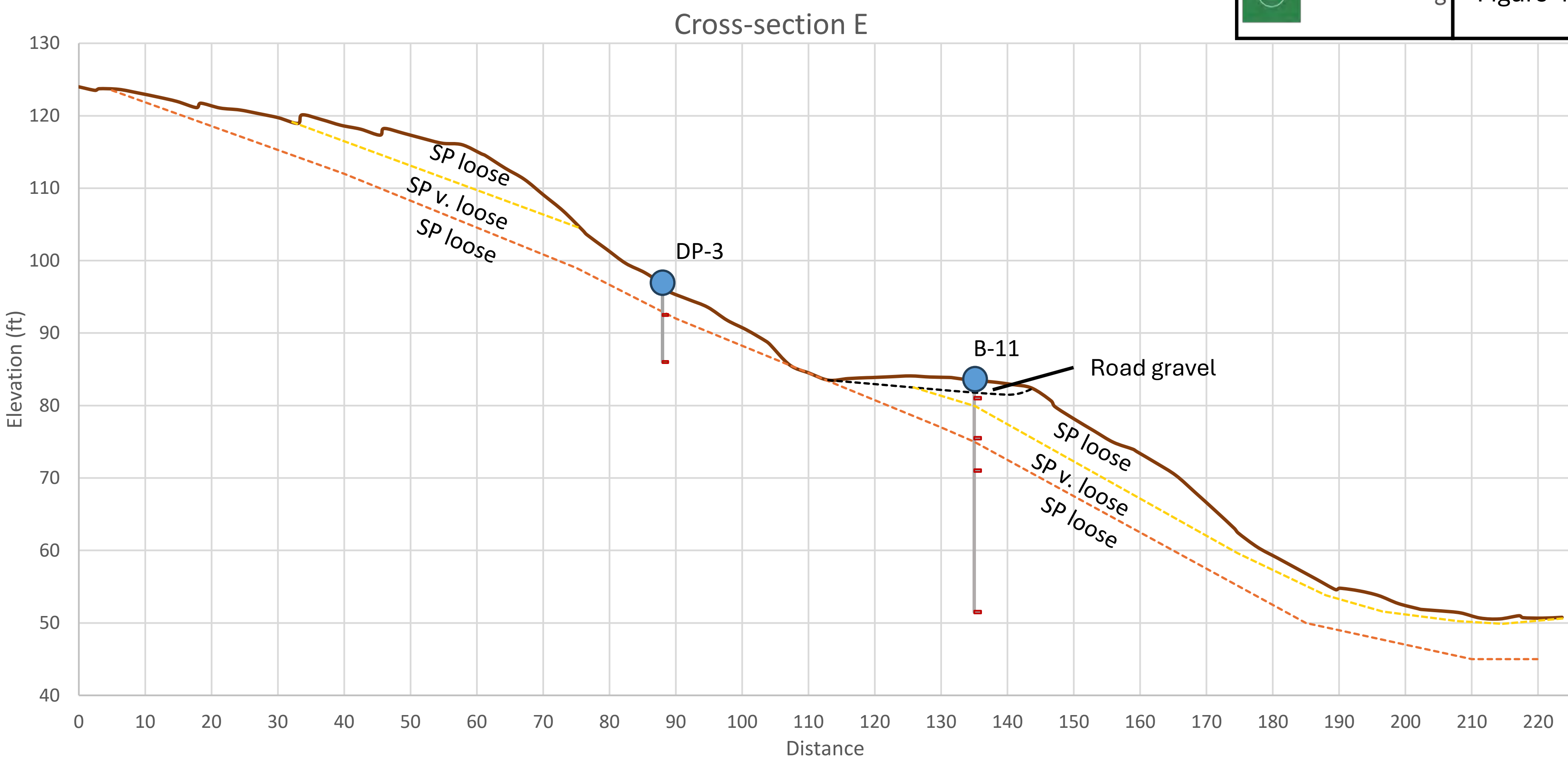
Notes:
No vertical exaggeration.
Looking south through site.

Road and Storm Improvements
Manzanita, OR

Cross-Section E-E'
#074-24-015 March 2025

Pali Consulting

Figure 4E

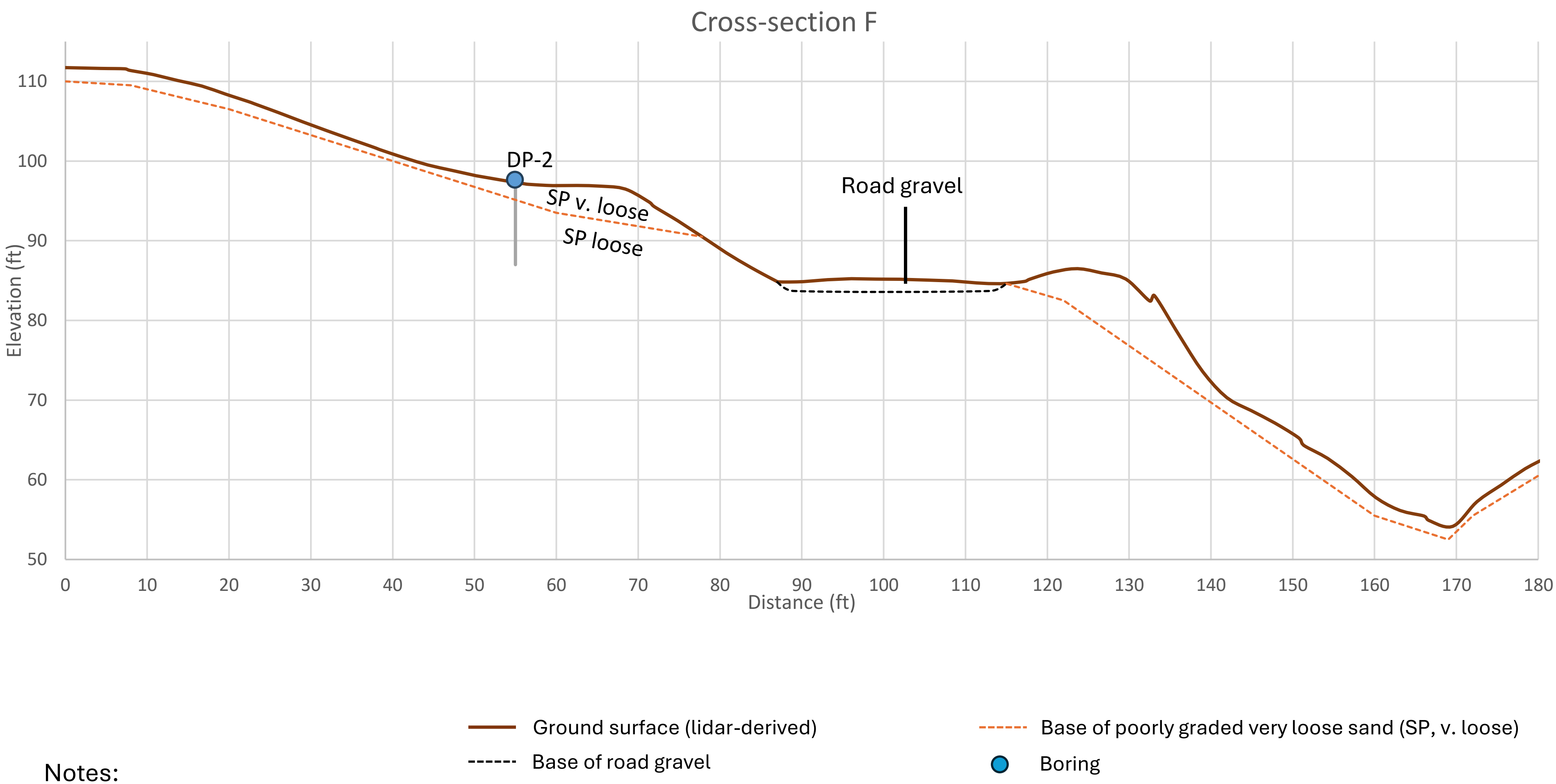


Notes:
No vertical exaggeration.
Looking south through site.

Road and Storm Improvements
Manzanita, OR

Cross-Section F-F’
#074-24-015 March 2025

Pali Consulting | Figure 4F



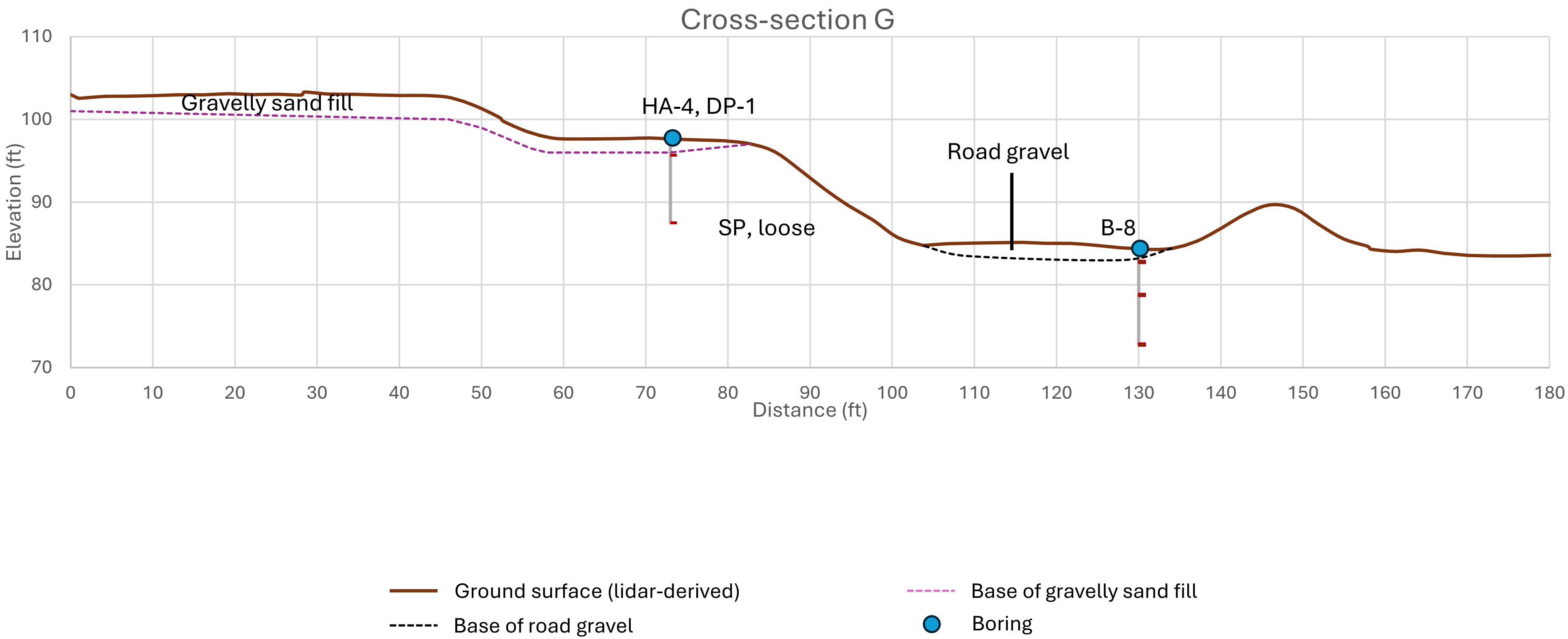
Notes:
No vertical exaggeration.
Looking south through site.

Road and Storm Improvements
Manzanita, OR

Cross-Section G-G'
#074-24-015 March 2025

Pali Consulting

Figure 4G



Notes:
No vertical exaggeration.
Looking south through site.

APPENDIX A -
FIELD EXPLORATIONS AND LABORATORY TESTING

Pali Consulting

FIELD EXPLORATIONS

GENERAL

We evaluated subsurface conditions in the Project area by completing eleven machine-drilled borings, four hand augers, and four drive probe soundings from November 12-15, 2024. Machine-drilled borings were completed using a track-mounted drill rig operated by Western States Soil Conservation, Inc. Hollow stem auger methods were used on all borings except B-6, which used mud rotary methods. The locations of the explorations are shown on Figure 2 of the report and were estimated based on field measurements.

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.

SAMPLING AND LOGGING

The exploration logs within this Appendix show our interpretation of the drilling, sampling, and testing data. They indicate the depth where the soils change. Note that the change may be gradual. In the field, we classified the samples taken from the explorations 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 explorations 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 borings using an SPT sampler 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 cathead operated 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. Undisturbed samples were also obtained from the borings using a Shelby tube sampler in general accordance with ASTM D1587.

INFILTRATION TESTING

We conducted infiltration tests at the intersection of Classic Street and Necarney City Road, as shown on report Figure 2. The test consisted of an encased falling head test in general accordance with US Bureau of Reclamation methods, as briefly described below.

- Hand auger borings were advanced at the test locations to approximate depths of 2.5 to 3.5 feet bgs.
- 4-inch diameter pipe was seated into the bottom of the hole by driving it carefully with a small sledgehammer to create a plug of soil at the base of the pipe.
- The pipe was filled with water to the top and the time for it to infiltrate fully into the ground measured to determine an infiltration rate.
- Two tests were conducted at each location and the data recorded.

The results of the infiltration testing are provided in our report.

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 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.

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 completed on samples from the explorations are presented on the exploration logs included in this Appendix.

Soil Density

The density of undisturbed soil samples were obtained in general accordance with ASTM Test Method D 7263. The results of the density tests 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 coarse- and fine-grained soils. 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.

Direct Shear

Direct shear testing was performed by Northwest Testing, Inc on a select undisturbed sample from Boring B-6 in general accordance with ASTM test method D3080. The test results are included in this Appendix.

Sieve Analyses

Sieve analysis tests were performed on select samples to determine the quantitative distribution of particle sizes in the original sample. The tests were performed in general accordance with ASTM D 6913-04. The test results are indicated in the table below.

Table A-1

Exploration	Depth (feet)	% Gravel	% Sand	% Silt/Clay
B-7	2.5	6	84	11
TP-4	2	42	51	7

KEY TO EXPLORATION LOGS

Pali Consulting

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

SOILS CLASSIFICATION CHART

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

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

SYMBOLS LETTER	DESCRIPTIONS
CC	CEMENT CONCRETE
AC	ASPHALT CONCRETE
TS	TOPSOIL/SOD FOREST DUFF

Stratigraphic Contact

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

Moisture Modifiers

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

Moist - Damp, but no visible water

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

Seepage Modifiers

None

Slow - < 1 gpm

Moderate - 1- 3 gpm

Heavy - > 3 gpm

Caving Modifiers

None

Minor - isolated

Moderate - frequent

Severe - general

Minor Constituents

Trace: < 5% (silt/clay)

Occasional: < 15% (sand/gravel)

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

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

Sampler Symbol Descriptions

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

Laboratory / Field Tests

%F Percent fines

AL Atterberg Limits

CP Laboratory compaction test

CS Consolidation test

DS Direct shear

HA Hydrometer analysis

Laboratory / Field Tests

DD Dry density

OC Organic content

PP Pocket penetrometer

SA Sieve analysis

TV Torvane shear

MC Moisture Content

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

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

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

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

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B-1

Road and Stormwater Improvements Manzanita, Oregon	
Project: Necarney City Road	Driller: Western States Soil Conservation
Proj No. 074-24-015	Date: 11/12/2024
Drilling Method: Hollow Stem Auger	Elevation: 80'
Diameter: 6" / Water Table: Not encountered	Logged by: A. Dunning

Pali Consulting

Sample No.	Sample Type	Recovery (%)	RQD (%)	Blow Count per 6 inches	Blows/Foot (N)	Water Table	Depth (ft BGS)	Graphic Log	USCS	Materials Description	Moisture (%)	Remarks
						GWT not encountered	0		GW	2 inches AC pavement		
									SP	Well-graded ROADWAY GRAVEL (Fill)		
										Loose, moist, brown, poorly-graded fine SAND		
S1		100		2-3-4	7							
							5					
S2		100		2-3-2	5						4	
S3		100		2-3-3	6							
							10					
S4		100		2-3-5	8						6	
										Boring completed at 11.5 ft bgs		
							15					
							20					
							25					
							30					

File: C:\Users\Tech\Pali Consulting Dropbox\1-Projects\Active-Projects\074-WindsorEngineers\074-24-015ManzanitaRoad&StormImprovements\Analysis\Logs\ManzanitaBorings2024.log Date: 12/20/2024

Pali Consulting

B-2

Road and Stormwater Improvements
Manzanita, Oregon

Pali Consulting

Project: Necarney City Road	Driller: Western States Soil Conservation
Proj No. 074-24-015	Date: 11/12/2024
Drilling Method: Hollow Stem Auger	Elevation: 95'
Diameter: 6" Water Table: Not encountered	Logged by: A. Dunning

Sample No.	Sample Type	Recovery (%)	RQD (%)	Blow Count per 6 inches	Blows/Foot (N)	Water Table	Depth (ft BGS)	Graphic Log	USCS	Materials Description	Moisture (%)	Remarks
						GWT not encountered	0		GW	Well-graded ROADWAY GRAVEL (Fill)		
									SP	Loose, moist, brown, poorly-graded fine SAND with trace fines		
S1		100		2-4-5	9						4	
							5			Grades to no fines		
S2		100		1-2-2	4						4	%F=2
S3		100		1-2-2	4						4	
							10					
S4		100		1-2-3	5							
										Boring completed at 11.5 ft bgs		
							15					
							20					
							25					
							30					

Pali Consulting

B-3

Road and Stormwater Improvements
Manzanita, Oregon

Pali Consulting

Project: Necarney City Road	Driller: Western States Soil Conservation
Proj No. 074-24-015	Date: 11/12/2024
Drilling Method: Hollow Stem Auger	Elevation: 90'
Diameter: 6" — Water Table: Not encountered	Logged by: A. Dunning

Sample No.	Sample Type	Recovery (%)	RQD (%)	Blow Count per 6 inches	Blows/Foot (N)	Water Table	Depth (ft BGS)	Graphic Log		Materials Description	Moisture (%)	Remarks
						GWT not encountered	0		GW	2 inches AC pavement		
										Well-graded ROADWAY GRAVEL (Fill)		
									SP			
S1		100		1-3-3	6					Loose, dry, yellow, poorly-graded fine SAND with trace angular gravel (Fill?)	3	
							5			Loose, dry, yellow, poorly-graded fine SAND without gravel (Native?)		
S2		100								Grades to gray		No SPT count
S3		100		3-4-3	7						1	
							10					
S4		100		3-4-7	11					Grades to medium dense		
										Boring completed at 11.5 ft bgs		
							15					
							20					
							25					
							30					

Pali Consulting

B-4

Road and Stormwater Improvements
Manzanita, Oregon

Pali Consulting

Project: Classic Street	Driller: Western States Soil Conservation
Proj No. 074-24-015	Date: 11/12/2024
Drilling Method: Hollow Stem Auger	Elevation: 75'
Diameter: 6" / Water Table: Not encountered	Logged by: A. Dunning

Sample No.	Sample Type	Recovery (%)	RQD (%)	Blow Count per 6 inches	Blows/Foot (N)	Water Table	Depth (ft BGS)	Graphic Log	USCS	Materials Description	Moisture (%)	Remarks
						GWT not encountered	0		GW	2 inches AC pavement Well-graded ROADWAY GRAVEL (Fill)		
									SP	Loose, moist, gray, poorly-graded fine SAND		
S1		75		2-4-4	8							
S2		100		2-2-1	3		5			Grades to very loose, brown	9	
S3		100		1-1-2	3							
S4		100		1-2-3	5		10			Grades to loose	5	
S5		100		2-3-4	7		15					
S6		100		3-3-4	7		20			Grades to gray	5	
S7		100		4-5-6	11		25			Grades to medium dense		
S8		100		2-3-3	6		30			Grades to loose	4	
										Boring completed at 31.5 ft bgs.		

File: C:\Users\Tech\Pali Consulting Dropbox\1-Projects\Active-Projects\074-WindsorEngineers\074-24-015ManzanitaRoad&StormImprovements\Analysis\Logs\ManzanitaBorings2024.log Date: 12/20/2024

File: C:\Users\Tech\Pali Consulting Dropbox\1-Projects\Active-Projects\074-WindsorEngineers\074-24-015ManzanitaRoad&StormImprovements\Analysis\Logs\ManzanitaBorings2024.log Date: 12/20/2024

Pali Consulting	Road and Stormwater Improvements
B-5	**Manzanita, Oregon**
Project: Classic Street	Driller: Western States Soil Conservation
Proj No. 074-24-015	Date: 11/12/2024
Drilling Method: Hollow Stem Auger	Elevation: 80'
Diameter: 6" / Water Table: 38'	Logged by: A. Dunning

Sample No.	Sample Type	Recovery (%)	RQD (%)	Blow Count per 6 inches	Blows/Foot (N)	Water Table	Depth (ft BGS)	Graphic Log	USCS	Materials Description	Moisture (%)	Remarks
							0		GW	2 inches AC pavement		
									SP	Well-graded ROADWAY GRAVEL (Fill)		
S1		100										
S2		100		1-1-1	2					Very loose, dry, gray, poorly-graded fine SAND		
S3		100		2-2-2	4		5				4	
S4		100		1-1-1	2							
S5		100		0-1-1	2		10				6	
S6		100		1-2-3	5		15			Grades to loose		
S7		100		1-2-4	6		20				2	
S8		100		0-1-1	2		25			Grades to very loose, moist		
S9		100		9-14-13	27		30			Grades to medium dense	6	

Pali Consulting

B-5

Road and Stormwater Improvements
Manzanita, Oregon

Pali Consulting

Project: Classic Street	Driller: Western States Soil Conservation
Proj No. 074-24-015	Date: 11/12/2024
Drilling Method: Hollow Stem Auger	Elevation: 80'
Diameter: 6" Water Table: 38'	Logged by: A. Dunning

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
							35				
S10		100		5-9-8	17					13	
						▼					
							40				
S11		100		2-4-7	11				Grades to wet	26	
									Boring completed at 43 ft bgs.		
							45				
							50				
							55				
							60				
							65				

File: C:\Users\Tech\Pali Consulting Dropbox\1-Projects\Active-Projects\074-WindsorEngineers\074-24-015ManzanitaRoad&StormImprovements\Analysis\Logs\ManzanitaBorings2024.log Date: 12/20/2024

Pali Consulting

B-6

Road and Stormwater Improvements
Manzanita, Oregon

Pali Consulting

Project: Classic Street	Driller: Western States Soil Conservation
Proj No. 074-24-015	Date: 11/14/2024
Drilling Method: Mud Rotary	Elevation: 80'
Diameter: 6" / Water Table: Could not determine	Logged by: A. Dunning

Sample No.	Sample Type	Recovery (%)	RQD (%)	Blow Count per 6 inches	Blows/Foot (N)	Water Table	Moisture (%)	Remarks
S1		33		2-2-2	4	GWT not encountered		SA
S2		33		2-3-3	6			Gravel likely sloughed from top of boring
S3		33		2-2-2	4		18	
S4		100						
S5		33		3-2-3	5			
S6		33		2-3-4	7		24	
S7		8						
S8		33		4-5-5	10		23	
S9		33		4-6-8	14			
S10		33		6-9-12	21			

Depth (ft BGS): 0, 5, 10, 15, 20, 25, 30

Graphic Log / Materials Description:

- **GW** — 2 inches AC pavement
- Well-graded ROADWAY GRAVEL (Fill)
- **SP-GP** — Loose, wet, brown, fine gravelly SAND (Fill)
- **SP** — Loose, wet, brown, poorly-graded fine SAND with trace gravel (Native)
  - Grades to moist
  - Grades to wet
  - Grades to medium dense

File: C:\Users\Tech\Pali Consulting Dropbox\1-Projects\Active-Projects\074-WindsorEngineers\074-24-015ManzanitaRoad&StormImprovements\Analysis\Logs\ManzanitaBorings2024.log Date: 12/20/2024

Pali Consulting

B-6

Road and Stormwater Improvements
Manzanita, Oregon

Pali Consulting

Project: Classic Street	Driller: Western States Soil Conservation
Proj No. 074-24-015	Date: 11/14/2024
Drilling Method: Mud Rotary	Elevation: 80'
Diameter: 6" / Water Table: Could not determine	Logged by: A. Dunning

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
							35		Grades to dense		
S11		33		9-18-20	38					18	
							40				
S12		33		14-20-25	45						
							45		Grades to very dense		
S13		100		20-25-40	65					21	
							50				
S14		100		15-25-31	56						
									Boring completed at 51.5 ft bgs.		
							55				
							60				
							65				

File: C:\Users\Tech\Pali Consulting Dropbox\1-Projects\Active-Projects\074-WindsorEngineers\074-24-015ManzanitaRoad&StormImprovements\Analysis\Logs\ManzanitaBorings2024.log Date: 12/20/2024

File: C:\Users\Tech\Pali Consulting Dropbox\1-Projects\Active-Projects\074-WindsorEngineers\074-24-015ManzanitaRoad&StormImprovements\Analysis\Logs\ManzanitaBorings2024.log Date: 12/20/2024

Pali Consulting

B-7

Road and Stormwater Improvements
Manzanita, Oregon

Pali Consulting

Project: Classic Street	Driller: Western States Soil Conservation
Proj No. 074-24-015	Date: 11/13/2024
Drilling Method: Hollow Stem Auger	Elevation: 85'
Diameter: 6" / Water Table: Not encountered	Logged by: A. Dunning

Sample No.	Sample Type	Recovery (%)	RQD (%)	Blow Count per 6 inches	Blows/Foot (N)	Water Table	Depth (ft BGS)	Graphic Log	USCS	Materials Description	Moisture (%)	Remarks
						GWT not encountered	0		GW	2 inches AC pavement		
										Well-graded ROADWAY GRAVEL (Fill)		
									SP			
S1		100		1-3-3	6					Loose, damp, brown, poorly-graded fine SAND with trace angular gravel (Fill)		
							5			Loose, damp, brown, poorly-graded fine SAND without gravel (Native)		
S2		100		1-1-1	2					Grades to very loose	10	%F=2
S3		100		0-1-1	2						14	%F=3
							10					
S4		100		1-1-1	2						5	
										Boring completed at 11.5 ft bgs.		
							15					
							20					
							25					
							30					

Pali Consulting

B-8

Road and Stormwater Improvements
Manzanita, Oregon

Pali Consulting

Project: Classic Street	Driller: Western States Soil Conservation
Proj No. 074-24-015	Date: 11/13/2024
Drilling Method: Hollow Stem Auger	Elevation: 90'
Diameter: 6" / Water Table: Not encountered	Logged by: A. Dunning

Sample No.	Sample Type	Recovery (%)	RQD (%)	Blow Count per 6 inches	Blows/Foot (N)	Water Table	Depth (ft BGS)	Graphic Log	USCS	Materials Description	Moisture (%)	Remarks
						GWT not encountered	0		GW	2 inches AC pavement Well-graded ROADWAY GRAVEL (Fill)		
S1		100		6-7-9	16				SP	Medium dense, moist, brown, poorly-graded fine SAND with trace organics	4	
S2		100		4-4-4	8		5			Grades to loose gray with orange mottling and no organics		
S3		100		1-1-1	2					Grades to very loose, brown, with thin beds containing trace organics	4	
S4		100		1-2-2	4		10			Grades to loose with no organics		
										Boring completed at 11.5 ft bgs.		

Depth scale: 0, 5, 10, 15, 20, 25, 30

File: C:\Users\Tech\Pali Consulting Dropbox\1-Projects\Active-Projects\074-WindsorEngineers\074-24-015ManzanitaRoad&StormImprovements\Analysis\Logs\ManzanitaBorings2024.log Date: 12/20/2024

Date: 12/20/2024

File: C:\Users\Tech\Pali Consulting Dropbox\1-Projects\Active-Projects\074-WindsorEngineers\074-24-015ManzanitaRoad&StormImprovements\Analysis\Logs\ManzanitaBorings2024.log

Pali Consulting

B-9

Road and Stormwater Improvements
Manzanita, Oregon

Pali Consulting

Project: Classic Street	Driller: Western States Soil Conservation
Proj No. 074-24-015	Date: 11/13/2024
Drilling Method: Hollow Stem Auger	Elevation: 100'
Diameter: 6" / Water Table: Not encountered	Logged by: A. Dunning

Sample No.	Sample Type	Recovery (%)	RQD (%)	Blow Count per 6 inches	Blows/Foot (N)	Water Table	Depth (ft BGS)	Graphic Log		Materials Description	Moisture (%)	Remarks
						GWT not encountered	0		GW	2 inches AC pavement Well-graded ROADWAY GRAVEL (Fill)		
									SP			
S1		100		6-13-15	28					Medium dense, dry, brown, poorly-graded fine SAND	4	
S2		100		5-13-17	30		5					
S3		100		6-13-15	28							
S4		100		5-7-8	15		10			Grades to orange Grades to gray	4	
										Boring completed at 11.5 ft bgs.		
							15					
							20					
							25					
							30					

Pali Consulting

B-10

Road and Stormwater Improvements
Manzanita, Oregon

Pali Consulting

Project: Classic Street	Driller: Western States Soil Conservation
Proj No. 074-24-015	Date: 11/13/2024
Drilling Method: Hollow Stem Auger	Elevation: 75'
Diameter: 6" / Water Table: Not encountered	Logged by: A. Dunning

Sample No.	Sample Type	Recovery (%)	RQD (%)	Blow Count per 6 inches	Blows/Foot (N)	Water Table	Depth (ft BGS)	Graphic Log	Materials Description	Moisture (%)	Remarks
						GWT not encountered	0	GW	2 inches AC pavement		
								SP	Well-graded ROADWAY GRAVEL (Fill)		
S1		100		3-4-5	9				Loose, moist, brown, poorly-graded fine SAND with trace angular gravel (Fill)	9	SA
S2		100		4-4-4	8		5		Loose, moist, brown, poorly-graded fine SAND without gravel (Native) Grades to gray	4	
S3		100		2-2-2	4				Grades to brown		
S4		100		1-2-3	5		10			4	
S5		100									
S6		100		2-4-6	10		15				
S7		100					20			13	DD = 98.7 PCF
S8		100		5-6-7	13		25		Grades to medium dense and gray		
S9		100		4-6-7	13		30			4	
									Boring completed at 31.5 ft bgs.		

Date: 12/20/2024

File: C:\Users\Tech\Pali Consulting Dropbox\1-Projects\Active-Projects\074-WindsorEngineers\074-24-015ManzanitaRoad&StormImprovements\Analysis\Logs\ManzanitaBorings2024.log

Pali Consulting

B-11

Road and Stormwater Improvements
Manzanita, Oregon

Pali Consulting

Project: Classic Street	Driller: Western States Soil Conservation
Proj No. 074-24-015	Date: 11/14/2024
Drilling Method: Hollow Stem Auger	Elevation: 80'
Diameter: 6" / Water Table: Not encountered	Logged by: A. Dunning

Sample No.	Sample Type	Recovery (%)	RQD (%)	Blow Count per 6 inches	Blows/Foot (N)	Water Table	Depth (ft BGS)	Graphic Log	USCS	Materials Description	Moisture (%)	Remarks
						GWT not encountered	0		GW	2 inches AC pavement Well-graded ROADWAY GRAVEL (Fill)		
S1		100		4-5-5	10				SP	Loose, moist, brown, poorly-graded fine SAND		
S2		100		2-3-3	6		5					
S3		100		1-1-1	2				SP	Grades to very loose		
S4		100					10					
S5		100		2-2-3	5				SP	Grades to loose		
S6		100		2-2-3	5		15					
S7		100					20				10	DD = 99 PCF
S8		100										No SPT count
S9		100		2-3-4	7		25					
S10		100		3-5-5	10		30					
										Boring completed at 31.5 ft bgs.		

HA-1

Classic St., Manzanita
074-24-015

Drill Rig: Hand auger
Sampling: Grab
Logged By: A. Dunning
Total Depth: 6.25 ft
Groundwater: Not encountered

Date Started: 11/14/24
Date Completed: 11/14/24
Elevation: 100'
Coordinates: N 45.7161
W 123.9291

Description	Graphic Log	Depth	Sample Type	SPT N-Value	MC (%)	Remarks
Top Soil with organics and sand	TS	0	GWT not encountered			
Loose, mosit, orange, poorly-graded fine SAND with organics and roots	SP		Grab		6	S1
		1				
		2	Grab			S2
Grades to brown without organics						
		3				
		4	Grab		6	S3
		5				
		6	Grab			S4
Boring completed at 6.25 ft bgs.						
		7				

Pali Consulting

HA-2

Classic St., Manzanita
074-24-015

Drill Rig: Hand auger
Sampling: Grab
Logged By: A. Dunning
Total Depth: 6.25
Groundwater: Not encountered

Date Started: 11/15/2024
Date Completed: 11/15/2024
Elevation: 120'
Coordinates: N 45.7162 N
W 123.9289

Description	Graphic Log	Depth	Sample Type	SPT N-Value	MC (%)	Remarks
Top Soil with organics and sand	TS	0	GWT not encountered			
Loose, moist, brown, poorly-graded fine SAND with trace organics and roots	SP	1	Grab			S1
		2				
		3	Grab		6	S2
Grades to no organics or roots		4				
		5				
		6	Grab			S3
Boring completed at 6.25 ft bgs.		7				

Pali Consulting

Plate A-13

HA-3

Classic St., Manzanita
074-24-015

Drill Rig: Hand auger
Sampling: Grab
Logged By: A. Dunning
Total Depth: 6.25
Groundwater: Not encountered

Date Started: 11/15/2024
Date Completed: 11/15/2024
Elevation: 90'
Coordinates: N 45.7162, W 123.9291

Description	Graphic Log	Depth	Sample Type	SPT N-Value	MC (%)	Remarks
Top Soil with organics and sand	TS	0	GWT not encountered			
Loose, moist, brown, poorly-graded fine SAND with trace organics and roots	SP	1	Grab			S1
Grades to no organics or roots	SP	2	Grab		5	S2
	SP	3				
	SP	4	Grab			S3
	SP	5				
	SP	6	Grab		8	S4
Boring completed at 6.25 ft bgs.		7				

Pali Consulting

HA-4

Classic St., Manzanita
074-24-015

Drill Rig: Hand auger
Sampling: Grab
Logged By: A. Dunning
Total Depth: 10.25
Groundwater: Not encountered

Date Started: 11/15/2024
Date Completed: 11/15/2024
Elevation: 97'
Coordinates: N 45.7143
W 123.9298

Description	Graphic Log	Depth	Sample Type	SPT N-Value	MC (%)	Remarks
Top Soil	TS	0				
Loose, moist, brown, poorly-graded fine silty SAND with organics and roots	ML-SP		Grab			S1
Loose, moist, brown poorly-graded fine SAND with occasional angular gravel 1"-3"	SP	1				
Grades to no gravel						
		2	Grab		7	S2
		3				
		4				
		5	Grab			S3
		6				
		7				

GWT not encountered

Pali Consulting

Plate A-15

HA-4

Classic St., Manzanita
074-24-015

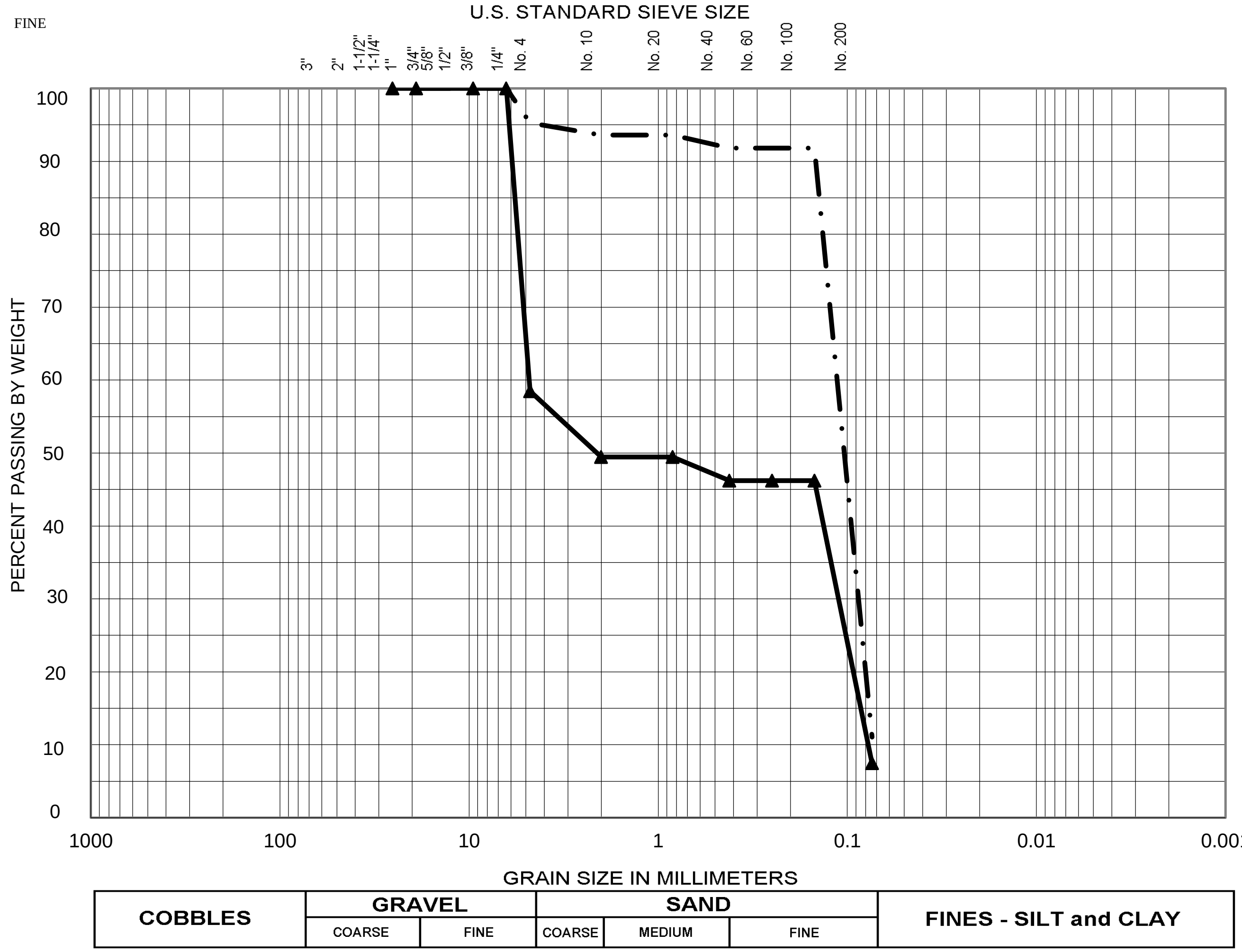
Drill Rig: Hand auger
Sampling: Grab
Logged By: A. Dunning
Total Depth: 10.25
Groundwater: Not encountered

Date Started: 11/15/2024
Date Completed: 11/15/2024
Elevation: 97'
Coordinates: N 45.7143
W 123.9298

Description	Graphic Log	Depth	Sample Type	SPT N-Value	MC (%)	Remarks
Loose, moist, brown poorly-graded fine SAND with occasional angular gravel 1"-3"	SP	7				
			Grab		7	S4
		8				
			Grab		7	S5
		9				
		10	Grab			S6
Boring completed at depth of 10.25 ft bgs.						
		11				
		12				
		13				
		14				

Pali Consulting

Plate A-15



Symbol	Boring and Sample No.	As Rec'd Water Content, %	C_u	C_c	USC	Description
— · —	**B10 ; S1 @ 2.5ft**	9	1.7	0.8	SP	Gravelly fine sand
———	**B6 ; S1 @ 2.5ft**	0	61.5	0.0	SP-GP	Gravelly fine sand

Boring and Sample No.	**Soil / Aggregate Composition in Percent**			
	Gravel	**Sand**	**Fines**	**Total**
B10 ; S1 @ 2.5ft	**5**	**84**	**11**	**100**
B6 ; S1 @ 2.5ft	**42**	**51**	**7**	**100**

Test Method: ASTM C136			
Project	Manzanita Road and Storm Improvements	**Date Tested**	11/21/2024
File No.	074-24-015	**Tested By**	AD
Lab ID No.	NA	**Checked By**	TB

NOTE: Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations or generated by separate operations or processes.

Pali Consulting

Boring B-6, B-10 Grain Size Analysis

Plate A-16

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

TECHNICAL REPORT

Report To:	Tim Blackwood, PE, GE, CEG Pali Consulting, Inc. 1120 SW Fifth Avenue, Suite 1302 Portland, Oregon 97204	**Date:** **Lab No.:**	12/16/2024 24-788
Project:	Manzanita	**Project No.:**	00-223425-1
Report of:	Direct shear testing of soil.		

Sample Identification

As requested, NTI provided direct shear testing of soil on a tube sample delivered to our laboratory by a Pali Consulting, Inc. representative on December 4, 2024. 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-12, S-4 @ 10.0 Ft.

Direct Shear Test of Soils Under Consolidated Drained Conditions – Sample Data (ASTM D3080)			
Test	**500psf Normal Load Initial Conditions**	**1500psf Normal Load Initial Conditions**	**2500psf Normal Load Initial Conditions**
Moisture Content, (%)	9.3	9.3	9.3
Dry Unit Weight, (pcf)	91.7	97.2	89.6
Peak Shear Strength, (psf)	437	919	1739

Note: Displacement rate used during testing, 0.025 inches/min.

Attachments: Laboratory Test Results – Direct Shear

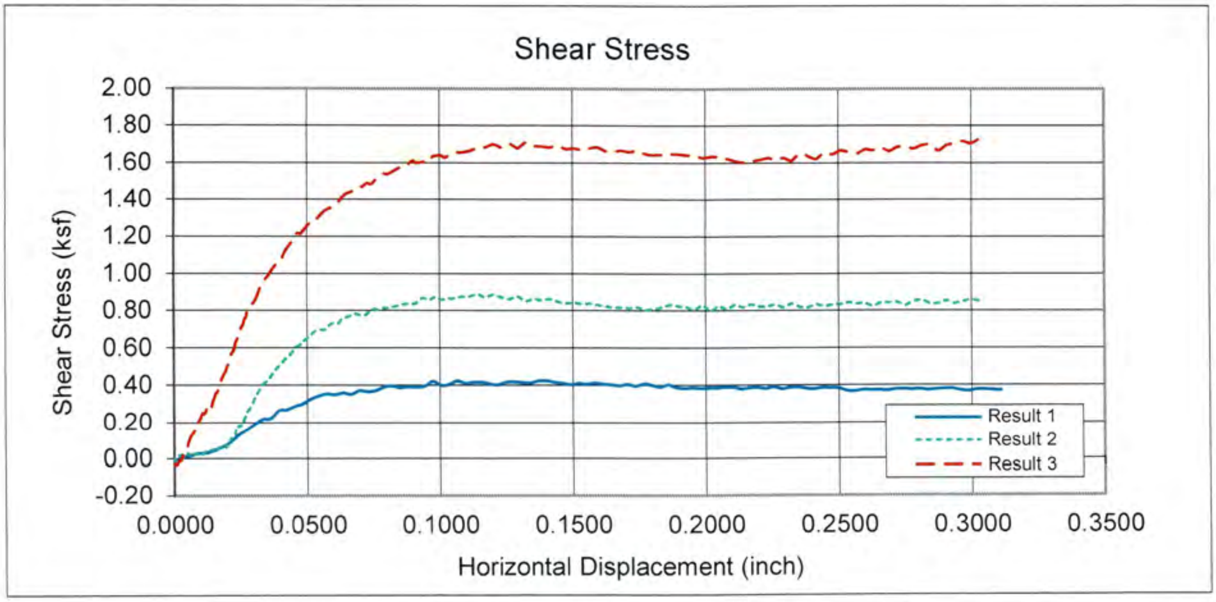
Copies: (1) Addressee
(1) Joshua Robles, Pali Consulting, Inc.

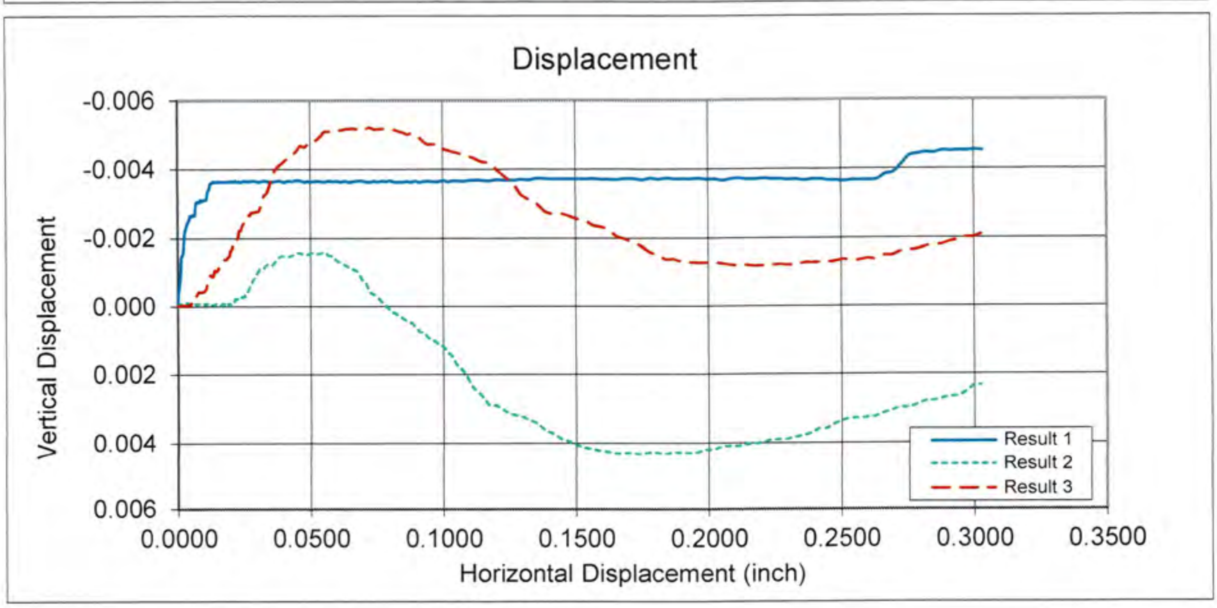
Northwest Testing, Inc.
An RMA Company
9120 SW Pioneer Court, Suite B, Wilsonville, Oregon 97070 | ph: 503.682.1880 fax: 503.682.2753 | www.nwgeotech.com

TECHNICAL REPORT

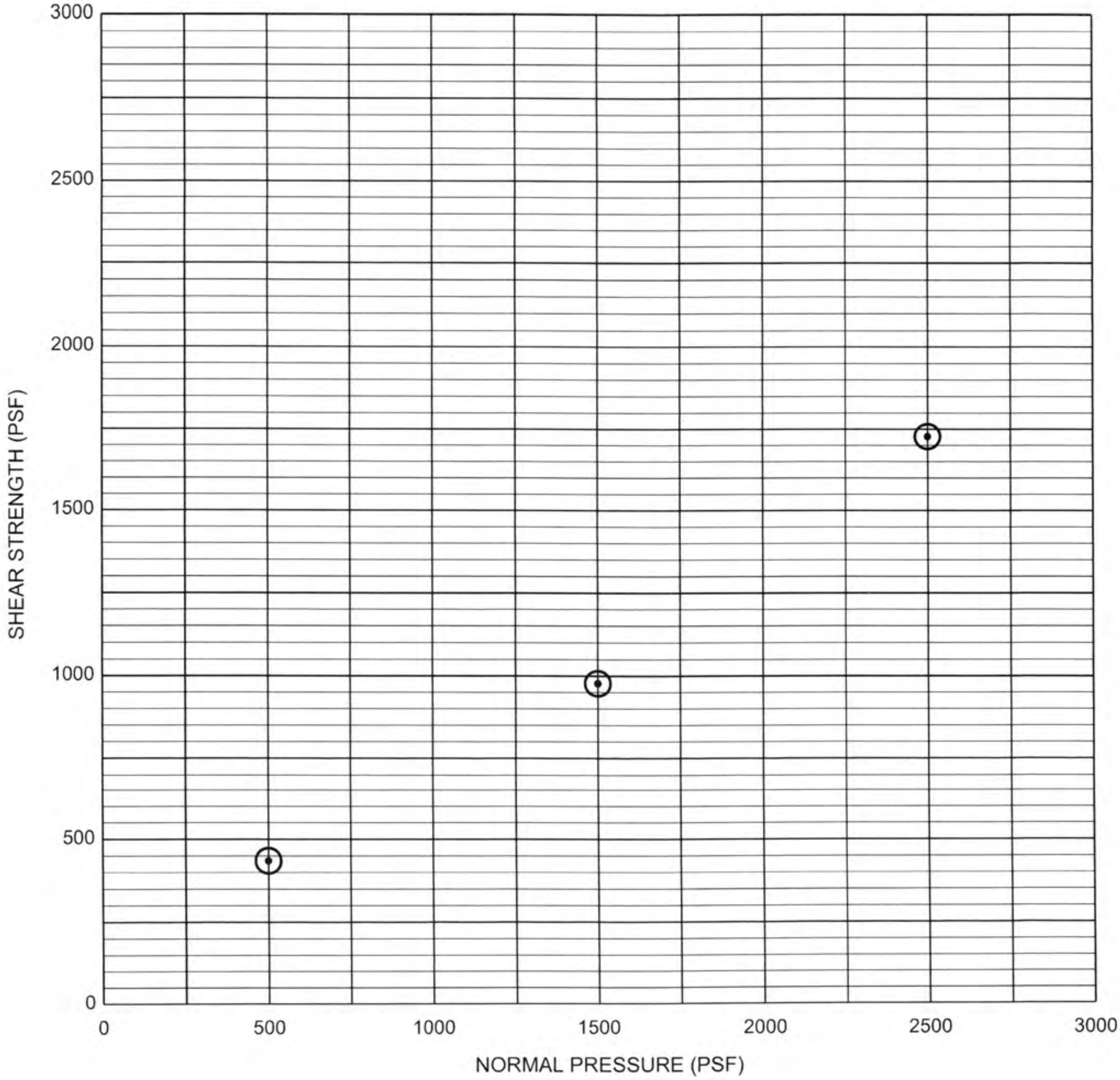
Report To:	Tim Blackwood, PE, GE, CEG Pali Consulting, Inc. 1120 SW Fifth Avenue, Suite 1302 Portland, Oregon 97204	**Date:** **Lab No.:**	12/16/2024 24-788
Project:	Manzanita	**Project No.:**	00-223425-1

Sample ID: B-12, S-4 @ 10.0 Ft.





 REVIEWED BY: Mitchell Guha



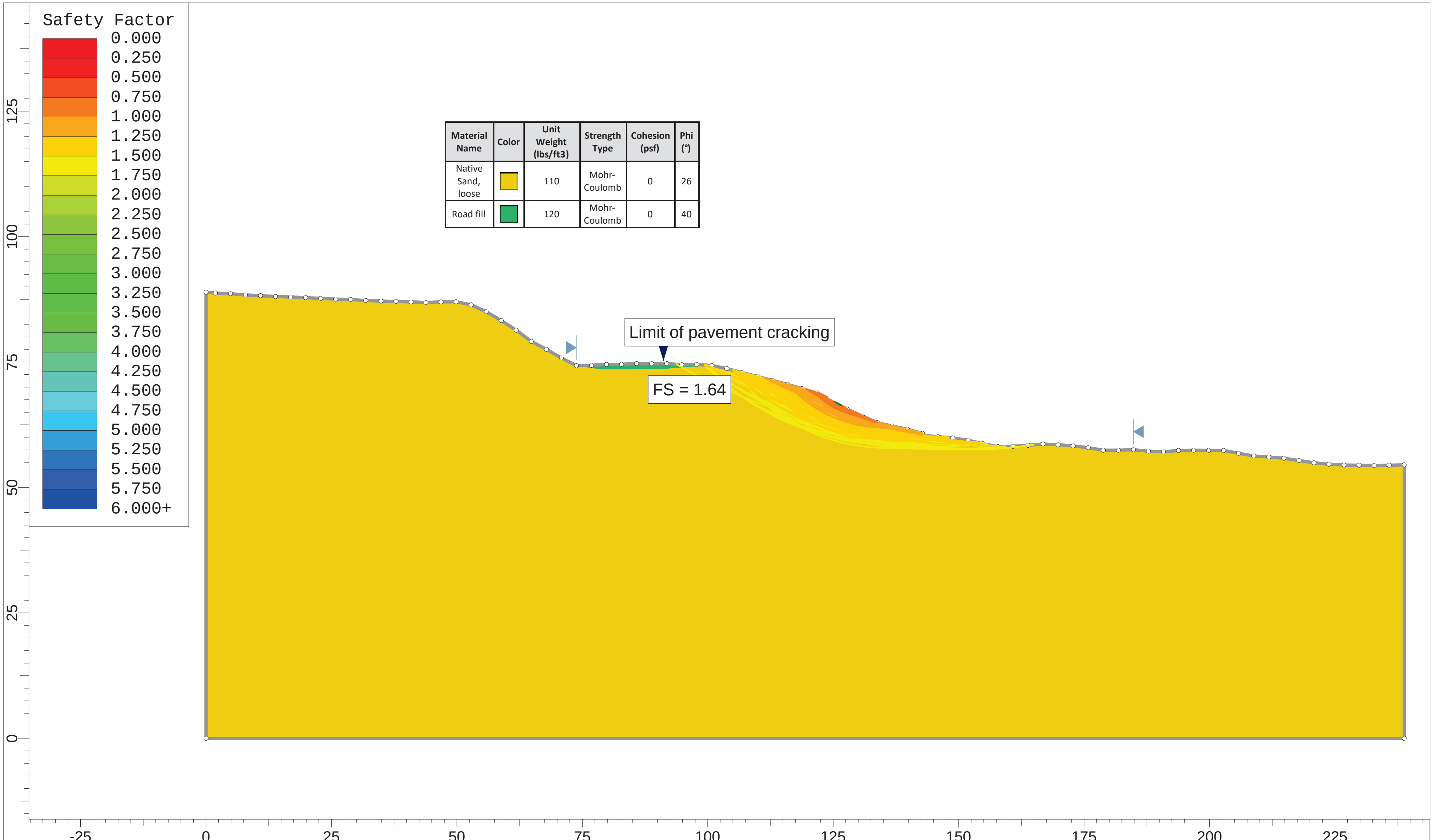
SYMBOL	SAMPLE LOCATION	COHESION (psf)	FRICTION ANGLE	REMARKS
	B-12, S-4 @ 10.0 FT.	--	--	SATURATED; UNDISTURBED

DIRECT SHEAR TEST RESULTS - ASTM D3080

PROJECT NO. 00-223425-1	MANZANITA PALI CONSULTING, INC.	LAB NO. 24-778

Northwest Testing, Inc.
An RMA Company

APPENDIX B -
SLOPE STABILITY ANALYSIS



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Native Sand, loose		110	Mohr-Coulomb	0	26
Road fill		120	Mohr-Coulomb	0	40

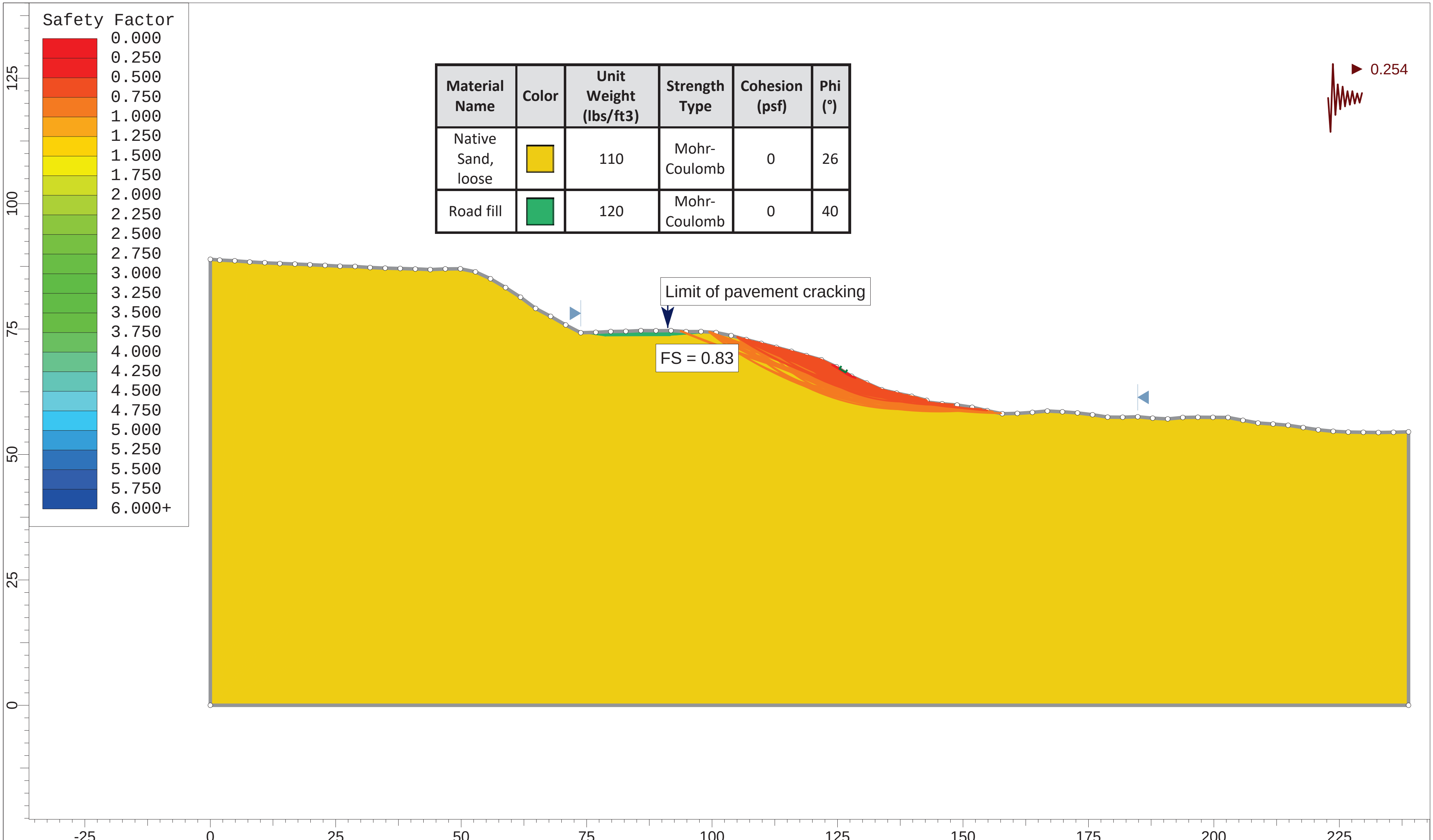
Manzanita - Cross-Section A
Slope Stability Analysis

Base Case

Scale 1:325 | 3/25/2025

Pali Consulting

Figure
B-1



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Native Sand, loose		110	Mohr-Coulomb	0	26
Road fill		120	Mohr-Coulomb	0	40

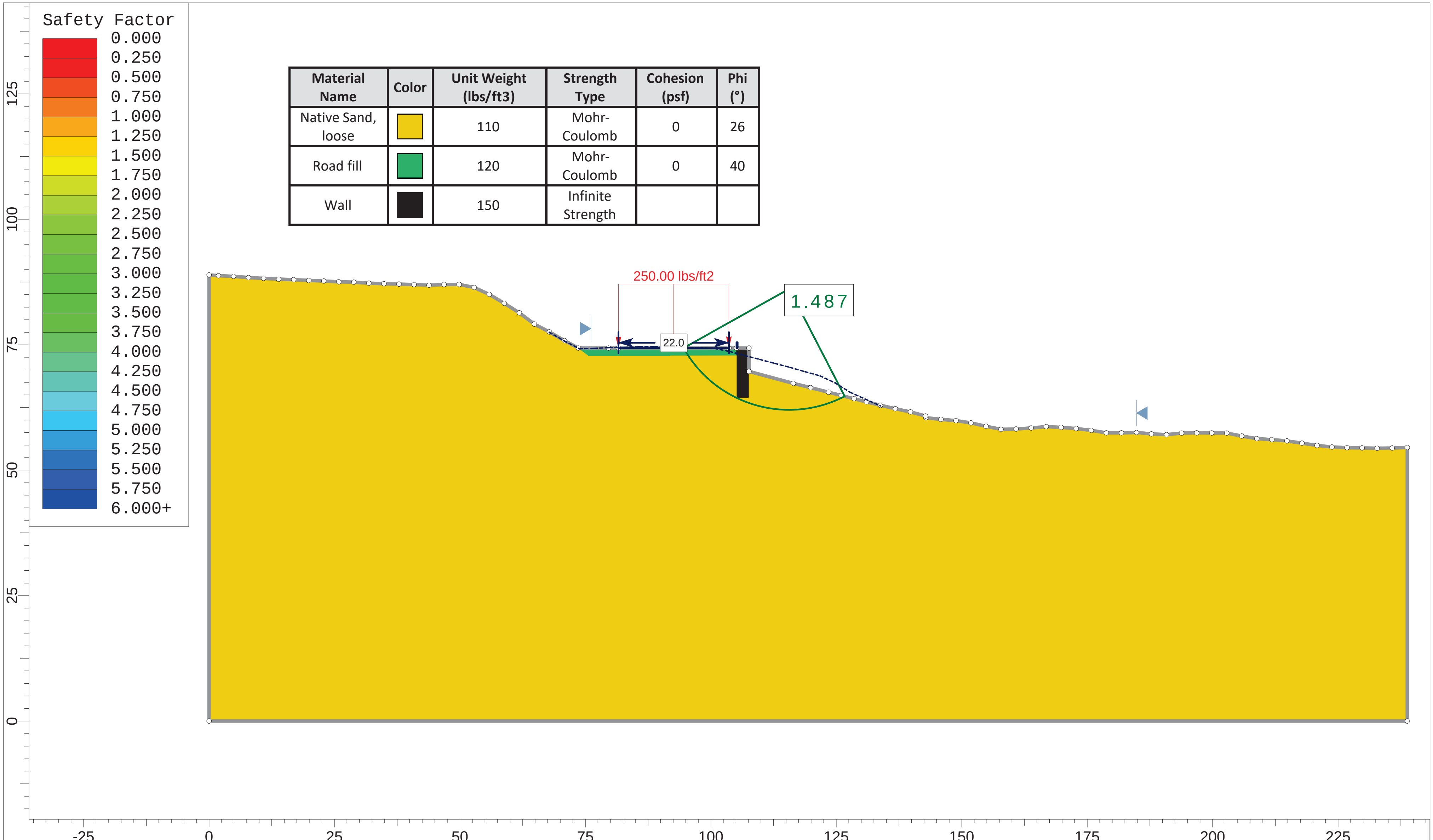
Manzanita - Cross-Section A
Slope Stability Analysis

Seismic

Scale 1:325 | 3/25/2025

Pali Consulting

Figure B-2



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Native Sand, loose		110	Mohr-Coulomb	0	26
Road fill		120	Mohr-Coulomb	0	40
Wall		150	Infinite Strength		

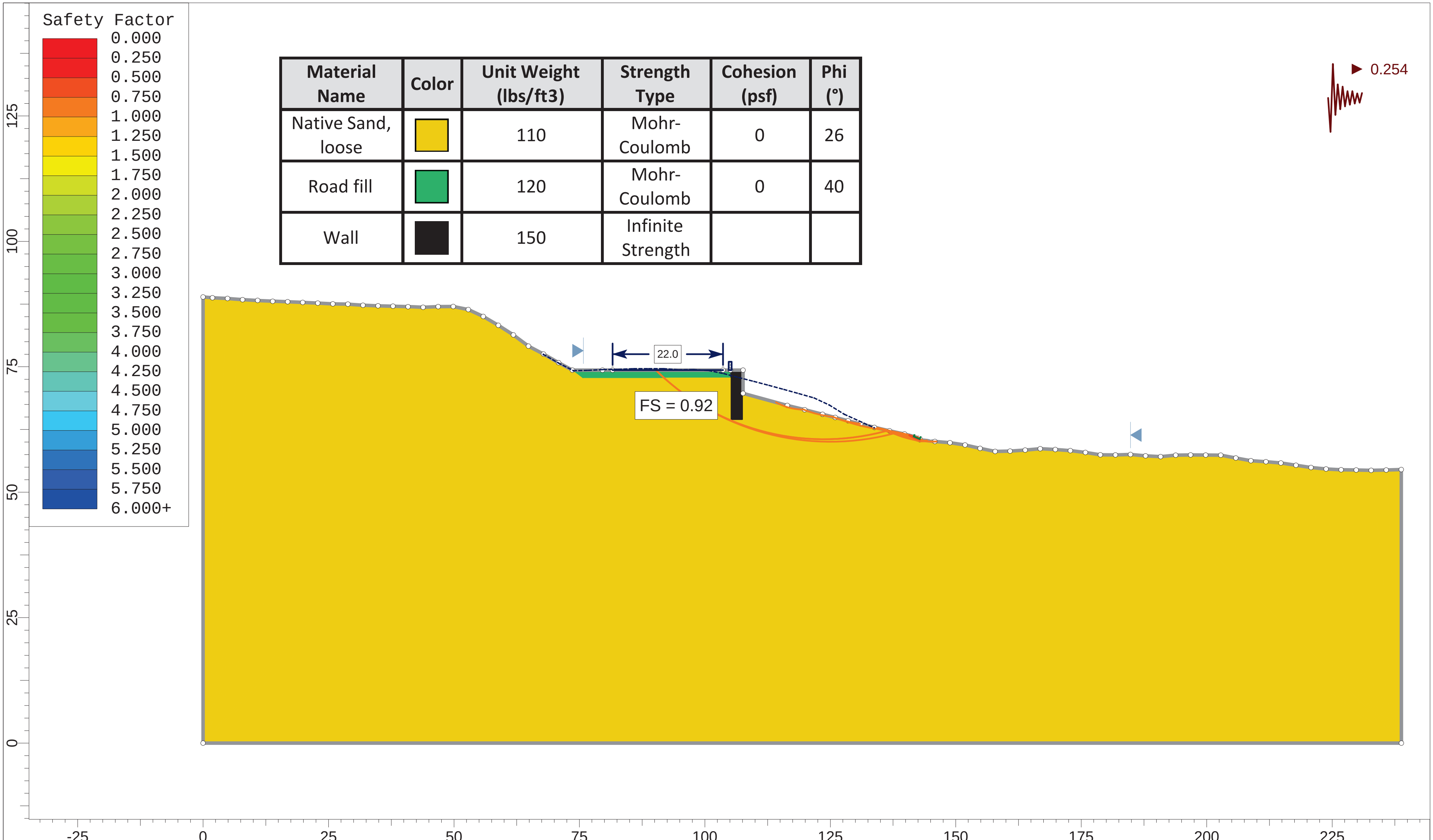
Manzanita - Cross-Section A
Slope Stability Analysis

90% Static

Scale 1:325 | 3/25/2025

Pali Consulting

Figure B-3



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Native Sand, loose		110	Mohr-Coulomb	0	26
Road fill		120	Mohr-Coulomb	0	40
Wall		150	Infinite Strength		

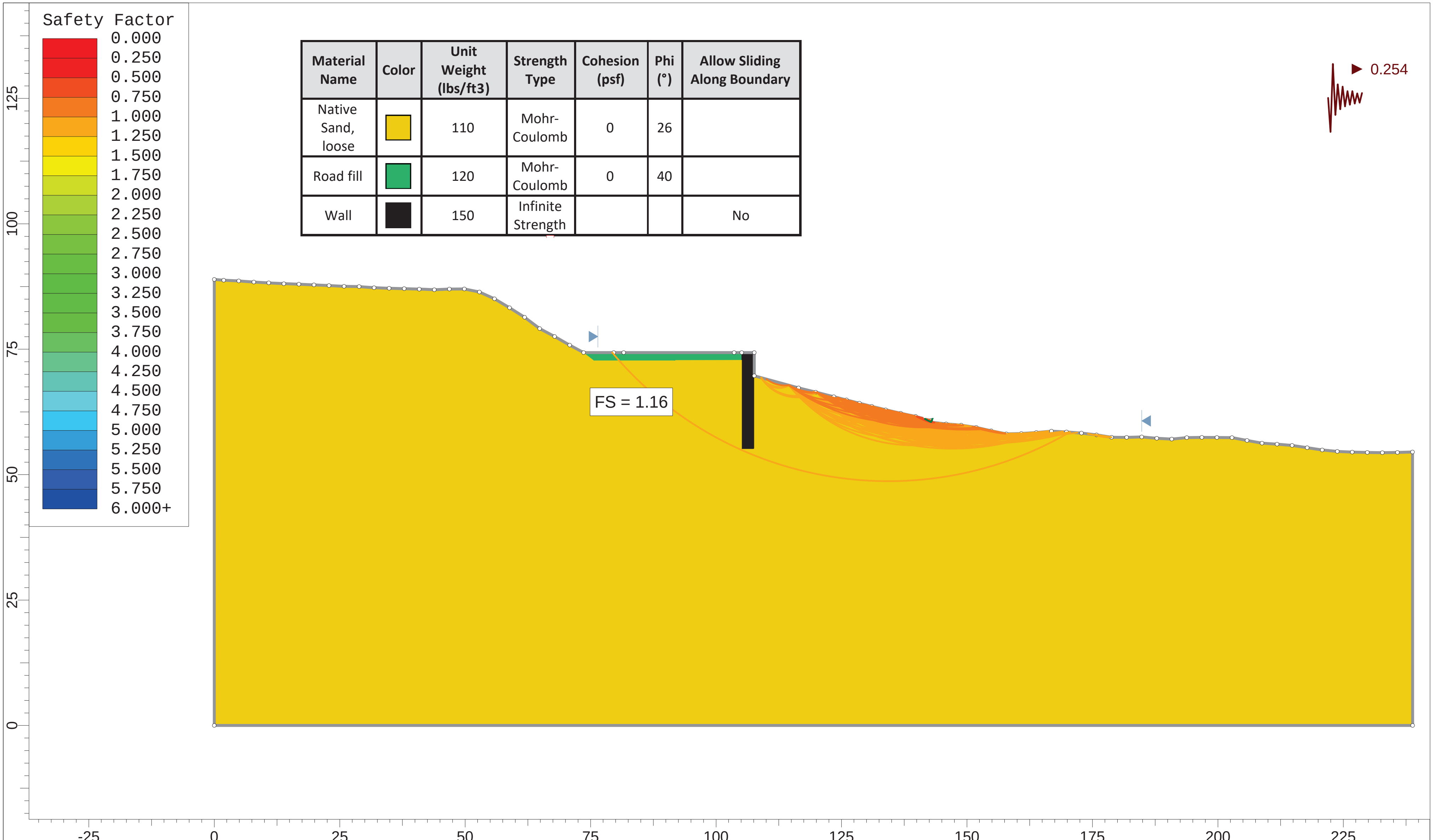
Manzanita - Cross-Section A
Slope Stability Analysis

90% Seismic

Scale 1:325 | 3/25/2025

Pali Consulting

Figure B-4

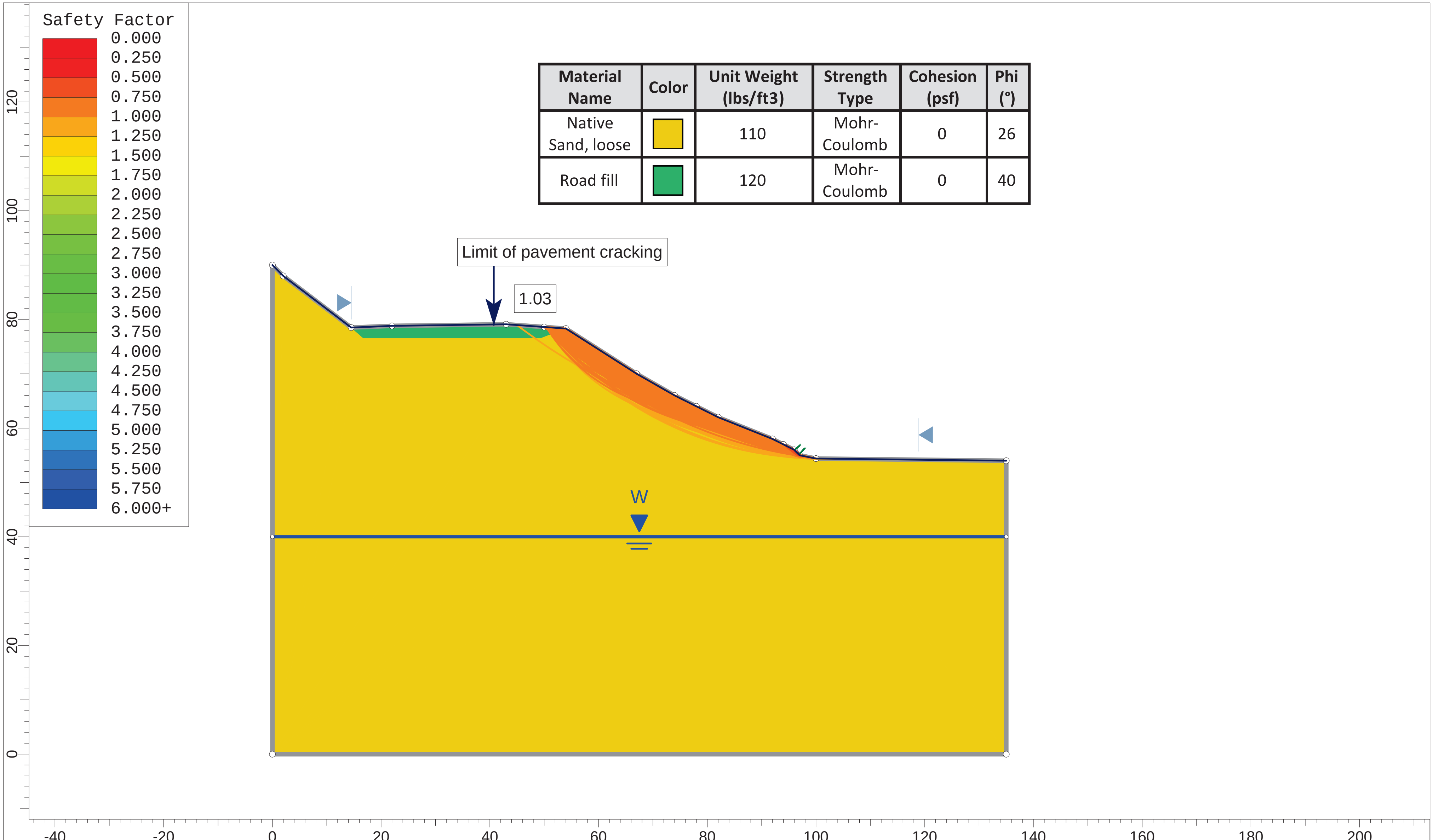


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Allow Sliding Along Boundary
Native Sand, loose		110	Mohr-Coulomb	0	26	
Road fill		120	Mohr-Coulomb	0	40	
Wall		150	Infinite Strength			No

Manzanita - Cross-Section A
Slope Stability Analysis

90% Seismic, FS >1.0

Scale 1:325	3/25/2025
Pali Consulting	Figure B-5



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Native Sand, loose		110	Mohr-Coulomb	0	26
Road fill		120	Mohr-Coulomb	0	40

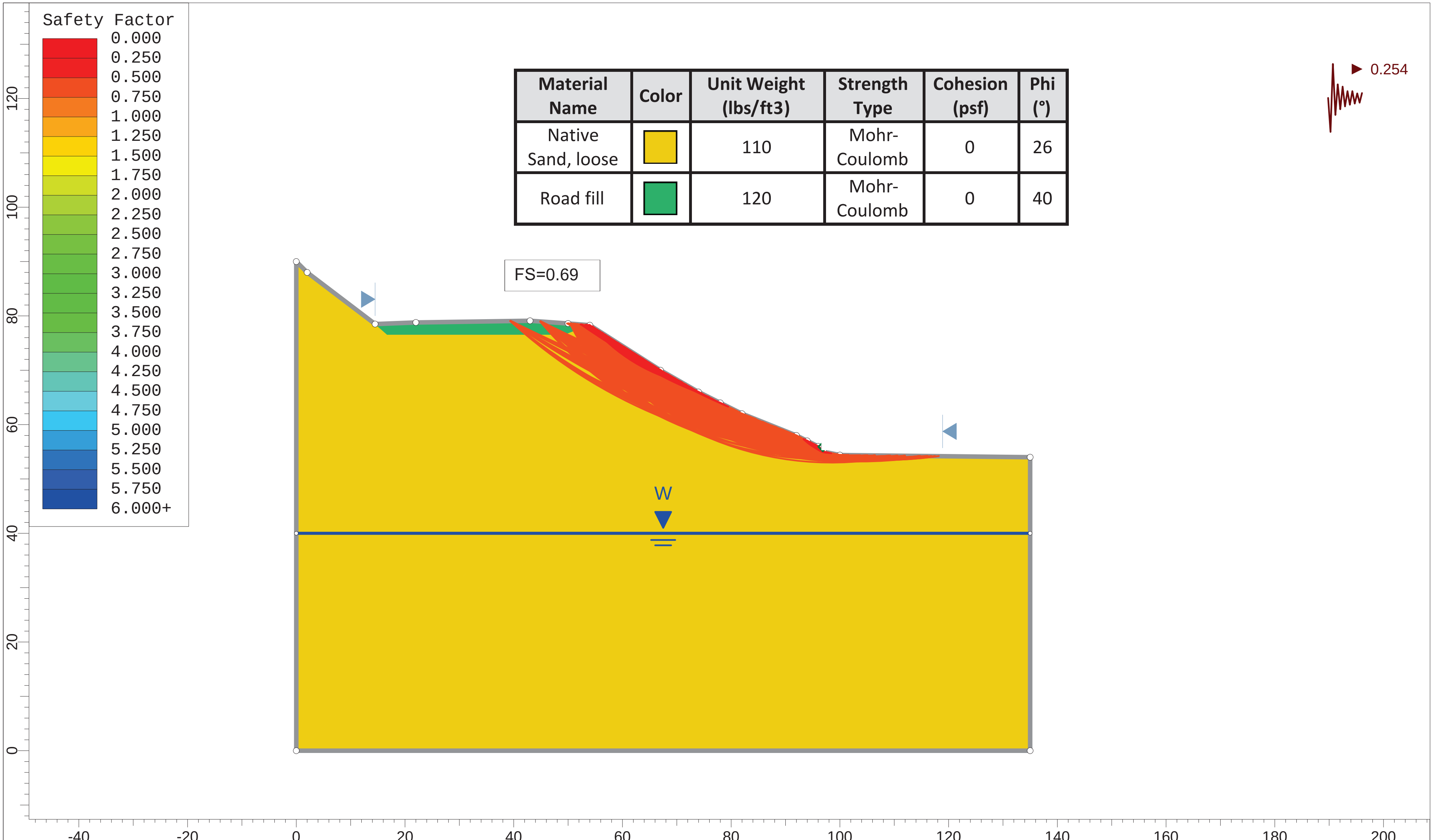
Manzanita Cross Section C
Slope Stability Analysis

Existing Conditions

Scale 1:300 | 3/25/2025

Pali Consulting

Figure B-6



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Native Sand, loose		110	Mohr-Coulomb	0	26
Road fill		120	Mohr-Coulomb	0	40

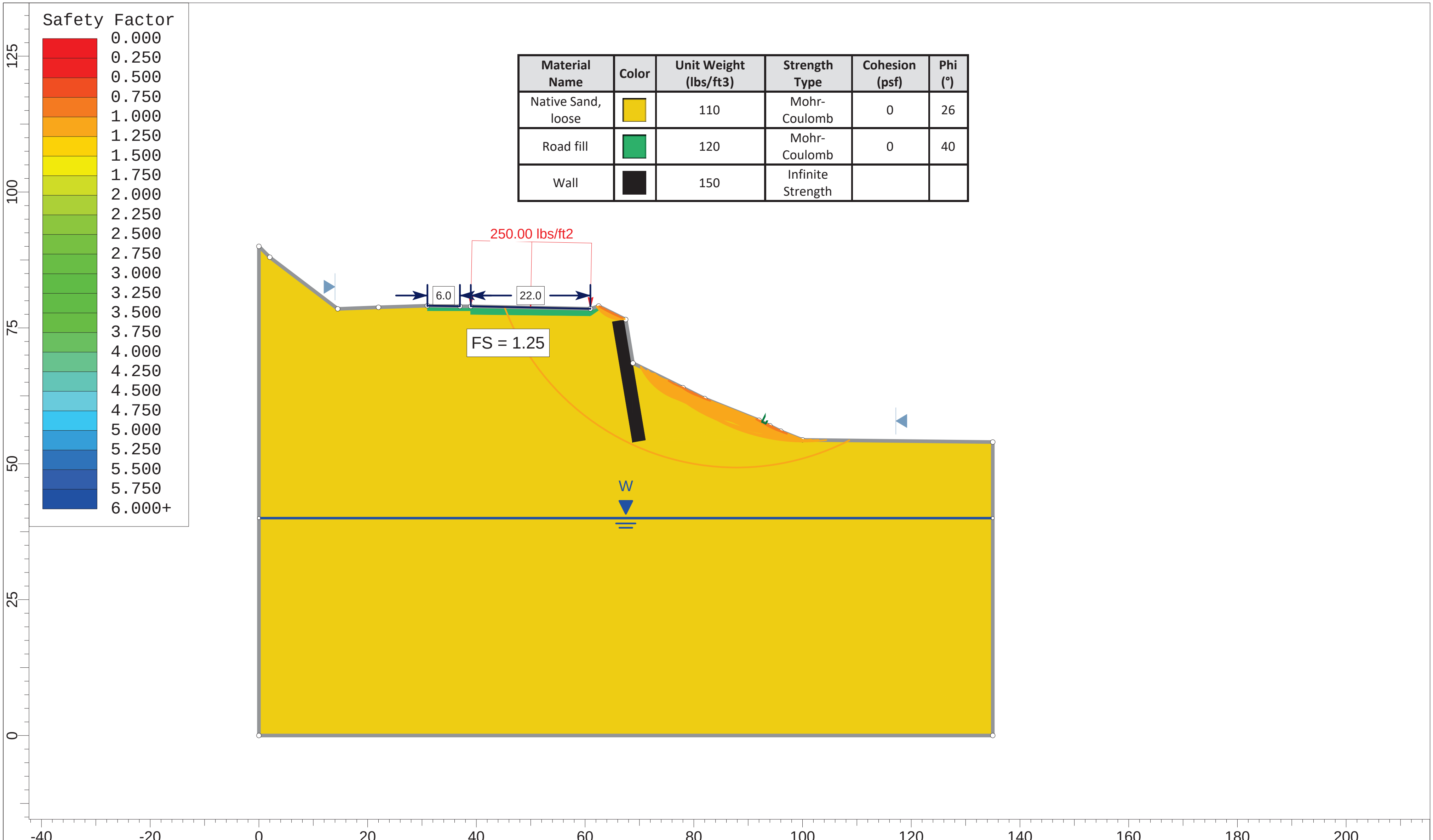
Manzanita Cross-Section C

Slope Stability Analysis

Existing Conditions

Seismic

Scale 1:300	3/25/2025
Pali Consulting	Figure B-7



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Native Sand, loose		110	Mohr-Coulomb	0	26
Road fill		120	Mohr-Coulomb	0	40
Wall		150	Infinite Strength		

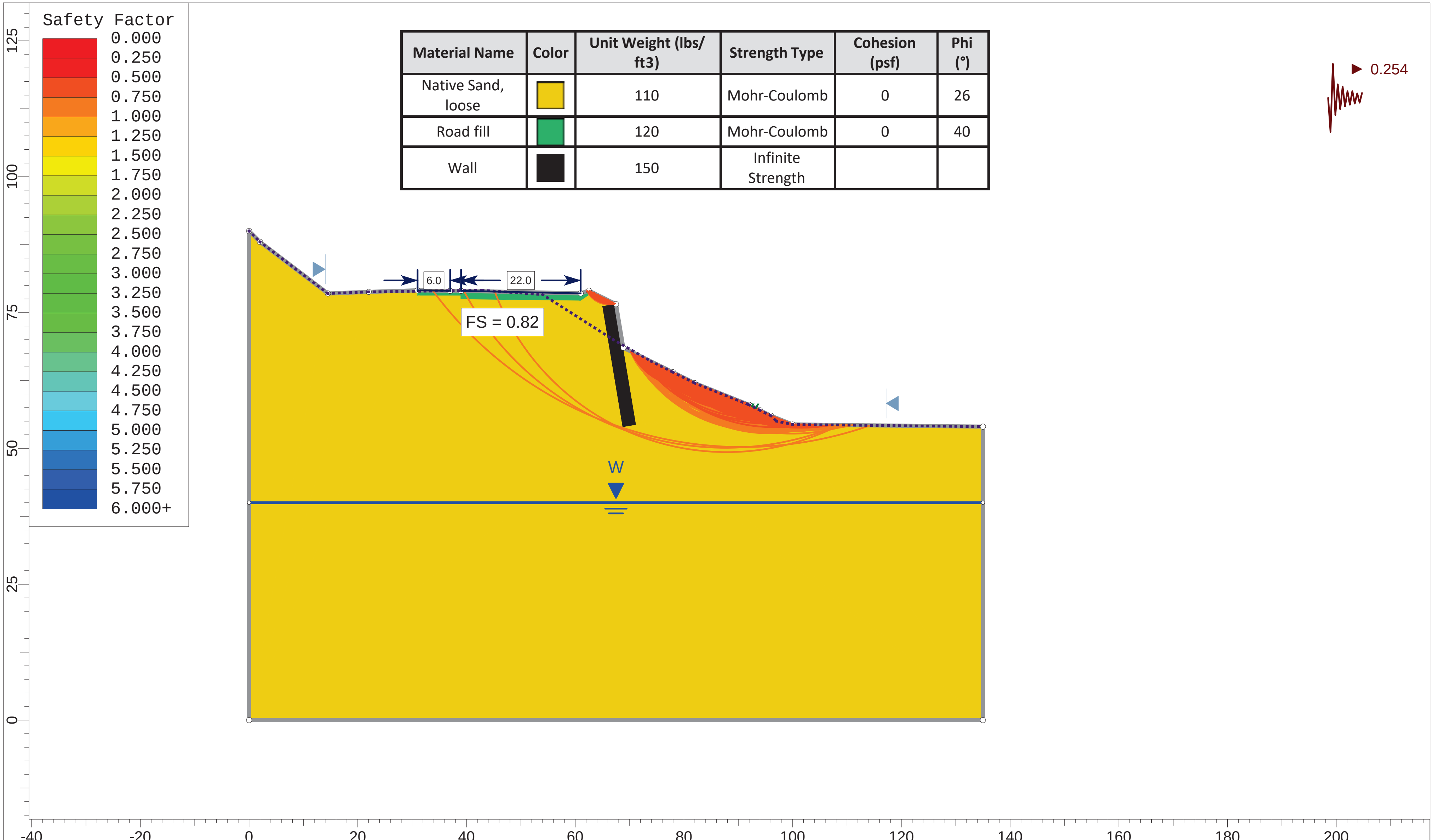
Manzanita Cross-Section C
Slope Stability Analysis

90% Plan (Static)

Scale 1:300 | 3/25/2025

Pali Consulting

Figure
B-8



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Native Sand, loose		110	Mohr-Coulomb	0	26
Road fill		120	Mohr-Coulomb	0	40
Wall		150	Infinite Strength		

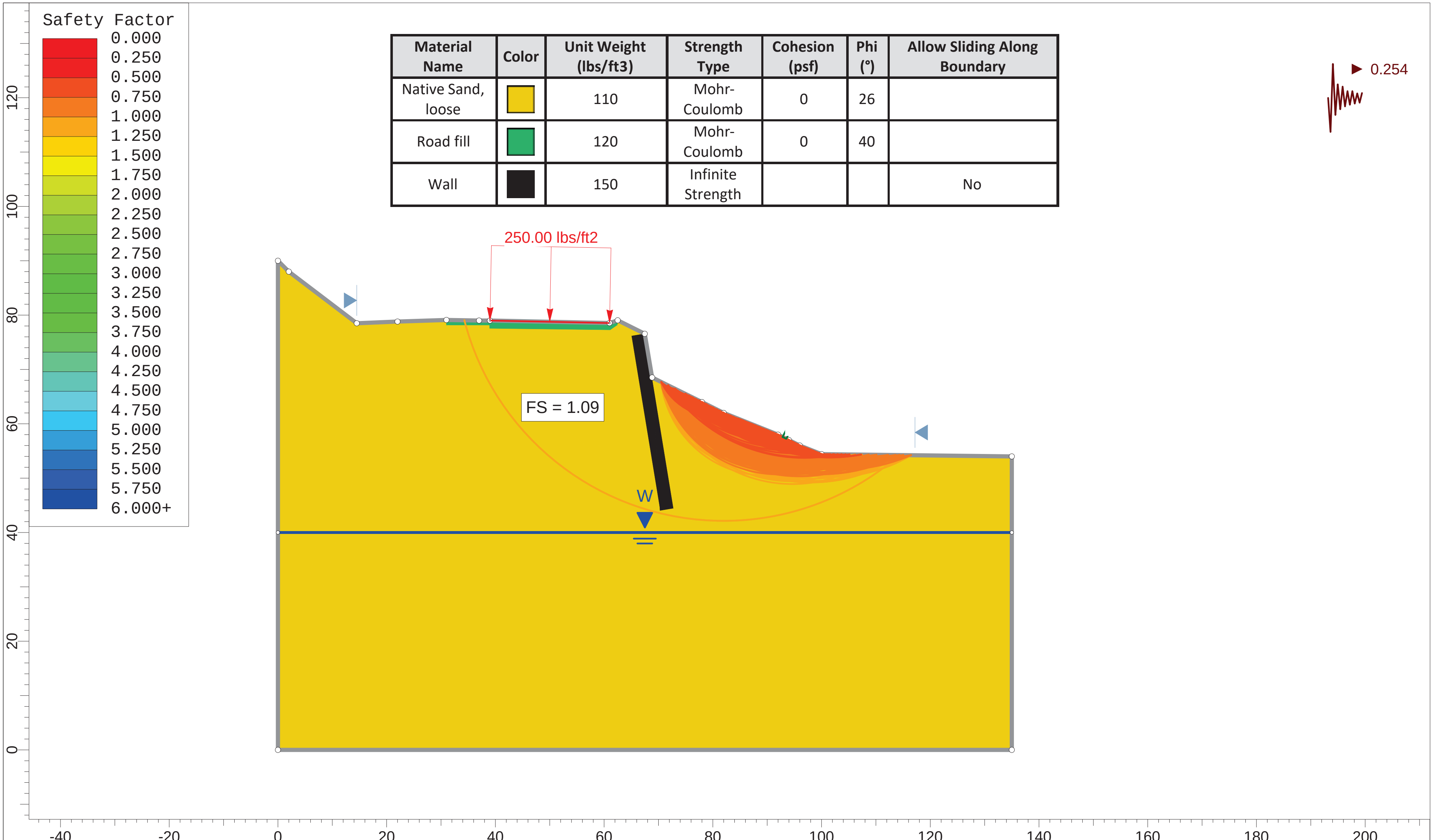
Manzanita Cross-Section C
Slope Stability Analysis

90% Plan (Static)

Scale 1:300 | 3/25/2025

Pali Consulting

Figure B-9



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Allow Sliding Along Boundary
Native Sand, loose		110	Mohr-Coulomb	0	26	
Road fill		120	Mohr-Coulomb	0	40	
Wall		150	Infinite Strength			No

Manzanita Cross-Section C

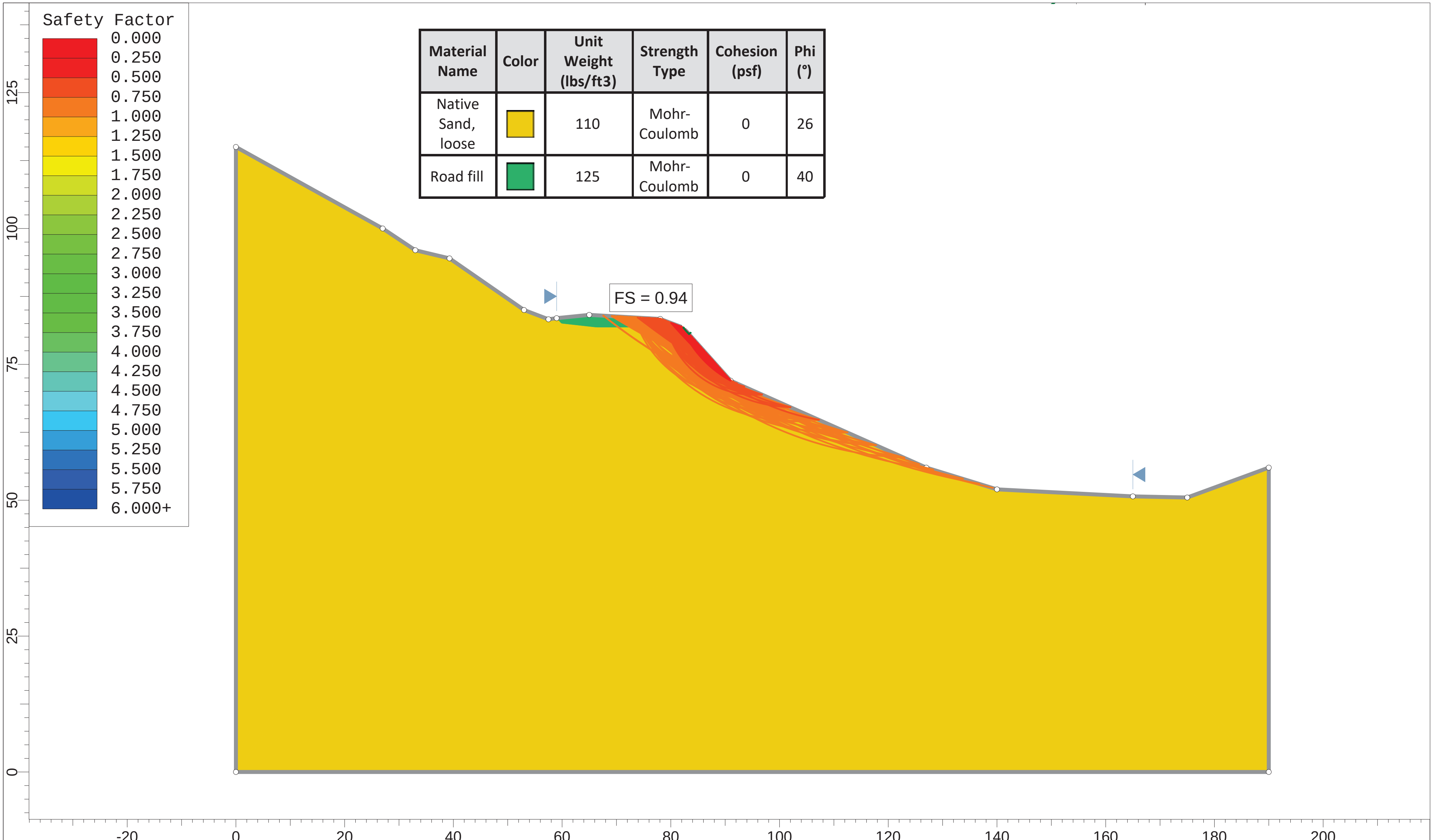
Slope Stability Analysis

Seismic FS >1.0

Scale 1:300 | 3/25/2025

Pali Consulting

Figure B-10

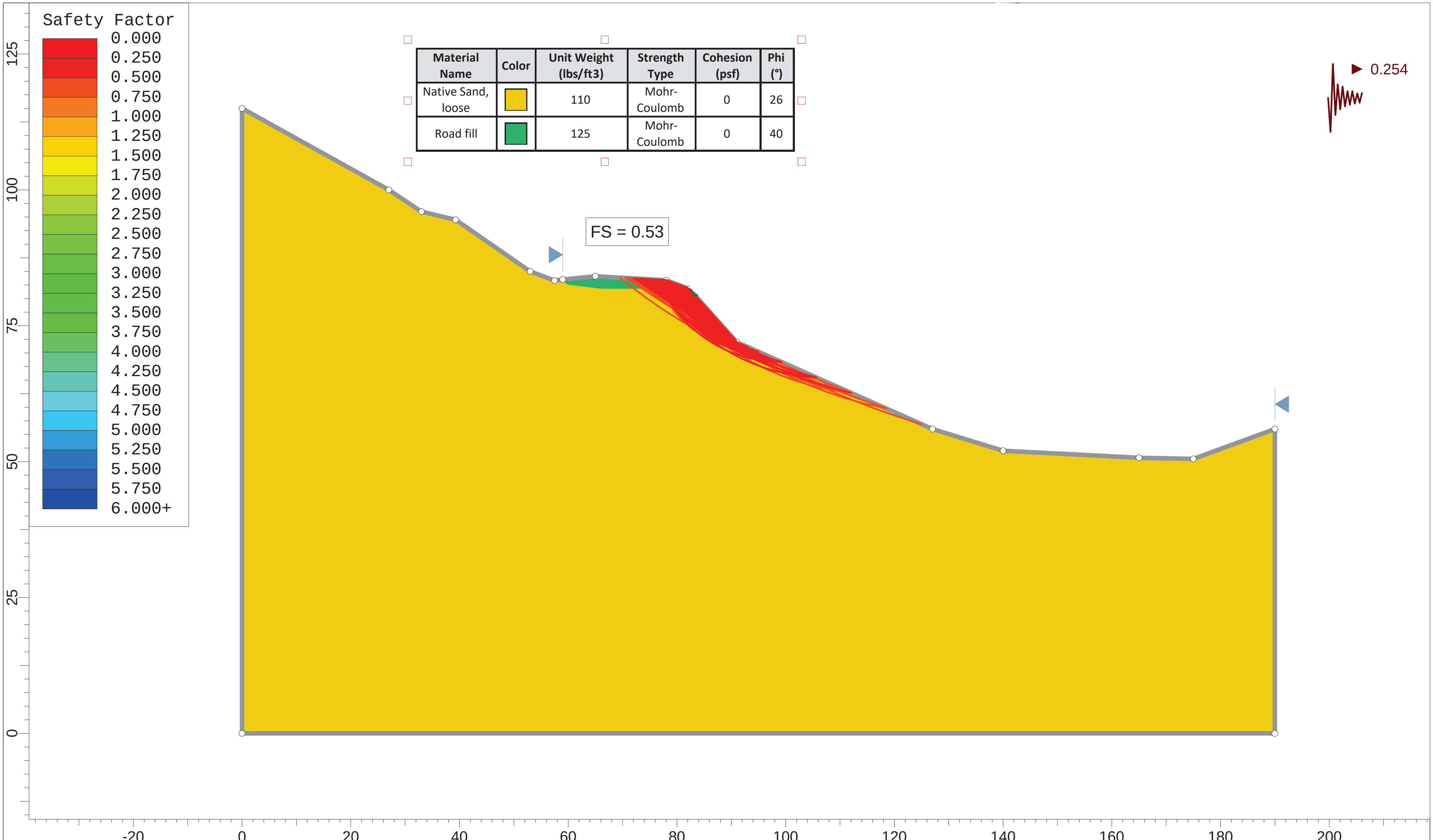


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Native Sand, loose		110	Mohr-Coulomb	0	26
Road fill		125	Mohr-Coulomb	0	40

Manzanita - Cross-Section E
Slope Stability Analysis

Current Condition (Static)

Scale 1:300	3/25/2025
Pali Consulting	Figure B-11



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Native Sand, loose		110	Mohr-Coulomb	0	26
Road fill		125	Mohr-Coulomb	0	40

Manzanita - Cross-Section E

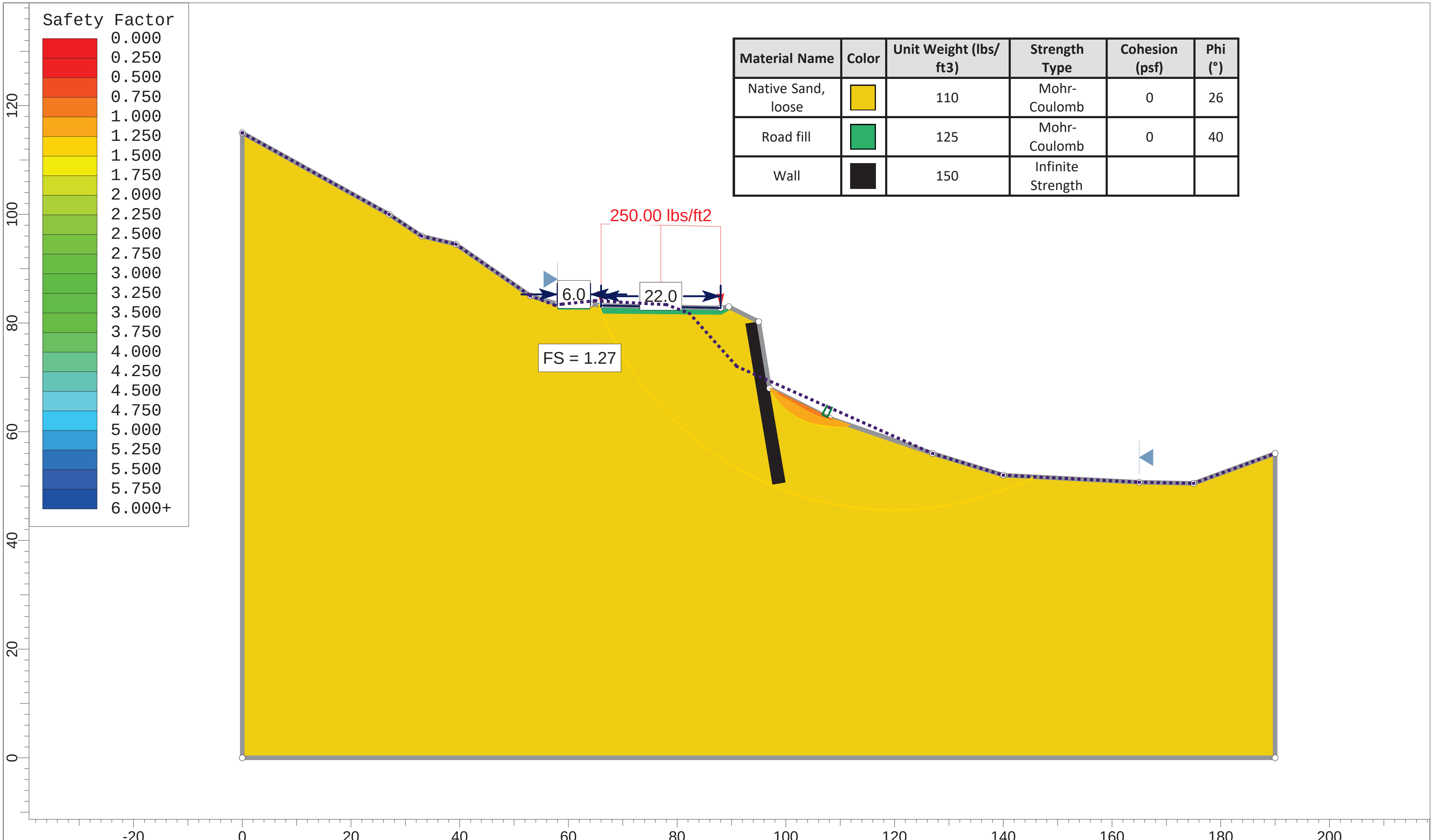
Slope Stability Analysis

Current Condition (Seismic)

Scale 1:300 3/25/2025

Pali Consulting

Figure B-12



Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (°)
Native Sand, loose		110	Mohr-Coulomb	0	26
Road fill		125	Mohr-Coulomb	0	40
Wall		150	Infinite Strength		

Manzanita - Cross-Section E

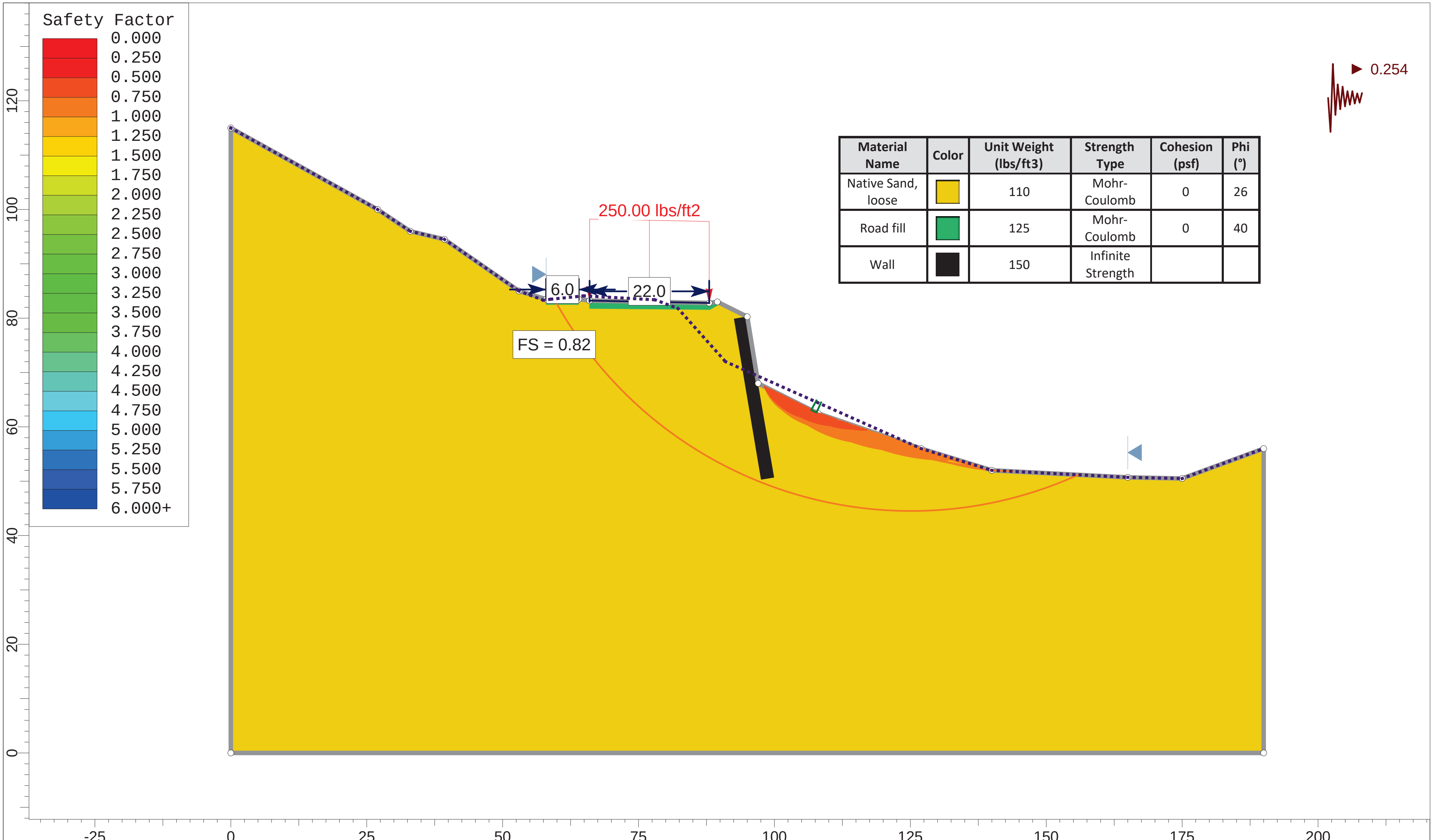
Slope Stability Analysis

90% Plan (Static)

Scale 1:300 | 3/25/2025

Pali Consulting

Figure B-13



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Native Sand, loose		110	Mohr-Coulomb	0	26
Road fill		125	Mohr-Coulomb	0	40
Wall		150	Infinite Strength		

Manzanita - Cross-Section E
Slope Stability Analysis

90% Plan (Seismic)

Scale 1:300 | 3/25/2025

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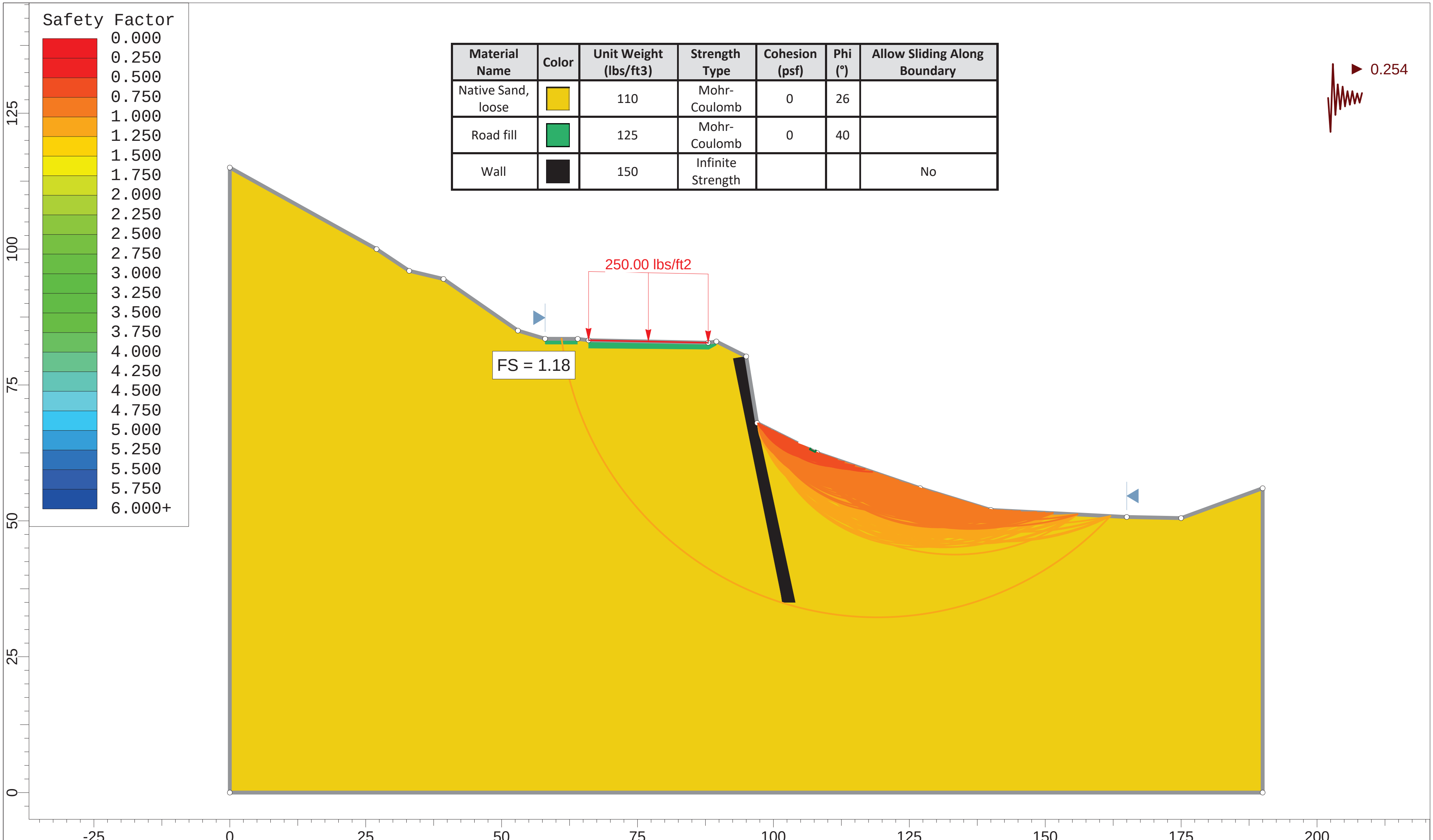
Figure B-14



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Allow Sliding Along Boundary
Native Sand, loose		110	Mohr-Coulomb	0	26	
Road fill		125	Mohr-Coulomb	0	40	
Wall		150	Infinite Strength			No

Manzanita - Cross-Section E
Slope Stability Analysis

Seismic FS >1.0

Scale 1:300	3/25/2025
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