REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Proposed Mixed-Use Development
4225 Highway 101
Lincoln City, Oregon

For
Housing Authority of Lincoln County
September 18, 2023

Project: KemperCo-5-01

September 18, 2023

Housing Authority of Lincoln County
1039 NW Nye Street
Newport, OR 97365

Attention: Dan Butler

Report of Geotechnical Engineering Services
Proposed Mixed-Use Development
4225 Highway 101
Lincoln City, Oregon
Project: KemperCo-5-01

NV5 is pleased to submit this report of geotechnical engineering services for the proposed mixed-use development in Lincoln City, Oregon. Our services were conducted in accordance with our proposal dated June 1, 2023.

We appreciate the opportunity to be of continued service to you. Please contact us if you have questions regarding this report.

Sincerely,

NV5

Jeffery D. Tucker, P.E., G.E.
Principal Engineer

cc: Thomas Kemper, KemperCo, LLC

JCH:JJP:JDT:kt
Attachments
One copy submitted
Document ID: KemperCo-5-01-091823-geor.docx

EXECUTIVE SUMMARY

This section provides a summary of the primary geotechnical considerations associated with the proposed mixed-use development in Lincoln City, Oregon. This summary is an overview and the report should be referenced for a thorough discussion of the subsurface conditions and geotechnical recommendations for the project.

- A significant amount of undocumented fill is present across the west side of the site. The undocumented fill encountered contains varying amounts of small organic debris and organic soil.

- Buildings underlain by undocumented fill can be supported by conventional spread footings on gravel pads if a risk of differential settlement is acceptable. If the risk of differential settlement is not tolerable, the buildings should be supported by conventional spread footings on gravel pads that are underlain by a ground improvement system. In our opinion, rammed aggregate piers are the most suitable and cost-effective improvement method.

- There is a risk for poor performance of floor slabs established directly over undocumented fill. If undocumented fill is present at the proposed finished floor slab elevations, we recommend that the undocumented fill be improved for the upper 12 inches, replaced with imported structural fill, or structural floor slabs be constructed and supported by a ground improvement system.

- Where the subgrade is identified as undocumented fill, the pavement established over the undocumented fill may experience settlement and distress over its design life. We recommend the top 12 inches of pavement subgrade should be improved by replacing it with imported granular structural fill, scarifying and recompacting it, or cement amending the subgrade.

TABLE OF CONTENTS

TABLE OF CONTENTS

APPENDICES (continued)

ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ACP	asphalt concrete pavement
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BGS	below ground surface
CDSM	cement deep soil mixing
DCP	dynamic cone penetrometer
ESAL	equivalent single-axle load
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
LiDAR	light detection and ranging
MCE	maximum considered earthquake
mil	milli-inch
MSL	mean sea level
OSHA	Occupational Safety and Health Administration
OSSC	2021 Oregon Standard Specifications for Construction
pcf	pounds per cubic foot
pci	pounds per cubic inch
PG	performance grade
psf	pounds per square foot
psi	pounds per square inch
SPT	standard penetration test

1.0 INTRODUCTION

NV5 has prepared this geotechnical engineering report for the proposed mixed-use development located at 4225 Highway 101 in Lincoln City, Oregon. Figure 1 shows the site relative to existing topographic and physical features. Figure 2 shows the existing and proposed site layout and our exploration locations. Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

2.0 PROJECT UNDERSTANDING

We understand the proposed development will consist of multiple duplex and triplex residential structures on the east side of the site and a multi-story, wood-framed apartment building and a single-story commercial structure on the west side of the site. Paved parking areas will occupy the central portion of the site, and site access via Highway 101 is proposed at the south edge of the site. Based on our experience with similar projects, we estimate that maximum column and wall loads will not exceed 80 kips and 4 kips per foot, respectively. Site plans were preliminary at the time of this report.

3.0 BACKGROUND

The site is currently occupied by a single-family home and detached garage in the central portion of the site. Based on publicly available LiDAR data, a drainage channel extending north of the property and along the south edge of Highway 101 south of the property was filled in sometime prior to 1994.

4.0 SCOPE OF SERVICES

The purpose of our geotechnical engineering services was to characterize site subsurface conditions and provide geotechnical engineering recommendations for design and construction of the proposed development. Our scope of services included the following:

- Reviewed published geotechnical data and our in-house files for existing information on subsurface conditions in the site vicinity.
- Coordinated and managed the field explorations and testing, including utility locates and scheduling subcontractors and NV5 field staff.
- Drilled seven borings to depths between 9.9 and 36.5 feet BGS.
- Maintained a continuous log of the explorations and collected soil samples at representative intervals.
- Conducted four DCP tests within the proposed paved and parking areas on site.
- Evaluated the DCP results and soil classification results to estimate the resilient modulus of the subgrade soil.
- Conducted the following laboratory tests on soil samples collected from the explorations:
    - Seventeen moisture content determinations in general accordance with ASTM D2216
    - Four particle-size analyses in general accordance with ASTM D1140
    - Two Atterberg limits tests in general accordance with ASTM D4318

- Provided recommendations for site preparation and grading, including temporary and permanent slopes, fill placement criteria, suitability of on-site soil for fill, trench excavation and backfill, subgrade preparation, and wet weather construction.
- Provided foundation support recommendations for the proposed buildings, including preferred foundation type, ground improvement, allowable bearing pressure, lateral resistance parameters, and settlement estimates.
- Provided pavement recommendations, including minimum AC and aggregate base thickness.
- Evaluated groundwater conditions at the site and provided general recommendations for temporary dewatering.
- Provided seismic design recommendations in accordance with the procedures outlined in ASCE 7-16. We have assumed that a site-specific seismic hazard evaluation is not required.
- Prepared this geotechnical engineering report that presents our findings, conclusions, and recommendations.

5.0 SITE CONDITIONS

5.1 GEOLOGIC CONDITIONS

The site is located on the Central Oregon Coast, which resides on the western flank of the Coast Range physiographic province. The area is flanked by ocean beaches to the west and Coast Range uplands to the east. Starting in the early Eocene, subduction of the Farallon Plate against the North American Plate resulted in the accretion of offshore volcanic arcs and associated sedimentary packages along the present-day Oregon Coast. Continued subduction by the Farallon Plate and its remnant, the Juan de Fuca Plate, has resulted in the uplift and erosion of these sediments and the creation of the present-day Coast Range (Orr and Orr, 2012).

Locally, the near-surface geologic unit is mapped as the Eocene Age Yamhill Formation, consisting of massive to thin-bedded siltstone with interspersed thin beds of sandstone. The Yamhill Formation is mapped as dipping gently (approximately 10 to 20 degrees) westward (Snavely et al., 1976).

The Yamhill Formation unconformably overlies subaerial volcanics and associated sedimentary deposits of the Siletz River Volcanics at an estimated depth of 100 to 250 feet BGS (Snavely et al., 1976). For the purposes of this report, the Yamhill Formation should be considered the geologic basement for the site and its surrounds.

5.2 SURFACE CONDITIONS

The site currently consists of an unoccupied, single-family residence surrounded by cleared and semi-forested areas adjacent to Highway 101. The property gently slopes from the north to south, with elevations across the site ranging from approximately 87 to 57 feet MSL.

5.3 SUBSURFACE CONDITIONS

5.3.1 General

We explored subsurface conditions at the site by drilling seven borings (B-1 through B-7) to depths between 9.9 and 36.5 feet BGS. The approximate exploration locations are shown on Figure 2. The exploration logs and laboratory testing results are presented in Appendix A.

In general, the subsurface conditions consist of undocumented fill on the west edge of the site and weathered, fine-grained deposits of the Yamhill Formation below the fill and elsewhere on site.

5.3.2 Undocumented Fill

Undocumented fill was encountered in borings B-5, B-6, and B-7 to depths of 4.5, 14, and 35 feet BGS, respectively. The undocumented fill generally consists of low plasticity clay with variable sand content and layers of sand and gravel. Trace amounts of organic debris (roots and rootlets, woody debris, and grass) were observed throughout. A layer of buried topsoil was observed from 14 to 15.8 feet BGS in boring B-6. The consistency of these deposits generally ranges from medium stiff to stiff, with layers of soft zones.

5.3.3 High Plasticity Silt (Yamhill Formation)

Underlying the topsoil and undocumented fill, all of the borings encountered weathered portions of the underlying Yamhill Formation. These deposits consist of high plasticity silt with trace to minor fine-grained sand, as well as isolated sand interbeds. The consistency of this soil generally ranges from stiff to hard.

5.3.4 Groundwater

Groundwater was encountered in borings B-6 and B-7 at depths of 26.5 and 33 feet BGS, respectively. The depth to groundwater may fluctuate in response to seasonal changes, prolonged rainfall, changes in surface topography, and other factors not observed in this study.

5.4 DCP TESTING

We conducted four DCP tests (DCP-1 through DCP-4) within the proposed paved and parking areas on site to determine the resilient modulus of the subgrade. Our methodology and calculations are presented in Appendix B. Table 1 lists our estimates of the resilient moduli at each test location.

Table 1. Subgrade Moduli Estimated from DCP Testing

Location	Estimated Subgrade Resilient Modulus (psi)
DCP-1	5,870
DCP-2	6,720
DCP-3	4,210
DCP-4	4,200

6.0 DESIGN

6.1 FOUNDATION SUPPORT

6.1.1 General

Based on the results of our explorations and preliminary site layout provided to us, the proposed structures can be constructed on conventional spread footings bearing on 1-foot-thick gravel pads as long as the risk of differential settlement of 1.5 inches over a 50-foot span is acceptable in the areas of the site underlain by undocumented fill. If the risk of differential settlement is not tolerable, buildings that will be founded on undocumented fill should be constructed on conventional spread footings bearing on gravel pads that are underlain by a ground improvement system.

The purpose of ground improvement is to mitigate excessive consolidation settlement beneath the buildings. In our opinion, rammed aggregate piers are the most suitable and cost-effective ground improvement method. CDSM columns may also be an effective method. Both methods are described later in this section. Other ground improvement methods may be applicable but have not been included due to the considerably greater cost. Figure 2 shows the specific proposed buildings where we recommend ground improvement to mitigate the undocumented fill.

6.1.2 Gravel Pads

The gravel pads should extend to a minimum depth of 12 inches below the base of foundations and should consist of imported granular material as described in the “Structural Fill” section. Gravel pad thickness may have to be increased in isolated areas to remove topsoil or potentially existing fill material encountered in the foundation subgrade. The granular pads should extend 6 inches beyond the margins of the foundations for every foot excavated below the foundations’ base grade and should consist of imported granular material. The imported granular material should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557, or until well keyed, as determined by one of our geotechnical staff. It is also acceptable to use stabilization rock (see “Structural Fill” section) for gravel pads. We recommend that a member of our geotechnical staff observe the prepared footing subgrade before placing gravel pads as well.

6.1.3 Dimensions and Capacities

Continuous wall and isolated spread footings should be at least 18 and 24 inches wide, respectively. The bottom of exterior footings should be at least 18 inches below the lowest adjacent exterior grade. The bottom of interior footings should be established at least 12 inches below the base of the slab.

Footings bearing on granular pads overlying native subgrade prepared as recommended above should be sized based on an allowable bearing pressure of 2,500 psf. This is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and may be increased by one-third for short-term loads such as those resulting from wind or seismic forces.

6.1.4 Lateral Resistance

Lateral loads on footings can be resisted by passive earth pressure on the sides of the structure and by friction on the bases of the footings. Our analysis indicates that the available passive earth pressure for footings confined by structural fill or footings constructed in direct contact with the undisturbed native soil or structural fill is 300 pcf. Typically, the movement required to develop the available passive resistance may be relatively large; therefore, we recommend using a reduced passive pressure of 225 pcf equivalent fluid pressure. Adjacent floor slabs, pavement, or the upper 12-inch depth of unpaved areas should not be considered when calculating passive resistance. In addition, in order to rely on the recommended passive resistance, a minimum of 5 feet of horizontal clearance must exist between the face of the footings and any adjacent downslopes.

For footings in contact with crushed rock granular pads, a coefficient of friction equal to 0.45 may be used when calculating resistance to sliding.

6.1.5 Rammed Aggregate Piers

Rammed aggregate piers consist of compacted aggregate columns that reinforce and improve the soil. Rammed aggregate piers typically consist of 2- to 3-foot-diameter drilled piers filled with crushed rock and installed to depths of up to 45 feet BGS. The aggregate is placed in drilled holes or in a driven mandrel in lifts varying between 10 and 16 inches in thickness and compacted using a high-energy hydraulic compaction ram. These systems are proprietary and designed and constructed by a specialty contractor. Conventional spread foundations are placed over the completed rammed aggregate piers. Displacement rammed aggregate piers are recommended for this site, but the specialty contractor should be consulted regarding the installation method. Rammed aggregate piers should extend to the top of the Yamhill Formation.

It may be possible to increase the bearing pressure to between 4,000 and 6,000 psf, as determined by the designer of rammed aggregate piers. A one-third increase in allowable bearing pressure is also typical for such systems when resisting short-term loads such as wind and seismic forces.

6.1.6 CDSM Column Ground Improvement

CDSM columns improve weak soil by mechanically mixing it with cement slurry. A drill equipped with radial mixing paddles located near the bottom of the drill rods is used to mix the cement slurry into the subsurface soil. Slurry is pumped through the drill rods as the drill bit advances, and the soil and slurry are mixed together as the drill bit advances and is withdrawn. The process constructs individual CDSM column elements to increase bearing capacity and decrease settlement. CDSM columns typically vary in diameter between 48 and 72 inches. Spoils generated during installation can be used on site as structural fill or hauled off site. A 12- to 24-inch-thick layer of compacted angular crushed rock is typically placed between the top of the CDSM columns and the bottoms of the foundations to distribute foundation loads to the CDSM columns and provide a working surface for constructing the mat foundation.

CDSM column ground improvement systems can be designed by a design-build contractor. Based on our experience, a mat foundation supported on CDSM column ground improvement can typically be sized using an allowable bearing pressure of 4,000 to 5,000 psf. This can typically be increased by one-third when considering transient loads, such as wind and seismic forces. A typical subgrade reaction modulus value for soil improved with CDSM columns is 150 to 200 pci. The CDSM column system can likely be designed to limit total mat foundation settlement to less than 1 inch. The design-build contractor should be contacted to provide the actual design values they recommend for this project.

If CDSM column ground improvement designed by a contractor is used for this project, we recommend that NV5 be allowed to review the final design and proposed installation methods. A representative of our firm should observe the installation of test columns and quality control testing. We should be present during installation of production columns to confirm that soil conditions are as anticipated. NV5 should also review the data obtained during installation to confirm that the expected design bearing pressure and settlement criteria can be achieved.

6.1.7 Subgrade Observation

All footing subgrades should be evaluated by a representative of NV5 to confirm suitable bearing conditions and the presence of ground improvement systems. Observations should also confirm that loose or soft material, organic material, unsuitable fill, prior topsoil zones, and softened subgrade (if present) have been removed. Localized deepening of footing excavations may be required to penetrate any deleterious material.

6.2 SEISMIC DESIGN CONSIDERATIONS

6.2.1 Seismic Design Parameters

The soil profile of the site is consistent with Site Class D in accordance with ASCE 7-16. The seismic design parameters presented in Table 2 can be used to compute design levels of ground shaking. ASCE 7-16 Section 11.4.8 requires a ground motion hazard study in accordance with Section 21.2 for structures on Site Class D sites with S_1 greater than or equal to 0.2 g (S_1 at the site is 0.679 g). Exception 2 of ASCE 7-16 Section 11.4.8 indicates a ground motion hazard study is not required for structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_S is determined for values of T less than or equal to 1.5 T_S and taken as equal to 1.5 times the value computed in accordance with either $T_L \geq T > 1.5\ T_S$ or $T > T_L$. The structural engineer should evaluate code requirements and exceptions to verify that these parameters can be used for design. If a site response analysis is needed, we can perform this additional analysis.

Table 2. Seismic Design Parameters[1]

Seismic Design Parameter	Short Period (T_s)	1 Second Period (T_1)
MCE Spectral Acceleration	$S_s = 1.311$ g	$S_1 = 0.679$ g
Site Class	D	
Site Coefficient	$F_a = 1.0$	$F_v = 1.7$
Adjusted Spectral Acceleration	$S_{MS} = 1.311$ g	$S_{M1} = 1.154$ g
Design Spectral Response Acceleration Parameters	$S_{DS} = 0.874$ g	$S_{D1} = 0.769$ g

1. Seismic design parameters can be used only if a site-specific analysis is not required.

6.2.2 Seismic Hazards

6.2.2.1 Liquefaction and Lateral Spreading

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the sudden loss of shear strength in a soil. Granular soil, which relies on interparticle friction for strength, is 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 soil with low silt and clay content is the most susceptible to liquefaction. Low plasticity, sandy silt may be moderately susceptible to liquefaction under relatively high levels of ground shaking. Non-plastic and low plasticity, fine-grained material may be subject to cyclic softening from an increase in pore water pressure and a reduction in strength during seismic shaking; however, the relatively poor drainage characteristics of silt deposits inhibit the occurrence of a rapid decrease in volume.

Due to the anticipated groundwater elevation and high plasticity of the predominantly fine-grained soil found at the site, there is a low risk of liquefaction. Consequently, lateral spreading is not considered to be a credible hazard.

6.3 FLOOR SLABS

Undocumented fill is present on the west side of the site. There is a risk for poor performance of floor slabs established directly over undocumented fill. If undocumented fill is present at the proposed finished floor slab elevations, we recommend that the undocumented fill be improved for the upper 12 inches, replaced with imported structural fill, or structural floor slabs be constructed and supported by a ground improvement system. Ground improvement systems are presented in the "Foundation Support" section.

We anticipate maximum slab loading will be up to 150 psf. Satisfactory subgrade support for slabs-on-grade is possible, provided the slab areas are prepared as described in this report. Subgrade preparation should include improvement of any unsuitable soil as recommended in the "Site Preparation" section. Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Load-bearing concrete slabs established over the Yamhill Formation may be designed assuming a modulus of subgrade reaction (k) of 400 pci. The subgrade modulus for slabs constructed over ground improvement systems will vary and should be provided by the designer.

We recommend a minimum 6-inch-thick layer of imported granular material be placed and compacted over the prepared soil subgrade. Imported granular material placed beneath building floor slabs should meet the requirements for floor slab base rock, as described in the "Structural Fill" section. The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

The near-surface native soil is fine grained and will tend to maintain a high moisture content. In areas where moisture-sensitive floor slab and flooring will be installed, installation of a vapor barrier is warranted in order to reduce the potential for moisture transmission through and efflorescence growth on the slab and flooring. In addition, flooring manufacturers often require vapor barriers to protect flooring and flooring adhesives and will warrant their product only if a vapor barrier is installed according to their recommendations. If the project includes highly moisture-sensitive flooring, we recommend 10- or 15-mil vapor barriers, which are often required by flooring manufacturers. Selection and design of an appropriate vapor barrier should be based on discussions among members of the design team.

We note that foundation drains are recommended in cut areas as discussed in the "Drainage" section.

6.4 RETAINING STRUCTURES

6.4.1 Assumptions

Our retaining wall design recommendations are based on the following assumptions: (1) the walls are cantilevered walls, (2) the walls are less than 15 feet in height, (3) drainage is provided behind walls, and (4) the ground surface at the toe of the wall has an inclination flatter than 5H:1V. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

6.4.2 Wall Design Parameters

Permanent retaining structures free to rotate slightly around the base should be designed for active earth pressures using an equivalent fluid unit pressure of 35 pcf. If retaining walls are restrained against rotation during backfilling, they should be designed for an at-rest earth pressure of 55 pcf.

Seismic lateral forces can be calculated using a dynamic force equal to $7H^2$ pounds per linear foot of wall, where H is the wall height. The seismic force should be applied as a distributed load with the centroid located at 0.6H from the base of the wall. Footings for retaining walls should be designed as recommended for shallow foundations.

The design equivalent fluid pressure should be increased for walls that retain sloping soil. We recommend the above lateral earth pressures be increased using the factors presented in Table 3 when designing walls that retain sloping soil.

Table 3. Lateral Earth Pressure Increase Factors for Sloping Soil

Slope of Retained Soil (degrees)	Lateral Earth Pressure Increase Factor
0	1.00
5	1.06
10	1.12
20	1.33
25	1.52
30	2.27

If other surcharges (e.g., slopes steeper than 2H:1V, foundations, vehicles, etc.) are located within a horizontal distance of twice the height of the wall from the back of the wall, additional pressures will need to be accounted for in the wall design. Our office should be contacted for appropriate wall surcharges based on the actual magnitude and configuration of the applied loads.

6.4.3 Wall Drainage and Backfill

The above design parameters have been provided assuming drains will be installed behind walls to prevent hydrostatic pressures from developing. If a drainage system is not installed, our office should be contacted for revised design forces.

Backfill material placed behind the walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of retaining wall select backfill placed and compacted in conformance with the “Structural Fill” and “Fill Placement and Compaction” section.

A minimum 6-inch-diameter, perforated collector pipe should be placed at the base of the walls. The pipe should be embedded in a minimum 2-foot-wide zone of angular drain rock that is wrapped in a drainage geotextile fabric and extends up the back of the wall to within 1 foot of the finished grade. The drain rock and drainage geotextile fabric should meet the specifications provided in the “Materials” section. The perforated collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe(s) should not be tied directly into stormwater drain systems, unless measures are taken to prevent backflow into the wall’s drainage system.

6.4.4 Construction Considerations

Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flatwork adjacent to retaining walls be postponed at least four weeks after backfilling of the wall, unless survey data indicates that settlement is complete prior to that time.

6.5 PAVEMENT

Pavement should be installed on native subgrade, new engineered fill, or cement-amended subgrade prepared in conformance with the “Construction” section.

Our AC pavement recommendations are based on the following assumptions:

- Design subgrade resilient modulus of 5,200 psi was selected based on DCP testing results.
- Resilient modulus of 20,000 psi was assumed for the aggregate base layer.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability of 85 percent and standard deviation of 0.49.
- Structural coefficients of 0.42 and 0.10 for the AC and aggregate base, respectively.
- A 20-year design life with no growth.
- Truck traffic will consist of two-axle trucks.

Design traffic loading was not available at the time of this report. Based on our experience with similar projects, we assumed that loading in light traffic areas will consist of up to 250 passenger vehicles per day and loading in heavy traffic areas will consist of up to 20 passenger vehicles and 3 two- to three-axle trucks (delivery and garbage trucks) per day. If any of these assumptions vary from project design values, our office should be contacted with the appropriate information so that the pavement designs can be revised. Our AC pavement design recommendations are summarized in Table 4.

Table 4. Pavement Section Thickness

Pavement Use	Average Daily Trucks	ESALs	AC Thickness[1] (inches)	Aggregate Base Thickness[1] (inches)	Aggregate Base Thickness Over Cement-Amended Subgrade[1] (inches)
Passenger vehicles only	0	<500	3.0 (one lift)	8.0	4.0
Heavy traffic	3	6,000	4.0 (two, 2.0-inch-thick lifts)	10.0	5.0

1. All thicknesses are intended to be the minimum acceptable values.

All recommended pavement thicknesses are intended to be the minimum acceptable. Pavement design is based on the assumption that construction will be completed during an extended period of dry weather. Wet weather construction could require an increased thickness of aggregate base as discussed in the “Subgrade Considerations” section.

Construction traffic should be limited to non-building, unpaved portions of the site or haul roads. Construction traffic should not be allowed on new pavement. If construction traffic is to be allowed on newly constructed road sections, an allowance for this additional traffic will need to be made in the design pavement section.

The AC and aggregate base should meet the requirements outlined in the “Materials” section.

6.6 DRAINAGE

6.6.1 Surface

Where possible, the finished ground surface around structures should be sloped away from the structures at a minimum 2 percent gradient for a distance of at least 5 feet. Downspouts or roof scuppers should discharge into a storm drain system that carries the collected water to an appropriate stormwater system. Trapped planter areas should not be created adjacent to buildings without providing means for positive drainage (e.g., swales or catch basins).

6.6.2 Foundation Drains

We recommend that perimeter foundation drains be installed in all areas where finished floor elevations will be more than 2 feet below existing grade. Foundation drains should be constructed at a minimum slope of approximately ½ percent and pumped or drained by gravity to a suitable discharge. The perforated drainpipe should not be tied to a stormwater drainage system without backflow provisions. Foundation drains should consist of 4-inch-diameter, perforated drainpipe embedded in a minimum 2-foot-wide zone of crushed drain rock that extends to the ground surface. The invert elevation of the drainpipe should be installed at least 18 inches below the elevation of the floor slab.

Perforated collector pipes for subsurface drains should be routed to a suitable discharge point at an appropriate location away from buildings. The discharge pipes should not be connected into the stormwater system or to other sub-drain systems, unless means for backflow prevention are installed.

The drain rock and geotextile should meet the requirements specified in the "Materials" section. The drain rock and geotextile should extend up the side of embedded walls to within a foot of the ground surface, geotextile wrapped over the top of the drain rock, as recommended in the "Retaining Structures" section.

7.0 CONSTRUCTION

7.1 SITE PREPARATION

7.1.1 Demolition

Demolition includes removal of the existing pavement, concrete curbs and sidewalks, and utilities that may be present underneath areas to be improved. Underground vaults, tanks, manholes, foundation elements, and other subsurface structures should be removed in areas of new foundation elements. Utility lines can be completely removed or grouted full if left in place. Soil disturbed during demolition should be removed and replaced in accordance with the "Structural Fill" section.

Material generated during demolition should be transported off site for disposal or stockpiled in areas designated by the owner. In general, this material will not be suitable for reuse as engineered fill. However, AC, concrete, and base rock material may be recycled in accordance with the "Structural Fill" section.

7.1.2 Grubbing, and Stripping

Trees and shrubs should be removed from development areas. In addition, root balls should be grubbed out to the depth of the roots, which could exceed 3 feet BGS. Depending on the methods used to remove root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm, undisturbed subgrade. The resulting excavations should be backfilled with structural fill.

Any existing topsoil zone should be stripped and removed from all fill areas. We anticipate that the depth of stripping will range from 2 to 6 inches, although greater stripping depths may be required to remove localized zones of loose or organic soil. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaped areas.

7.1.3 Subgrade Evaluation

Upon completion of stripping and subgrade stabilization and prior to the placement of fill or other improvements, the exposed subgrade should be evaluated by proof rolling. The subgrade should be proof rolled with a fully loaded dump truck or similarly heavy, rubber tire construction equipment to identify soft, loose, or unsuitable areas. A member of our geotechnical staff should observe proof rolling to evaluate yielding of the ground surface. During wet weather, subgrade evaluation should be performed by probing with a foundation probe rather than proof rolling.

Areas containing undocumented fill or where loose/soft or otherwise unsuitable soil is identified during subgrade evaluation, should be improved by scarifying and re-compacting (dry weather only), replacing with imported granular material in accordance with the "Structural Fill" and "Fill Placement and Compaction" sections, or by cement amending the soil in accordance with the "Cement Amendment" section. Scarifying and re-compacting the surficial soil may require that the soil be dried, which is only possible in the dry summer months.

7.2 SUBGRADE CONSIDERATIONS

The fine-grained soil present on this site is easily disturbed. If not carefully executed, site preparation, utility trench work, and other earthwork activities can create extensive soft areas and significant repair costs can result. Earthwork planning, regardless of the time of year, should include considerations for minimizing subgrade disturbance. Subgrade protection will be critical during the wet season.

If construction occurs during or extends into the wet season or if the moisture content of the surficial soil is more than a couple percentage points above optimum, site stripping and cutting may need to be accomplished using track-mounted equipment. The use of granular haul roads and staging areas will be necessary for support of construction traffic during the rainy season or when the moisture content of the surficial soil is more than a few percentage points above optimum. This design base rock thickness for slabs and pavement will likely not support construction traffic. If construction is planned for periods when the subgrade soil is wet, staging and haul roads with increased thicknesses of base rock will be required. The amount of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor's sequencing of a project and the type/frequency of construction equipment and should, therefore, be the responsibility of the contractor. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul roads areas. The contractor should also be responsible for selecting the type of material for construction of haul roads and staging areas. A geotextile fabric can be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic to help prevent silt/clay migration into the base rock. The imported granular material, stabilization material, and geotextile fabric should meet the specifications in the "Materials" section.

As an alternative to thickened crushed rock sections, the subgrade can be cement amended to provide wet weather protection from construction traffic. The cement-amended subgrade should be covered by at least 4 inches of granular fill material. This recommendation is based on an assumed minimum unconfined compressive strength of 100 psi for subgrade amended to a depth of 12 to 16 inches. The actual thickness of the amended material and imported granular material will depend on the contractor’s means and methods and, accordingly, should be the contractor’s responsibility. Cement amendment is discussed in the “Materials” section.

7.3 PERMANENT SLOPES

Permanent cut and fill slopes should not exceed 2H:1V. Access roads and pavement should be located at least 5 feet from the top of new cut and fill slopes. The setback should be increased to 10 feet for buildings. All exposed slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

7.4 EXCAVATION

7.4.1 General

Conventional earthmoving equipment should be capable of excavating the on-site fill soil. Excavation will be considerably more difficult in the Yamhill Formation and may require special equipment such as hydraulic breakers. Sloughing and caving of the undocumented fill soil may occur in excavations left open for extended periods of time. Accordingly, the contractor should expect to flatten excavations or shore excavations as described below where sloughing or caving occurs. In addition to safety considerations, caving and loss of ground will increase backfill volumes and can result in damage to adjacent structures or utilities.

7.4.2 Excavation and Shoring

Temporary excavation sidewalls should stand vertical to a depth of approximately 4 feet, provided groundwater seepage is not observed in the sidewalls. Open excavation techniques may be used to excavate trenches with depths between 4 and 8 feet, provided the walls of the excavation are cut at a slope of 1H:1V and groundwater seepage is not present. At this inclination, the slopes may ravel and require some ongoing repair. Excavations should be flattened if excessive sloughing or raveling occurs. In lieu of large and open cuts, approved temporary shoring may be used for excavation support. A wide variety of shoring and dewatering systems are available. Consequently, we recommend the contractor be responsible for selecting the appropriate shoring and dewatering systems.

If box shoring is used, it should be understood that box shoring is a safety feature used to protect workers and does not prevent caving. If the excavations are left open for extended periods of time, caving of the sidewalls will likely occur. The presence of caved material will limit the ability to properly backfill and compact the trenches. The contractor should be prepared to fill voids between the box shoring and the sidewalls of the trenches with sand or gravel before caving occurs.

If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation.

7.4.3 Trench Dewatering

If perched groundwater is encountered in excavations, it can likely be dewatered using sumps and pumps. More intense use of pumps may be required at certain times of the year and where more intense seepage occurs. Dewatering systems are best designed by the contractor. Removed water should be routed to a suitable discharge point.

We note that these recommendations are for guidance only. Dewatering of excavations is the sole responsibility of the contractor, as the contractor is in the best position to select these systems based on their means and methods.

7.4.4 Safety

All excavations should be made in accordance with applicable OSHA requirements and regulations of the state, county, and local jurisdiction. While this report describes certain approaches to excavation and dewatering, the contract documents should specify that the contractor is responsible for selecting excavation and dewatering methods, monitoring the excavations for safety, and providing shoring (as required) to protect personnel and adjacent structural elements.

7.5 MATERIALS

7.5.1 Structural Fill

7.5.1.1 General

Fill should be placed on subgrade that has been prepared in conformance with the “Site Preparation” section. A variety of materials may be used as structural fill at the site. However, all material used as structural fill should be free of organic material or other unsuitable material and particles over 6 inches in diameter and should meet the specifications provided in OSSC 00330.12 (Borrow Material). A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill are provided below.

7.5.1.2 On-Site Soil

The material at the site should be suitable for use as general structural fill, provided it is properly moisture conditioned and free of debris, organic material, and particles over 6 inches in diameter. Drying the on-site soil will take a significant amount of time and effort as well as a broad area to spread across. It will require almost constant tilling and extended periods of dry and warm weather. It will be difficult, if not impossible, to adequately compact on-site soil during the rainy season or during prolonged periods of rainfall. On-site soil should only be used as structural fill during the dry summer months. When used as structural fill, the on-site fine-grained soil should be placed in lifts with a maximum uncompacted thickness of 8 inches and compacted to not less than 92 percent of the maximum dry density, as determined by ASTM D1557.

7.5.1.3 Imported Granular Material

Imported granular material used as structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The imported granular material should also be durable, angular, and fairly well graded between coarse and fine material; should have less than 5 percent fines (material passing the U.S. Standard No. 200 sieve) by dry weight; and should have at least two mechanically fractured faces.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. During the wet season or when wet subgrade conditions exist, the initial lift should be approximately 18 inches in uncompacted thickness and should be compacted by rolling with a smooth-drum roller without using vibratory action.

7.5.1.4 Stabilization Material

Stabilization material used in staging or haul road areas or in trenches should consist of durable, 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The material should have a maximum particle size of 6 inches, should have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and should have at least two mechanically fractured faces. The material should be free of organic material and other deleterious material. Stabilization material should be placed in lifts between 12 and 24 inches thick and compacted to a firm condition.

7.5.1.5 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of durable, well-graded granular material with a maximum particle size of 1½ inches, should have less than 7 percent fines by dry weight, and should have at least two mechanically fractured faces. The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of durable, well-graded granular material with a maximum particle size of 2½ inches; should have less than 7 percent fines by dry weight; and should have at least two mechanically fractured faces. This material should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department. The upper 3 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557.

Outside of structural improvement areas (e.g., roadway alignments or building pads), trench backfill placed above the pipe zone may consist of general fill material that is free of organic material and material over 6 inches in diameter. This general trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

7.5.1.6 Aggregate Base

Imported granular material used as base rock for building floor slabs and pavement should consist of ¾- or 1½-inch-minus material (depending on the application). In addition, the aggregate should have less than 5 percent fines by dry weight and have at least two mechanically fractured faces. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

7.5.1.7 Retaining Wall Select Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of imported granular material as described above and should have less than 7 percent fines by dry weight. We recommend the wall backfill be separated from general fill, native soil, and/or topsoil using a geotextile fabric that meets the specifications provided below for drainage geotextiles.

The wall backfill should be compacted to a minimum of 95 percent of the maximum dry density, as determined by ASTM D1557. However, backfill located within a horizontal distance of 3 feet from a retaining wall should only be compacted to approximately 90 percent of the maximum dry density, as determined by ASTM D1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor). If flatwork (sidewalks or pavement) will be placed atop the wall backfill, we recommend that the upper 2 feet of material be compacted to 95 percent of the maximum dry density, as determined by ASTM D1557.

7.5.1.8 Drain Rock Material

Drain rock should consist of angular, granular material with a maximum particle size of 2 inches. The material should be free of roots, organic material, and other unsuitable material; should have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis); and should have at least two mechanically fractured faces. Drain rock should be compacted to a well-keyed, firm condition.

7.5.1.9 Retaining Wall Leveling Pad

Imported granular material placed at the base of retaining wall footings should consist of select granular material. The granular material should be 1”-0 to ¾”-0 aggregate size and have at least two mechanically fractured faces. The leveling pad material should be placed in a 6- to 12-inch-thick lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

7.5.1.10 Recycled On-Site Material

AC and conventional concrete from demolished on-site structures may be used as fill if it is processed to meet the requirements for its intended use. Processing includes crushing and screening, grinding in place, or other methods to meet the requirements for structural fill as described above. The processed material should be fairly well graded and not contain metal, organic material, or other deleterious material. The processed material may be mixed with on-site soil or imported fill to assist in achieving the gradation requirements. We recommend that processed recycled fill have the maximum particles size as listed in Table 5.

Table 5. Processed Fill Maximum Particle Size

Depth of Placement[1]	Maximum Particle Size
0 feet to 1 foot	Not recommended
1 foot to 2 feet	2 inches
2 to 6 feet	4 inches
6 to 10 feet	8 inches
deeper than 10 feet	12 inches

1. Below subgrade of structural element

Recycled on-site fill material should not be used within a depth of 1 foot from foundations, floor slabs, pavement, or other subsurface elements. We also caution that excavation through recycled material that is placed as structural fill may be difficult if it has a significant fraction of oversized particles. In addition, these excavations may also be prone to raveling and caving.

7.5.2 Geotextile Fabric

7.5.2.1 Subgrade Geotextile

Subgrade geotextile should conform to OSSC Table 02320-4 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles. All drainage aggregate and stabilization material should be underlain by a subgrade geotextile.

7.5.2.2 Drainage Geotextile

Drainage geotextile should conform to Type 2 material of OSSC Table 02320-1 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

7.5.3 AC

7.5.3.1 ACP

The AC should be Level 2, ½-inch, dense ACP according to OSSC 00744 (Asphalt Concrete Pavement) and compacted to 91 percent of the theoretical maximum density of the mix, as determined by AASHTO T 209. The minimum and maximum lift thicknesses are 2 and 3.5 inches, respectively, for ½-inch ACP. Asphalt binder should be performance graded and conform to PG 64-22.

7.5.3.2 Cold Weather Paving Considerations

In general, AC paving is not recommended during cold weather (surface temperatures less than 40 degrees Fahrenheit). Compacting under these conditions can result in low compaction and premature pavement distress.

Each AC mix design has a recommended compaction temperature range that is specific for the particular AC binder used. In colder temperatures, it is more difficult to maintain the temperature of the AC mix as it can lose heat while stored in the delivery truck, as it is placed, and in the time between placement and compaction. In Oregon, the surface temperature during paving should be at least 40 degrees Fahrenheit for lift thickness greater than 2.5 inches and at least 50 degrees Fahrenheit for lift thickness between 2 and 2.5 inches.

If paving activities must take place during cold weather construction as defined above, the project team should be consulted and a site meeting should be held to discuss ways to lessen low compaction risks.

7.5.4 Cement Amendment

7.5.4.1 General

As an alternative to the use of imported granular material for subgrade improvement, an experienced contractor may be able to amend the on-site soil with portland cement to obtain suitable support properties. Successful use of soil amendment depends on the use of correct mixing techniques, soil moisture content, and amendment quantities. Cement amendment should not be used if runoff during construction cannot be directed away from adjacent wetlands.

7.5.4.2 Subbase Stabilization

We recommend a target strength for cement-amended subgrade for building and pavement subbase (below aggregate base) soil of 100 psi. Successful use of soil amendment depends on use of correct techniques and equipment, soil moisture content, and the amount of cement added to the soil. The recommended percentage of cement is based on soil moisture contents at the time of placing the structural fill. Based on our experience, 6 percent cement by weight of dry soil is generally satisfactory when the soil moisture content does not exceed approximately 25 percent. If the soil moisture content is in the range of 25 to 35 percent, 7 to 9 percent by weight of dry soil is recommended. It is difficult to accurately predict field performance due to the variability in soil response to cement amendment. The amount of cement added to the soil may need to be adjusted based on field observations and performance. Moreover, depending on the time of year and moisture content levels during amendment, water may need to be applied during tilling to appropriately condition the soil moisture content. The amount of cement used during amendment should be based on an assumed soil dry unit weight of 110 pcf. For preliminary design purposes, we recommend a minimum of 6 percent cement. It is not possible to amend soil during heavy or continuous rainfall. Work should be completed during suitable conditions.

We recommend cement-spreading equipment be equipped with balloon tires to reduce rutting and disturbance of the fine-grained soil. A static sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds should be used for initial compaction of the fine-grained soil. A smooth-drum roller with a minimum applied linear force of 700 pounds per inch should be used for final compaction.

A minimum curing time of four days is required between amendment and construction traffic access. Construction traffic should not be allowed on unprotected, cement-amended subgrade. To protect the cement-amended surfaces from abrasion or damage, the finished surface should be covered with 4 to 6 inches of imported granular material.

Amendment depths for building/pavement, haul roads, and staging areas are typically on the order of 12, 16, and 12 inches, respectively. The crushed rock typically becomes contaminated with soil during construction. Contaminated base rock should be removed and replaced with clean rock in pavement areas. The actual thickness of the amended material and imported granular material for haul roads and staging areas will depend on the anticipated traffic, as well as the contractor's means and methods and, accordingly, should be the contractor's responsibility. Cement amendment should not be attempted when the air temperature is below 40 degrees Fahrenheit or during moderate to heavy precipitation. Cement should not be placed when the ground surface is saturated or standing water exists.

7.5.4.3 Cement-Amended Structural Fill

On-site silt soil that is not suitable for structural fill due to high moisture content may be amended and placed as fill over a subgrade prepared in conformance with the "Site Preparation" section. Cement-amended fill lift thicknesses should be limited to 12 inches. The cement ratio for general cement-amended fill can generally be reduced by 1 percent (by dry weight). Typically, a minimum curing time of four days is required between amendment and construction traffic access. Consecutive lifts of fill may be amended immediately after the previous lift has been amended and compacted (e.g., the four-day wait period does not apply). However, where the final lift of fill is a building or roadway subgrade, the four-day wait period is in effect for the final lift of cement-amended soil.

7.5.4.4 Other Considerations

Portland cement-amended soil is hard and has low permeability. This soil does not drain well and it is not suitable for planting. Future planted areas should not be cement amended, if practical, or accommodations should be made for drainage and planting. Moreover, cement amending soil within building areas must be done carefully to avoid trapping water under floor slabs. We should be contacted if this approach is considered. Cement amendment should not be used if runoff during construction cannot be directed away from adjacent wetlands (if any). Cement amendment runoff should be collected, monitored, and treated in accordance with Oregon Department of Environmental Quality requirements prior to being discharged.

7.5.4.5 Specification Recommendations

We recommend that the following comments be included in the specifications for the project:

- In general, cement amendment is not recommended during cold weather (temperatures less than 40 degrees Fahrenheit) or during rainfall.
- Mixing Equipment
  - Use a pulverizer/mixer capable of uniformly mixing the cement into the soil to the design depth. Blade mixing will not be allowed.
  - Pulverize the soil-cement mixture such that 100 percent by dry weight passes a 1-inch sieve and a minimum of 70 percent passes a No. 4 sieve, exclusive of gravel or stone retained on these sieves. If water is required, the pulverizer should be equipped to inject water to a tolerance of ¼ gallon per square foot of surface area.
  - Use machinery that will not disturb the subgrade, such as using low-pressure "balloon" tires on the pulverizer/mixer vehicle. If subgrade is disturbed, the tilling/amendment depth shall extend the full depth of the disturbance.
  - Multiple "passes" of the tiller may be required to adequately blend the cement and soil mixture.

- Spreading Equipment
  - Use a spreader capable of distributing the cement uniformly on the ground to within 5 percent variance of the specified application rate.
  - Use machinery that will not disturb the subgrade, such as using low-pressure "balloon" tires on the spreader vehicle. If subgrade is disturbed, the tilling/amendment depth shall extend the full depth of the disturbance.
- Compaction Equipment
  - Use a static, sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds for initial compaction of fine-grained soil (silt and clay) or an alternate approved by the geotechnical engineer.

7.6 FILL PLACEMENT AND COMPACTION

Fill soil should be compacted at a moisture content that is within 3 percent of optimum. The maximum allowable moisture content varies with the soil gradation and should be evaluated during construction. Fill and backfill material should be placed in uniform, horizontal lifts and compacted with appropriate equipment. The maximum lift thickness will vary depending on the material and compaction equipment used but should generally not exceed the loose thicknesses provided in Table 6. Fill material should be compacted in accordance with the compaction criteria provided in Table 7.

Table 6. Recommended Uncompacted Lift Thickness

Compaction Equipment	Recommended Uncompacted Lift Thickness (inches)		
	Silty/Clayey Soil	Granular and Crushed Rock Maximum Particle Size ≤ 1½ Inches	Crushed Rock Maximum Particle Size > 1½ Inches
Hand tools: Plate compactor and jumping jack	4 to 8	4 to 8	Not recommended
Rubber tire equipment	6 to 8	10 to 12	6 to 8
Light roller	8 to 10	10 to 12	8 to 10
Heavy roller	10 to 12	12 to 18	12 to 16
Hoe pack equipment	12 to 16	18 to 24	18 to 24

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

Table 7. Compaction Criteria

Fill Type	Compaction Requirements in Structural Zones Percent Maximum Dry Density Determined by ASTM D1557		
	0 to 2 Feet Below Subgrade (percent)	Greater Than 2 Feet Below Subgrade (percent)	Pipe Zone (percent)
Area fill (granular)	95	95	-----
Area fill (fine grained)	92	92	-----
Aggregate base	95	95	-----
Trench backfill[1,2]	95	92	90[1,2]
Retaining wall backfill	95[3]	92[3]	-----

1. Trench backfill above the pipe zone in non-structural areas should be compacted to 85 percent.
2. Or as recommended by the pipe manufacturer.
3. Should be reduced to 90 percent within a horizontal distance of 3 feet from the retaining wall.

7.7 EROSION CONTROL

The site soil is susceptible to erosion; therefore, erosion control measures should be carefully planned and in place before construction begins. Surface water runoff should be collected and directed away from slopes to prevent water from running down the slope face. Erosion control measures (such as straw bales, sediment fences, and temporary detention and settling basins) should be used in accordance with local and state ordinances.

8.0 OBSERVATION OF CONSTRUCTION

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

We recommend that NV5 be retained to observe earthwork activities, including stripping, proof rolling of the subgrade and repair of soft areas, footing subgrade and gravel pad preparation, final proof rolling of the pavement subgrade and base rock, and AC placement and compaction, and performing laboratory compaction and field moisture-density testing.

9.0 LIMITATIONS

We have prepared this report for use by the Housing Authority of Lincoln County and members of the design and construction teams for the proposed project. The data and report can be used for bidding or estimating purposes, but our report, conclusions, and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other nearby building sites.

Exploration observations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The site development plans and design details were preliminary at the time this report was prepared. When the design has been finalized and if there are changes in the site grades, location, or configuration; design loads; or type of construction, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

The scope does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in this report for consideration in design.

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

◆ ◆ ◆

We appreciate the opportunity to be of service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

NV5

John C. Hook, R.G.
Associate Geologist

Jeffery D. Tucker, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
74942PE
OREGON
July 13, 2004
JEFFERY D. TUCKER
EXPIRES: 6/30/24

REFERENCES

ASCE, 2016. *Minimum Design Loads and Associated Criteria for Buildings and Other Structures.* ASCE Standard ASCE/SEI 7-16.

ODOT, 2021. *Oregon Standard Specifications for Construction,* Oregon Department of Transportation, 2021 Edition.

Orr, E.L., and W.N. Orr, 2012, Oregon Geology: Oregon State University Press, 6th Edition; 304 p.

Snavely, P.D., Jr., MacLeod, N.S., Wagner, H.C., and Rau, W.W., 1976, Geologic map of the Cape Foulweather and Euchre Mountain quadrangles, Lincoln County, Oregon: Oregon Department of Geology and Mineral Industries, Miscellaneous Investigations Series Map I-868, 1:62,500 scale.

FIGURES

VICINITY MAP BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®

KEMPERCO-5-01

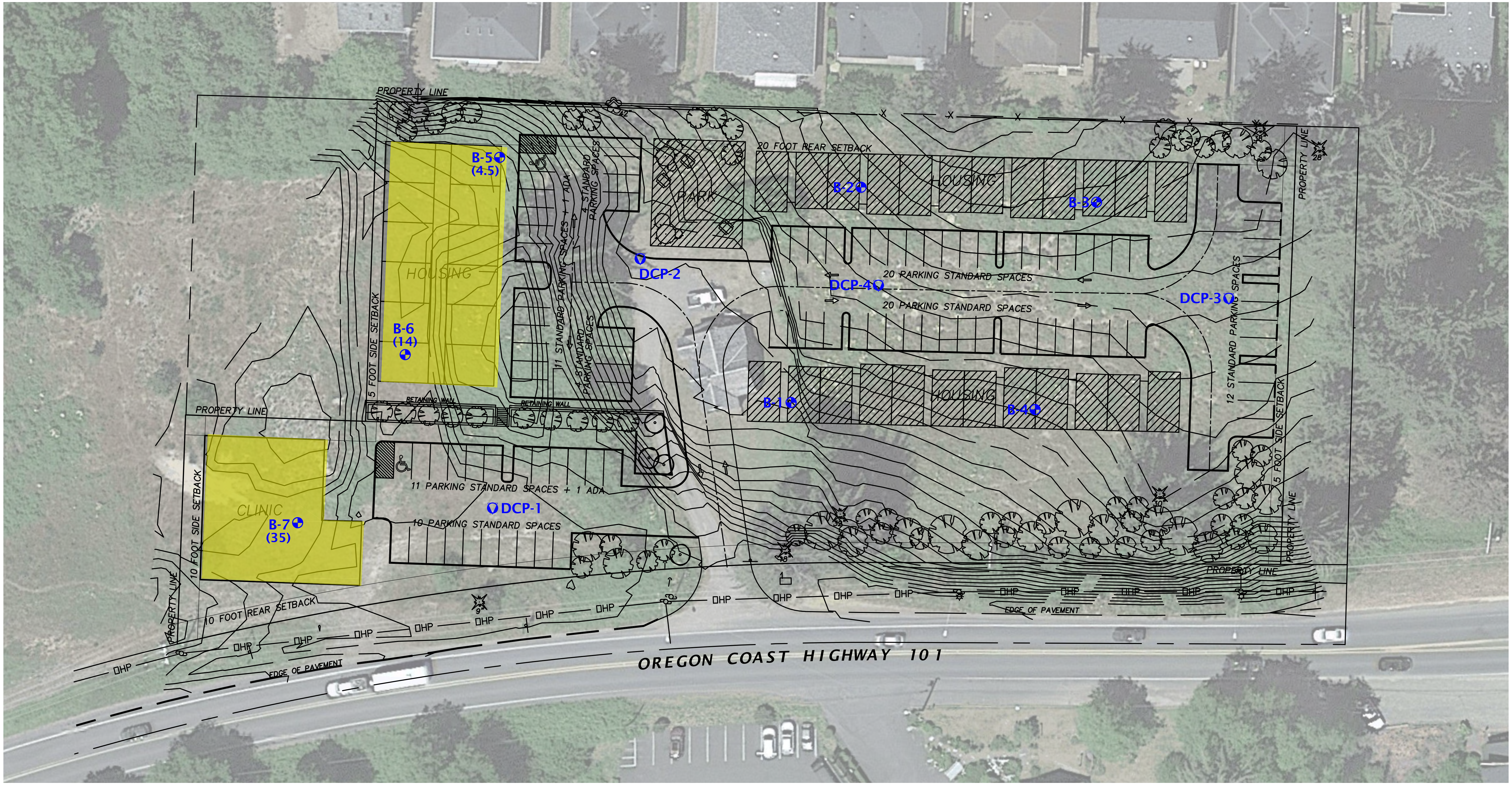
SEPTEMBER 2023

VICINITY MAP

PROPOSED MIXED-USE DEVELOPMENT
LINCOLN CITY, OR

FIGURE 1

Printed By: Mike.Miller | Print Date: 9/18/2023 3:28:08 PM
File Name: J:\E-L\KemperCo\KemperCo-5\KemperCo-5-01\Figures\CAD\KemperCo-5-01-VM01.dwg | Layout: FIGURE 1



LEGEND:

B-1 ⊕	BORING
DCP-1 ⊕	DCP TEST
(4.5)	THICKNESS OF FILL (FEET)
🟨	GROUND IMPROVEMENTS ARE RECOMMENDED FOR THESE BUILDINGS

NOTES:

1. SITE PLAN BASED ON PDF OBTAINED FROM KEMPER CO, LLC.
2. AERIAL PHOTOGRAPH DATED JULY 24, 2019, OBTAINED FROM GOOGLE EARTH PRO.

FIGURE 2

SITE PLAN

PROPOSED MIXED-USE DEVELOPMENT
LINCOLN CITY, OR

KEMPERCO-5-01

SEPTEMBER 2023

NV5

Printed By: Mike.Miller | Print Date: 9/18/2023 3:29:14 PM
File Name: J:\E-L\KemperCo\KemperCo-5\KemperCo-5-01\Figures\CAD\KemperCo-5-01-SP01.dwg | Layout: FIGURE 2

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at the site by drilling seven borings (B-1 through B-7) to depths between 9.9 and 36.5 feet BGS. Drilling services were provided by Dan J. Fischer Excavating, Inc. of Forest Grove, Oregon, on June 22, 2023, using a trailer-mounted drill rig and solid-stem augers. The exploration logs are presented in this appendix.

The approximately exploration locations are shown on Figure 2. The exploration locations were determined in the field by pacing and measuring from existing site features. This information should be considered accurate only to the degree implied by the methods used.

SOIL SAMPLING

Samples were collected from the borings using 1½-inch-inner diameter SPT split-barrel samplers in general accordance with ASTM D1586. The samplers were driven into the soil with a 140-pound hammer free-falling 30 inches. The sampler was driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the exploration logs, unless otherwise noted. Sampling intervals are shown on the exploration logs.

The hammer used to conduct the SPTs was lifted using a rope and cathead system. The hammer was raised using two wraps of the rope around the cathead to conduct the SPTs.

SOIL CLASSIFICATION

The soil samples were classified in the field in accordance with the “Exploration Key” (Table A-1) and “Soil Classification System” (Table A-2), which are presented in this appendix. The exploration logs indicate the depths at which the soil characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration logs.

LABORATORY TESTING

CLASSIFICATION

The soil samples were classified in the laboratory to confirm field classifications. The laboratory classifications are shown on the exploration logs if those classifications differed from the field classifications.

MOISTURE CONTENT

The natural moisture content of select soil samples was determined in general accordance with ASTM D2216. The natural moisture content is a ratio of the weight of the water to dry soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

PARTICLE-SIZE ANALYSIS

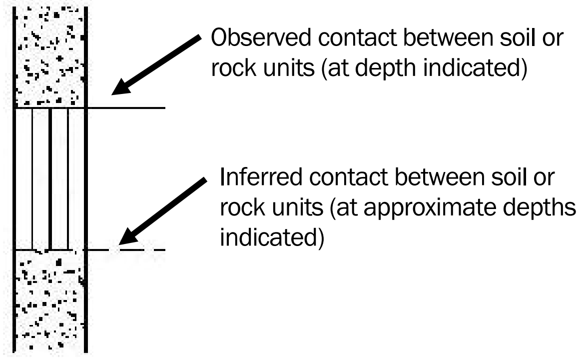
Particle-size analysis was performed on select soil samples in general accordance with ASTM D1140. This test is a quantitative determination of the amount of material finer than the U.S. Standard No. 200 sieve expressed as a percentage of soil weight. The test results are presented in this appendix.

ATTERBERG LIMITS

The plastic limit and liquid limit (Atterberg limits) of select soil samples were determined in accordance with ASTM D4318. The Atterberg limits and the plasticity index were completed to aid in the classification of the soil. The plastic limit is defined as the moisture content (in percent) where the soil becomes brittle. The liquid limit is defined as the moisture content where the soil begins to act similar to a liquid. The plasticity index is the difference between the liquid and plastic limits. The test results are presented in this appendix.

SYMBOL	SAMPLING DESCRIPTION
◨	Location of sample collected in general accordance with ASTM D1586 using Standard Penetration Test (SPT) with recovery
▮	Location of sample collected using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D1587 with recovery
▥	Location of sample collected using Dames & Moore sampler and 300-pound hammer or pushed with recovery
▦	Location of sample collected using Dames & Moore sampler and 140-pound hammer or pushed with recovery
◪	Location of sample collected using 3-inch-outside diameter California split-spoon sampler and 140-pound hammer with recovery
⊠	Location of grab sample
▩	Rock coring interval
▽	Water level during drilling
▼	Water level taken on date shown

Graphic Log of Soil and Rock Types



Observed contact between soil or rock units (at depth indicated)

Inferred contact between soil or rock units (at approximate depths indicated)

GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	P	Pushed Sample
CBR	California Bearing Ratio	PP	Pocket Penetrometer
CON	Consolidation	P200	Percent Passing U.S. Standard No. 200 Sieve
DD	Dry Density		
DS	Direct Shear	RES	Resilient Modulus
HYD	Hydrometer Gradation	SIEV	Sieve Gradation
MC	Moisture Content	TOR	Torvane
MD	Moisture-Density Relationship	UC	Unconfined Compressive Strength
NP	Non-Plastic	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

NV5

EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	Standard Penetration Test (SPT) Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very loose	0 – 4	0 – 11	0 – 4
Loose	4 – 10	11 – 26	4 – 10
Medium dense	10 – 30	26 – 74	10 – 30
Dense	30 – 50	74 – 120	30 – 47
Very dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOIL

Consistency	Standard Penetration Test (SPT) Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 – 4	3 – 6	2 – 5	0.25 – 0.50
Medium stiff	4 – 8	6 – 12	5 – 9	0.50 – 1.0
Stiff	8 – 15	12 – 25	9 – 19	1.0 – 2.0
Very stiff	15 – 30	25 – 65	19 – 31	2.0 – 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOIL (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVEL WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SAND (<5% fines)	SW or SP	SAND
		SAND WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SAND WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOIL (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOIL			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

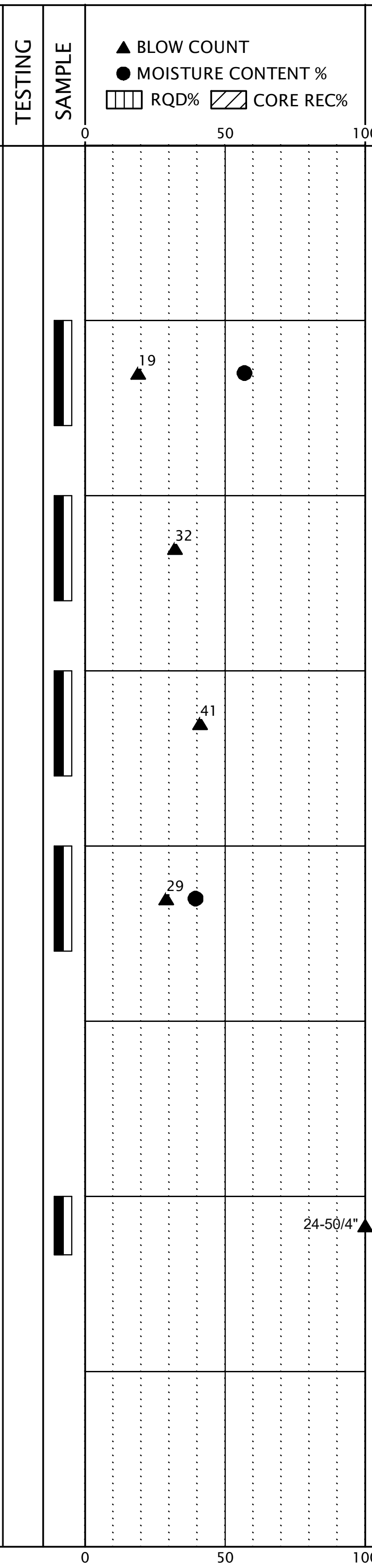
Secondary granular components or other materials such as organics, man-made debris, etc.

Percent	Silt and Clay In:		Percent	Sand and Gravel In:	
	Fine-Grained Soil	Coarse-Grained Soil		Fine-Grained Soil	Coarse-Grained Soil
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
0.0	Medium stiff, brown SILT (MH), trace sand and organics (rootlets); moist (topsoil to 7 inches, 3-inch-thick root zone).	0.6	
	Very stiff, light gray with orange streaked SILT (MH), trace sand and organics (rootlets); dry to moist, sand is fine.		
2.5			
5.0	hard, light gray-brown with orange and black streaks, trace to minor sand, without organics; interbeds of SILT with sand (2 inches thick) (weathered mudstone) at 5.0 feet		
7.5	light brown with orange and red streaks, minor sand at 7.5 feet		
10.0	very stiff at 10.0 feet interbeds of light brown SAND with silt (1 inch to 2 inches thick) at 10.7 feet		Driller Comment: Hard drilling from 10.0 to 14.0 feet.
12.5			
15.0	hard, gray, trace sand at 15.0 feet		
	Exploration terminated at a depth of 15.8 feet due to refusal. SPT completed using two wraps with a cathead.	15.8	Surface elevation was not measured at the time of exploration.
17.5			
20.0			



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: I. Allen
COMPLETED: 06/22/23

BORING METHOD: solid-stem auger (see document text)
BORING BIT DIAMETER: 4 inches

NV5

KEMPERCO-5-01

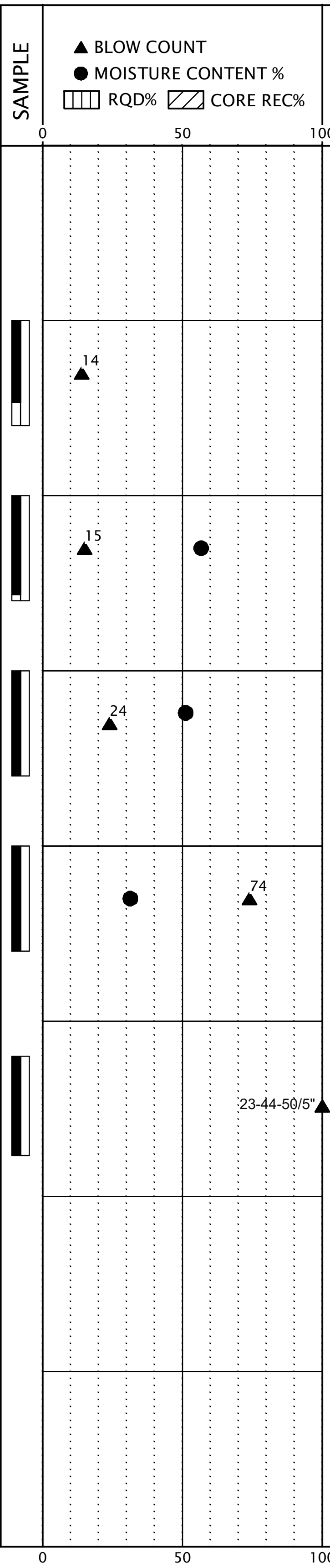
SEPTEMBER 2023

BORING B-1

PROPOSED MIXED-USE DEVELOPMENT
LINCOLN CITY, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Medium stiff, brown SILT (MH), trace sand and organics (rootlets); moist (topsoil to 12 inches, 3-inch-thick root zone).	1.0		
	Stiff, light brown with orange streaked SILT (MH), minor sand, trace organics (rootlets); dry to moist, sand is fine.			
5.0	stiff to very stiff, without organics at 5.0 feet		P200	P200 = 74%
	Very stiff, light brown with orange streaked, sandy SILT (ML); moist, sand is fine. dark brown at 8.5 feet light brown with dark brown streaks at 8.8 feet	8.1	P200	P200 = 55%
	Hard, light gray-brown with orange streaked SILT (MH), trace sand; dry to moist, sand is fine (weathered mudstone).	9.5		
	Exploration terminated at a depth of 14.4 feet due to refusal. SPT completed using two wraps with a cathead.	14.4		Surface elevation was not measured at the time of exploration.



DEPTH FEET: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0

DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: I. Allen
COMPLETED: 06/22/23

BORING METHOD: solid-stem auger (see document text)
BORING BIT DIAMETER: 4 inches

KEMPERCO-5-01

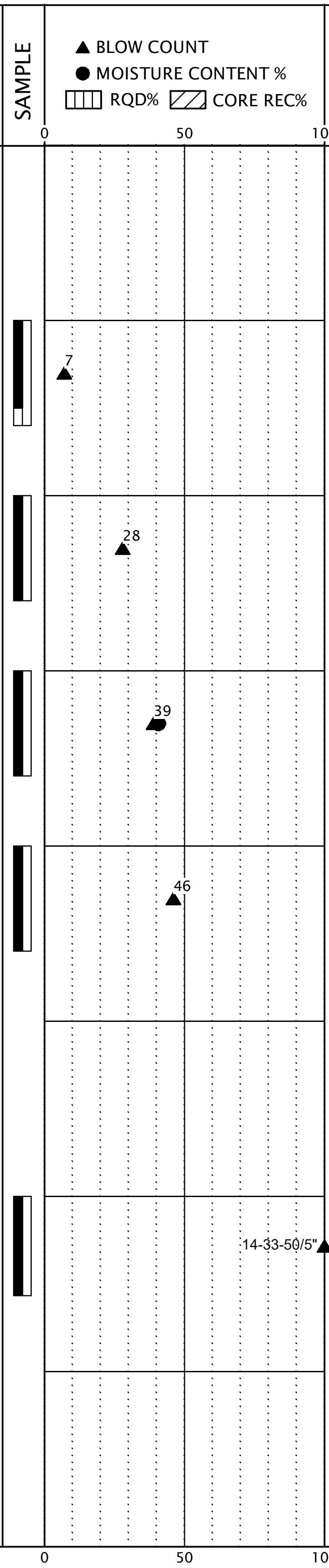
SEPTEMBER 2023

BORING B-2

PROPOSED MIXED-USE DEVELOPMENT
LINCOLN CITY, OR

FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	Medium stiff, brown SILT (MH), trace sand and organics (rootlets); moist (topsoil to 18 inches, 3-inch-thick root zone).			
	Medium stiff, dark brown SILT with organics (ML), trace sand; moist, sand is fine, organics are rootlets and straw.	1.5		
2.5	Medium stiff, light brown with orange and dark brown streaked, sandy SILT (MH), trace organics (rootlets); moist, sand is fine.	2.8	7	
5.0	very stiff, light brown with orange and red streaks, minor sand, without organics at 5.0 feet		28	
7.5	hard, trace sand (weathered mudstone) at 7.5 feet		39	
10.0	light gray-brown with orange and red streaks, trace to minor sand at 10.0 feet		46	
12.5				
15.0			14-33-50/5"	
	Exploration terminated at a depth of 16.4 feet due to refusal. SPT completed using two wraps with a cathead.	16.4		Surface elevation was not measured at the time of exploration.
17.5				
20.0				



DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: I. Allen

COMPLETED: 06/22/23

BORING METHOD: solid-stem auger (see document text)

BORING BIT DIAMETER: 4 inches

NV5

KEMPERCO-5-01

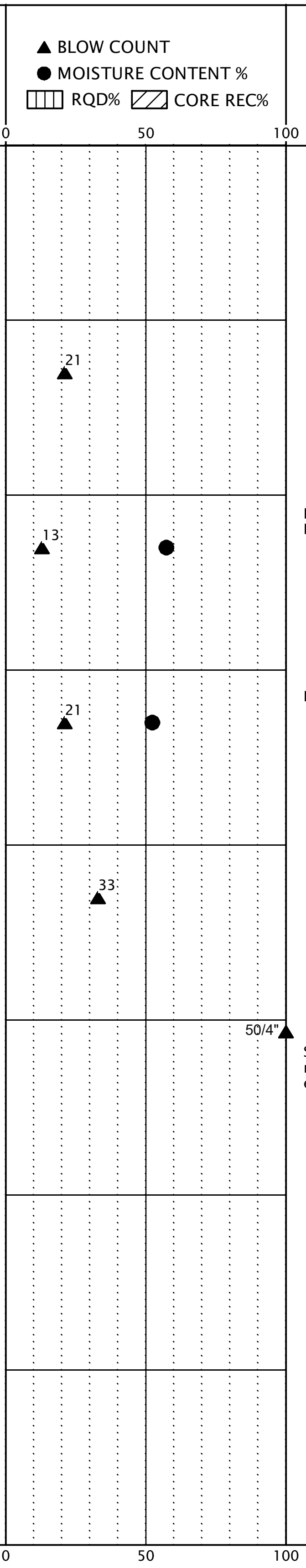
BORING B-3

SEPTEMBER 2023

PROPOSED MIXED-USE DEVELOPMENT
LINCOLN CITY, OR

FIGURE A-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Medium stiff, brown SILT (MH), trace sand and organics (rootlets); moist (topsoil to 12 inches, 3-inch-thick root zone).			
	Very stiff, light gray-brown with orange and red streaked SILT (MH), trace sand and organics (rootlets); moist, sand is fine.	1.0		
2.5				
5.0	stiff, without organics at 5.0 feet		ATT	LL = 90% PL = 60%
7.5	very stiff, minor sand at 7.5 feet		P200	P200 = 91%
10.0	hard, brown with orange and red streaks (weathered mudstone) at 10.0 feet			
12.5	gray, trace sand at 12.5 feet			Surface elevation was not measured at the time of exploration.
	Exploration terminated at a depth of 12.8 feet due to refusal. SPT completed using two wraps with a cathead.	12.8		
15.0				
17.5				
20.0				



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: I. Allen
COMPLETED: 06/22/23

BORING METHOD: solid-stem auger (see document text)
BORING BIT DIAMETER: 4 inches

KEMPERCO-5-01

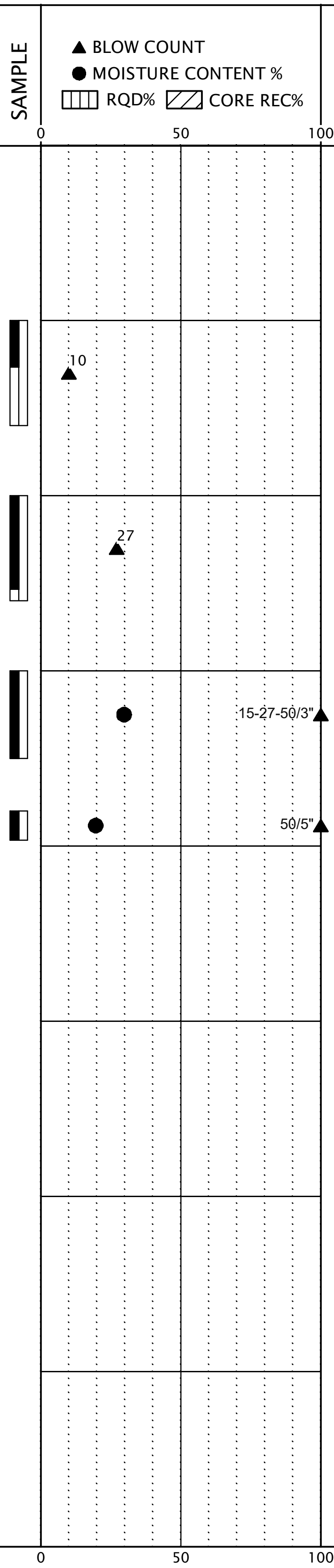
SEPTEMBER 2023

BORING B-4

PROPOSED MIXED-USE DEVELOPMENT
LINCOLN CITY, OR

FIGURE A-4

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	INSTALLATION AND COMMENTS
0.0		Stiff, dark brown SILT (MH), trace sand and organics (rootlets, straw); moist, sand is fine (3-inch-thick root zone) - **FILL**.				
		Very stiff, light brown with orange and red streaked SILT (MH), trace sand; dry to moist, sand is fine.	4.5			
7.5		hard at 7.5 feet				
		interbeds of red-brown SAND with silt at 8.3 feet gray; without interbeds at 8.6 feet		ATT		LL = 59% PL = 36%
		Exploration terminated at a depth of 9.9 feet due to refusal. SPT completed using two wraps with a cathead.	9.9			Surface elevation was not measured at the time of exploration.



DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: I. Allen

COMPLETED: 06/22/23

BORING METHOD: solid-stem auger (see document text)

BORING BIT DIAMETER: 4 inches

KEMPERCO-5-01

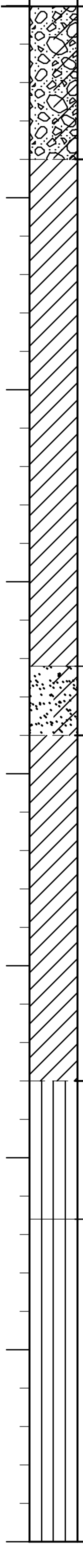
SEPTEMBER 2023

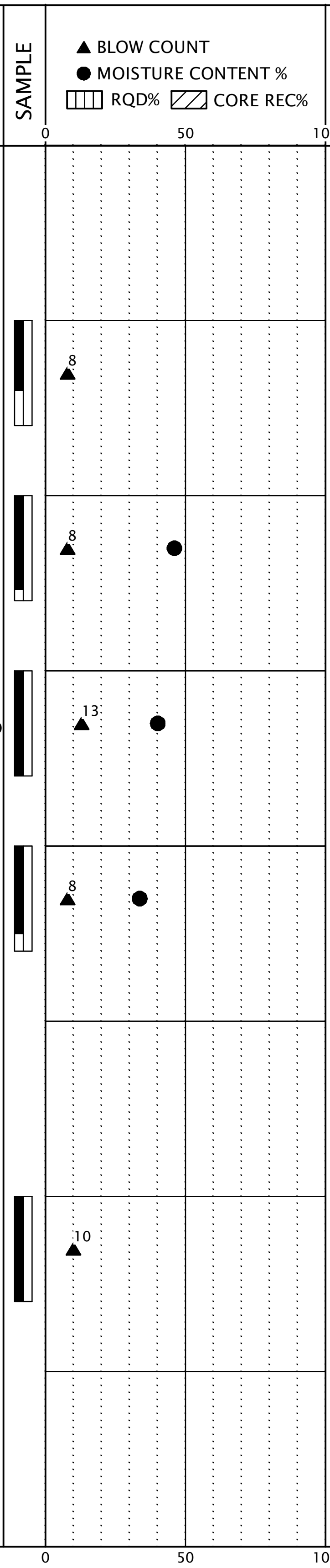
BORING B-5

PROPOSED MIXED-USE DEVELOPMENT
LINCOLN CITY, OR

FIGURE A-5

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Medium dense, gray-brown GRAVEL with clay (GP-GC), minor sand; moist, gravel is subangular to angular, sand is fine (3-inch-thick root zone) - **FILL**.			
	Medium stiff to stiff, brown with orange mottled CLAY (CL), minor gravel, trace sand and organics (rootlets, straw); moist, gravel is fine to coarse and subangular, sand is fine, lenses of light gray CLAY - **FILL**.	2.0		
7.5	stiff, brown with orange and light gray mottles, sandy, without gravel at 7.5 feet light gray with orange mottles at 7.9 feet		P200	P200 = 59%
	Medium dense, orange-brown SAND with clay (SP-SC); moist, sand is fine - **FILL**.	8.6		
10.0	Medium stiff to stiff, brown with orange mottled CLAY (CL), trace to minor sand; moist, sand is fine, lenses of SAND with clay (1 inch to 2 inches thick) - **FILL**.	9.5		
	Stiff, light blue to dark gray SILT (MH), trace sand and organics (grass, rootlets); moist, sand is fine, minor organic odor (buried topsoil).	14.0		
	Stiff, brown with orange and gray mottled SILT (MH), minor sand; moist, interbeds of SAND with silt (1 inch to 3 inches thick).	15.8		
20.0				





DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: I. Allen

COMPLETED: 06/22/23

BORING METHOD: solid-stem auger (see document text)

BORING BIT DIAMETER: 4 inches

NV5

KEMPERCO-5-01

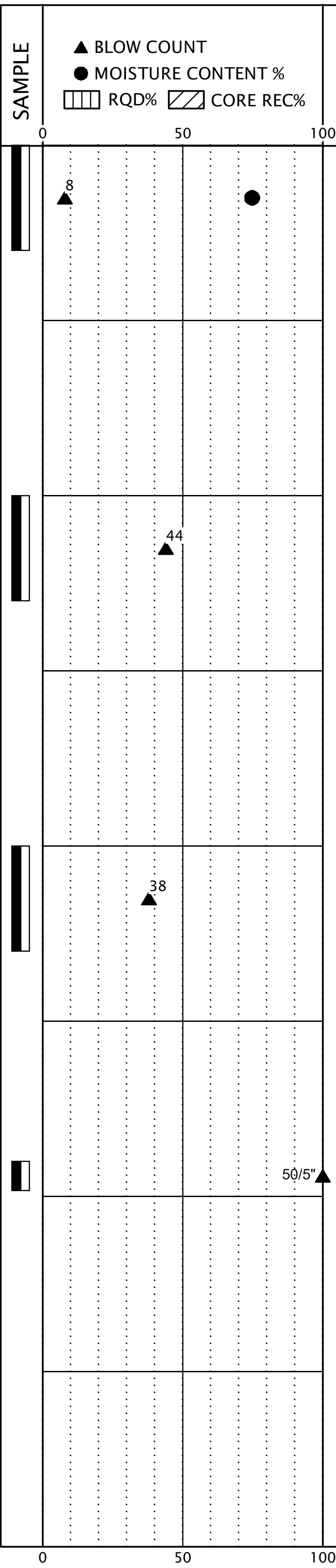
SEPTEMBER 2023

BORING B-6

PROPOSED MIXED-USE DEVELOPMENT
LINCOLN CITY, OR

FIGURE A-6

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
20.0	medium stiff to stiff, blue-gray and dark brown, trace sand and organics (rootlets); wet at 20.0 feet		
	Hard, light brown with orange streaked SILT (MH), minor sand; moist, sand is fine, interbeds of sandy CLAY (3 to 6 inches thick) (weathered mudstone).	21.4	
22.5			
25.0			
	minor to with sand at 26.3 feet		▼ 26.5 feet, after drilling
27.5			
30.0	dark gray, trace sand at 30.0 feet		
32.5			
	Exploration terminated at a depth of 34.9 feet due to refusal. SPT completed using two wraps with a cathead.	34.9	Surface elevation was not measured at the time of exploration.
35.0			
37.5			
40.0			



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: I. Allen
COMPLETED: 06/22/23

BORING METHOD: solid-stem auger (see document text)
BORING BIT DIAMETER: 4 inches

NV5

KEMPERCO-5-01

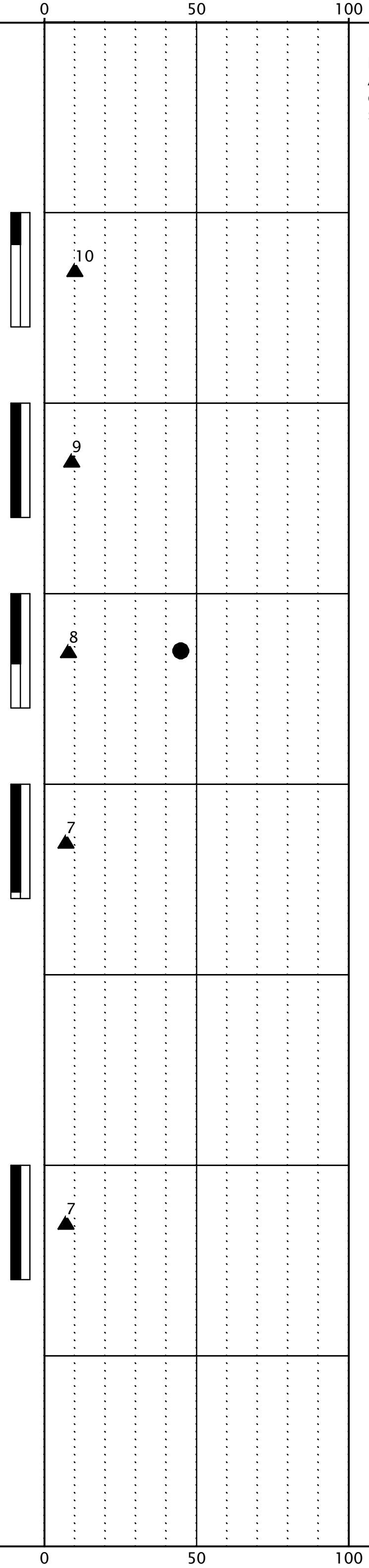
SEPTEMBER 2023

BORING B-6
(continued)

PROPOSED MIXED-USE DEVELOPMENT
LINCOLN CITY, OR

FIGURE A-6

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	Medium dense, gray-brown GRAVEL with clay and cobbles (GP-GC), minor sand and debris (asphalt pieces); moist, gravel is subangular to angular, cobbles are approximately 15%, sand is fine (1-inch-thick root zone) - **FILL**.	1.0			Driller Comment: Approximately 1 inch of cobbles and gravel on surface.
	Stiff, light brown with orange mottled CLAY (CL), trace sand; moist, sand is fine - **FILL**.				
2.5				10	
5.0	light gray-brown with red and orange streaks, trace organics (woody debris) at 5.0 feet dark gray to blue-gray at 5.7 feet brown with orange streaks at 6.0 feet			9	
7.5	medium stiff to stiff; interbeds of gray GRAVEL with sand, gravel is angular (2 inches thick) at 7.9 feet			8	
10.0	medium stiff, light gray-brown with orange and black streaks, minor sand, without organics; without interbeds at 10.0 feet			7	
12.5					
15.0	with gravel at 15.8 feet dark brown, with sand and organics (woody debris), without gravel at 15.9 feet			7	
17.5					
20.0					



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: I. Allen
COMPLETED: 06/22/23

BORING METHOD: solid-stem auger (see document text)
BORING BIT DIAMETER: 4 inches

NV5

KEMPERCO-5-01

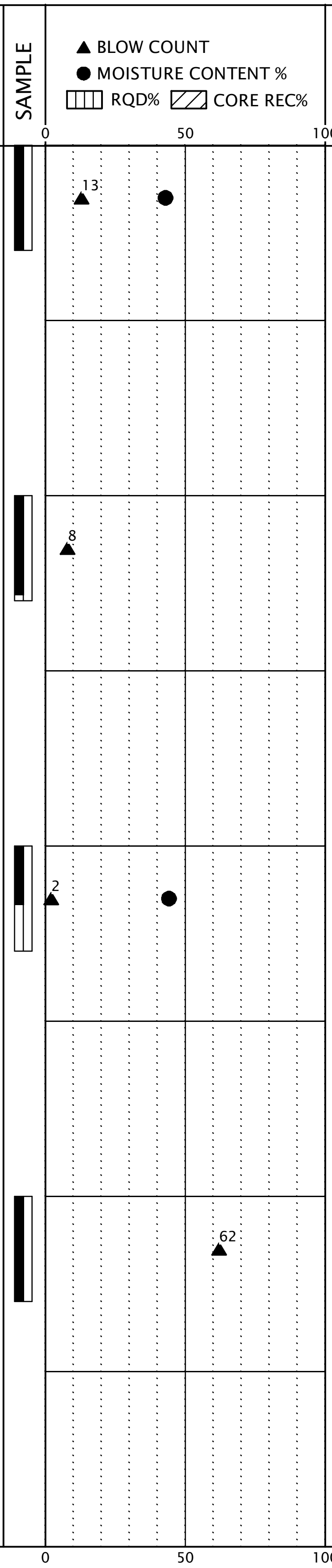
SEPTEMBER 2023

BORING B-7

PROPOSED MIXED-USE DEVELOPMENT
LINCOLN CITY, OR

FIGURE A-7

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	BLOW COUNT	INSTALLATION AND COMMENTS
20.0	stiff, brown with orange and red streaks; interbeds of SAND with clay (1 inch thick) at 20.0 feet dark gray with blue-gray mottles, trace organics (woody debris) at 20.5 feet brown with orange, red, and black streaks, without organics at 20.7 feet		13	
22.5				
25.0	medium stiff to stiff at 25.0 feet light gray to light brown with orange and brown mottles, minor to with sand at 26.1 feet		8	
27.5				
30.0	soft, dark gray to black, sandy to with sand; slight organic odor at 30.0 feet blue-gray at 30.6 feet		2	
32.5	wet at 33.0 feet			▼ 33.0 feet, after drilling
35.0	Hard, light brown with orange and red streaked SILT (MH), trace sand; wet, sand is fine (weathered mudstone).	35.0	62	
	Exploration completed at a depth of 36.5 feet. SPT completed using two wraps with a cathead.	36.5		Surface elevation was not measured at the time of exploration.
37.5				
40.0				



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: I. Allen
COMPLETED: 06/22/23

BORING METHOD: solid-stem auger (see document text)
BORING BIT DIAMETER: 4 inches

NV5

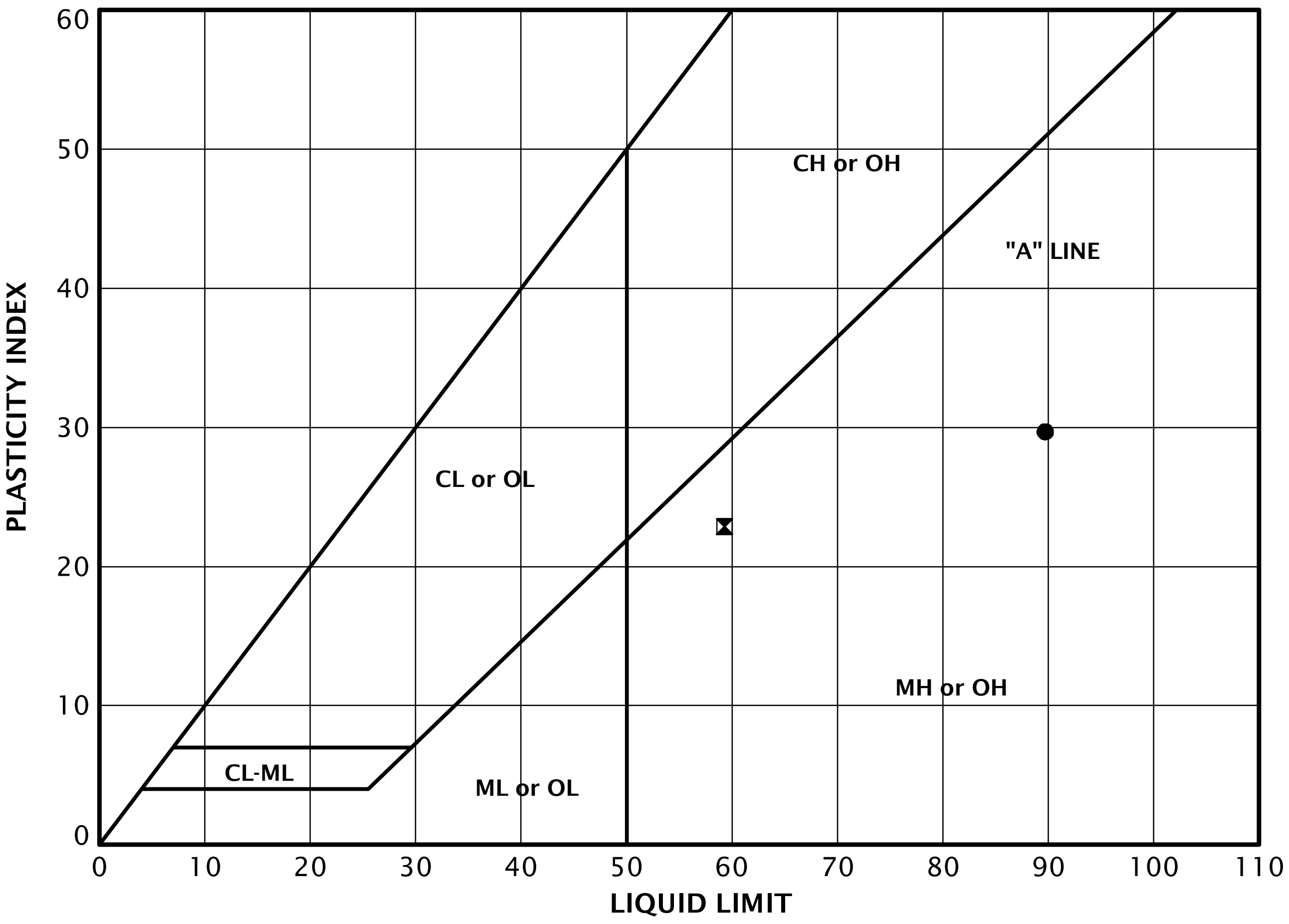
KEMPERCO-5-01

SEPTEMBER 2023

BORING B-7
(continued)

PROPOSED MIXED-USE DEVELOPMENT
LINCOLN CITY, OR

FIGURE A-7



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-4	5.0	57	90	60	30
☒	B-5	7.5	30	59	36	23

ATTERBERG LIMITS TEST RESULTS

PROPOSED MIXED-USE DEVELOPMENT
LINCOLN CITY, OR

FIGURE A-8

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	2.5		57							
B-1	10.0		39							
B-2	5.0		57				74			
B-2	8.1		51				55			
B-2	10.0		31							
B-3	7.5		41							
B-4	5.0		57					90	60	30
B-4	7.5		52				91			
B-5	7.5		30					59	36	23
B-5	9.5		20							
B-6	5.0		46							
B-6	7.5		40				59			
B-6	10.0		34							
B-6	20.0		75							
B-7	7.5		45							
B-7	20.0		43							
B-7	30.0		44							

LAB SUMMARY - GDI-NV5 KEMPERCO-5-01-B1_7.GPJ GDI_NV5.GDT PRINT DATE: 9/18/23:KT

NV5

KEMPERCO-5-01

SEPTEMBER 2023

SUMMARY OF LABORATORY DATA

PROPOSED MIXED-USE DEVELOPMENT
LINCOLN CITY, OR

FIGURE A-9

APPENDIX B

APPENDIX B

DCP TEST RESULTS

We conducted DCP testing in general accordance with ASTM D6951 to estimate the resilient modulus of the subgrade material at each test location. We recorded penetration depth of the cone for each blow of the hammer and terminated testing when at refusal of penetration or end of rod length. We plotted depth of penetration versus blow count and visually assessed where the slope of the data plot was relatively constant and at depths where the slope of the data plot changed significantly. We used least squares regression to determine the slopes and the equation from the ODOT Pavement Design Guide[1] to estimate the moduli using a correction factor of c_f = 0.35 for the subgrade.

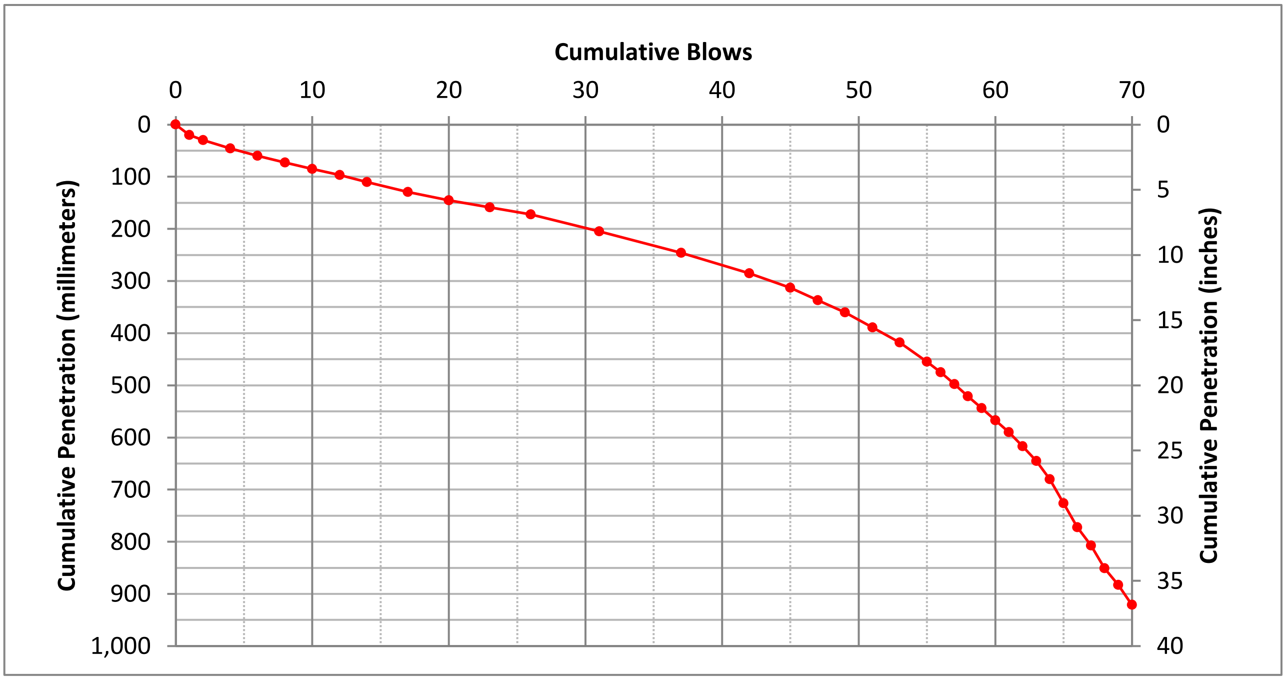
The DCP results are presented in this appendix.

---

[1] *ODOT Pavement Design Guide,* Pavement Services Unit, Oregon Department of Transportation, January 2019.

DYNAMIC CONE PENETROMETER RESULTS - DCP-1

Layer	Layer Type and Location	Slope (mm/blow)	C_f	M_R (psi)
1	No base material	---	---	---
2	Subgrade	7.0	0.35	8,030
3	Subgrade	31.5	0.35	4,470
Equivalent subgrade modulus based on Odemark's Method of Equivalent Thickness				5,870



$M_R = C_f \times 49023 \times S^{-0.39}$

M_R = resilient modulus (pounds per square inch)

C_f = conversion coefficient

S = slope (millimeters per blow)

References:

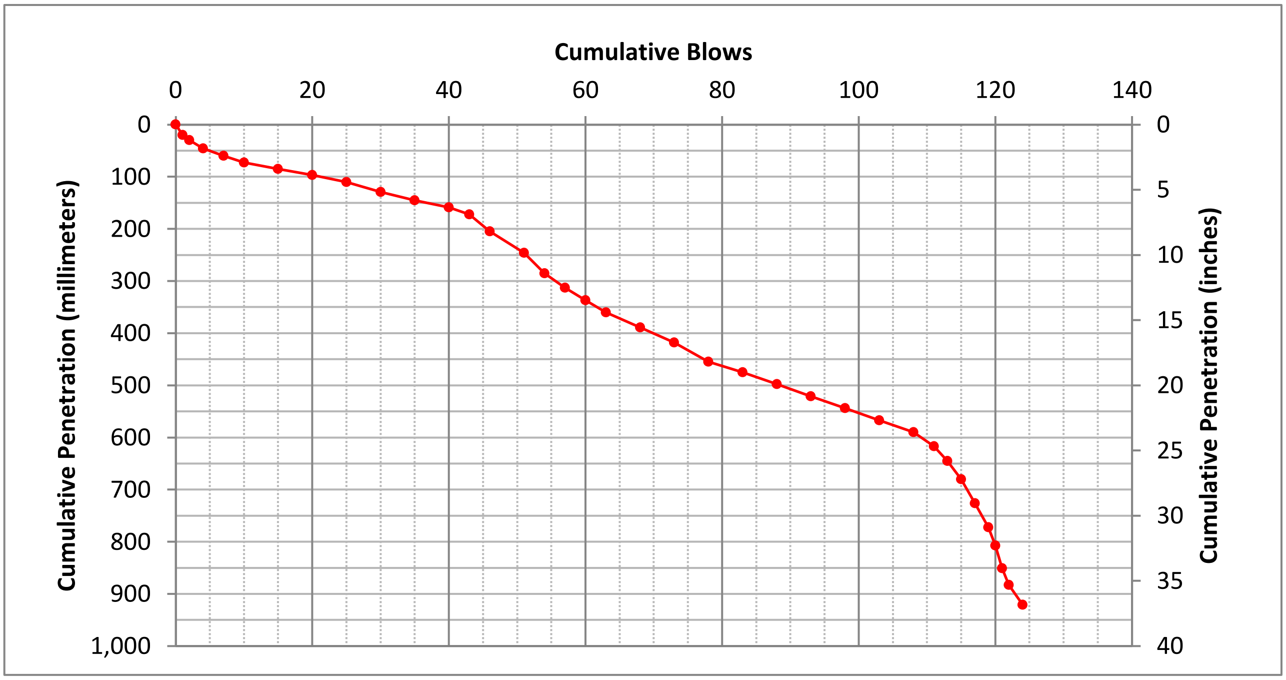
ODOT Pavement Design Guide, Pavement Services Unit, Oregon Department of Transportation, January 2019.

Jianzhou Chen, Mustaque Hossain, and Todd M. LaTorella, "Use of Falling Weight Deflectometer and Dynamic Cone Penetrometer in Pavement Evaluation," Paper No. 99-1007, Transportation Research Record 1655, pp 145-151, Transporation Research Board, Washington, D.C., 1999.

Per Ullidtz, *Modelling Flexible Pavement Response and Performance* , Tech Univ. of Denmark Polytekn, 1998.

DYNAMIC CONE PENETROMETER RESULTS - DCP-2

Layer	Layer Type and Location	Slope (mm/blow)	C_f	M_R (psi)
1	Subgrade	3.3	0.35	10,750
2	Subgrade	6.2	0.35	8,420
3	Subgrade	24.6	0.35	4,920
Equivalent subgrade modulus based on Odemark's Method of Equivalent Thickness				6,720



$M_R = C_f \times 49023 \times S^{-0.39}$

M_R = resilient modulus (pounds per square inch)

C_f = conversion coefficient

S = slope (millimeters per blow)

References:

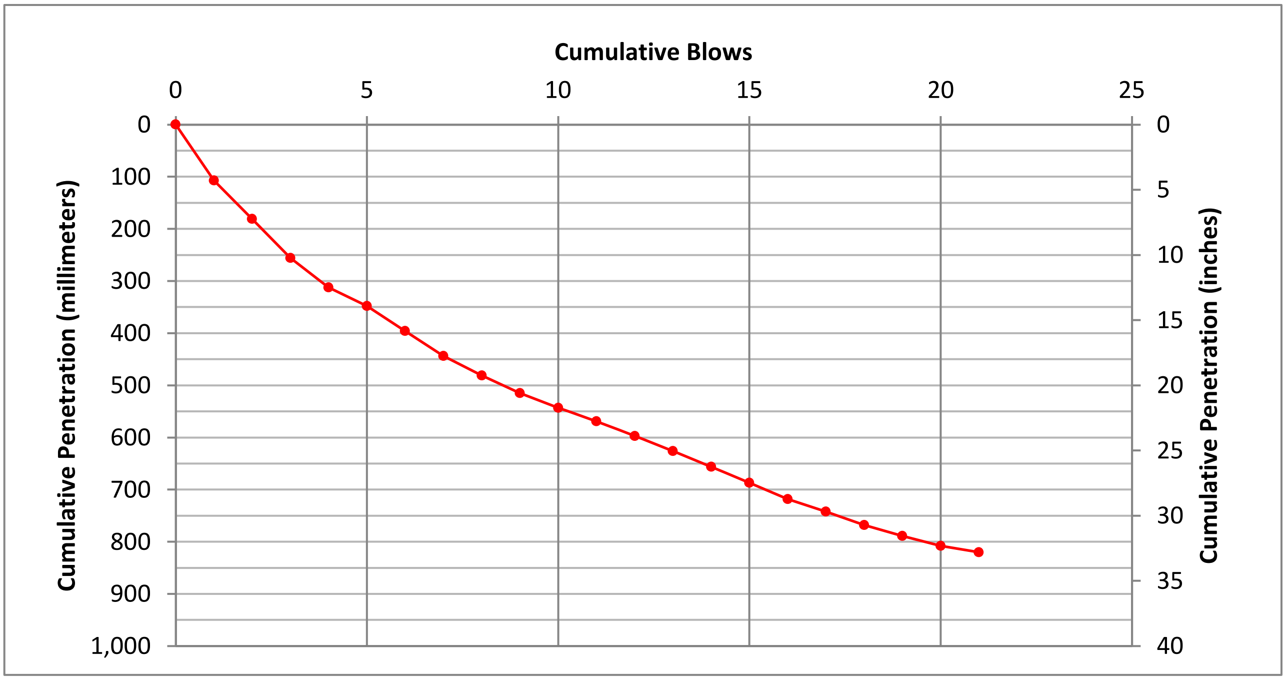
ODOT Pavement Design Guide, Pavement Services Unit, Oregon Department of Transportation, January 2019.

Jianzhou Chen, Mustaque Hossain, and Todd M. LaTorella, "Use of Falling Weight Deflectometer and Dynamic Cone Penetrometer in Pavement Evaluation," Paper No. 99-1007, Transportation Research Record 1655, pp 145-151, Transporation Research Board, Washington, D.C., 1999.

Per Ullidtz, *Modelling Flexible Pavement Response and Performance* , Tech Univ. of Denmark Polytekn, 1998.

DYNAMIC CONE PENETROMETER RESULTS - DCP-3

Layer	Layer Type and Location	Slope (mm/blow)	C_f	M_R (psi)
1	No base material	---	---	---
2	Subgrade	57.2	0.35	3,540
3	Subgrade	27.5	0.35	4,710
Equivalent subgrade modulus based on Odemark's Method of Equivalent Thickness				4,210



$M_R = C_f \times 49023 \times S^{-0.39}$

M_R = resilient modulus (pounds per square inch)

C_f = conversion coefficient

S = slope (millimeters per blow)

References:

ODOT Pavement Design Guide, Pavement Services Unit, Oregon Department of Transportation, January 2019.

Jianzhou Chen, Mustaque Hossain, and Todd M. LaTorella, "Use of Falling Weight Deflectometer and Dynamic Cone Penetrometer in Pavement Evaluation," Paper No. 99-1007, Transportation Research Record 1655, pp 145-151, Transporation Research Board, Washington, D.C., 1999.

Per Ullidtz, *Modelling Flexible Pavement Response and Performance* , Tech Univ. of Denmark Polytekn, 1998.

DYNAMIC CONE PENETROMETER RESULTS - DCP-4

Layer	Layer Type and Location	Slope (mm/blow)	C_f	M_R (psi)
1	Subgrade	36.9	0.35	4,200
2		---	---	---
3		---	---	---
Equivalent subgrade modulus based on Odemark's Method of Equivalent Thickness				---

$M_R = C_f \times 49023 \times S^{-0.39}$

M_R = resilient modulus (pounds per square inch)

C_f = conversion coefficient

S = slope (millimeters per blow)

References:

ODOT Pavement Design Guide, Pavement Services Unit, Oregon Department of Transportation, January 2019.

Jianzhou Chen, Mustaque Hossain, and Todd M. LaTorella, "Use of Falling Weight Deflectometer and Dynamic Cone Penetrometer in Pavement Evaluation," Paper No. 99-1007, Transportation Research Record 1655, pp 145-151, Transporation Research Board, Washington, D.C., 1999.

Per Ullidtz, *Modelling Flexible Pavement Response and Performance* , Tech Univ. of Denmark Polytekn, 1998.

NV5

Delivering Solutions
Improving Lives