# REPORT OF GEOTECHNICAL ENGINEERING SERVICES

1st and Strand Streets
St Helens, Oregon

For
Otak
January 5, 2022

Project:  StHelens-3-01

January 5, 2022

Otak
808 SW Third Avenue, Suite 800
Portland, OR 97204

Attention: Keith Buisman, P.E.

**Report of Geotechnical Engineering Services**
1st and Strand Streets
St Helens, Oregon
Project: StHelens-3-01

NV5 is pleased to submit this report of geotechnical engineering services for the road and utility extensions for South 1st and Strand Streets in St Helens, Oregon. The report discusses geotechnical engineering recommendations for design and construction of the proposed roadway and utility improvements.

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

Sincerely,

NV5

Shawn M. Dimke, P.E., G.E.
Principal Engineer

cc: Mike Peebles, Otak (via email only)
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**TABLE OF CONTENTS** **PAGE NO.**

**TABLE OF CONTENTS** **PAGE NO.**

## ACRONYMS AND ABBREVIATIONS

| | |
|---|---|
| AASHTO | American Association of State Highway and Transportation Officials |
| AC | asphalt concrete |
| ACP | Asphalt Concrete Pavement |
| ADT | average daily traffic |
| ASCE | American Society of Civil Engineers |
| ASTM | American Society for Testing and Materials |
| BGS | below ground surface |
| CRBG | Columbia River Basalt Group |
| DCP | dynamic cone penetrometer |
| ESAL | equivalent single-axle load |
| FHWA | Federal Highway Administration |
| g | gravitational acceleration (32.2 feet/second$^2$) |
| GPS | global positioning system |
| H:V | horizontal to vertical |
| MCE | maximum considered earthquake |
| NA | not applicable |
| NP | not present |
| ODOT | Oregon Department of Transportation |
| OSHA | Occupational Safety and Health Administration |
| OSSC | Oregon Standard Specifications for Construction (2021) |
| pcf | pounds per cubic foot |
| PG | performance grade |
| psf | pounds per square foot |
| psi | pounds per square inch |
| RQD | rock quality designation |
| SPT | standard penetration test |

## 1.0 INTRODUCTION

NV5 is pleased to present this report documenting our geotechnical engineering services for the road and utility extensions for South 1st and Strand Streets near the riverfront in St Helens, Oregon. The improvements will extend through the former Veneer property and provide infrastructure for redevelopment of the riverfront. The site location with respect to the surrounding physical features is shown on Figure 1. All elevations referenced in this report are relative to North American Vertical Datum 88.

The planned improvements include extending South 1st Street from Cowlitz Street south to Plymouth Street. Strand Street will extend south and west from Columbia View Park to intersect South 1st Street. New utility extensions and the relocation of the existing sanitary sewer lift station on the Veneer property will support new development and improve the existing City of St. Helens (City) systems. A site plan showing the proposed roadway alignment and the approximate subsurface exploration locations is shown on Figure 2.

Other than the riverbank slopes, the site is relatively flat and cuts and fills will be less than a few feet each. The planned lift station will be west of the new section of South 1st Street, near the location of our boring B-8. This report discusses geotechnical engineering recommendations for design and construction of the proposed roadway improvements.

Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

## 2.0 PURPOSE AND SCOPE

The purpose of our geotechnical engineering services was to explore the subsurface conditions near the proposed improvements and develop geotechnical engineering recommendations for design and construction of the proposed roadway and utility improvements. Our specific scope of services included the following:

- Reviewed available existing documentation.
- Coordinated and managed the field investigation, including locating utilities, access preparation, and scheduling of contractors and NV5 staff.
- Prepared traffic control plans and obtained right-of-way permits from the City.
- Completed the following subsurface explorations;
  - Five borings with pavement cores in existing roadway areas to depths between 7.5 and 16.5 feet BGS
  - Thirteen borings in undeveloped areas of the site to depths between 2.8 and 32 feet BGS
- Completed DCP testing in 12 borings.
- Drummed samples and stored them on site for disposal pending results from environmental testing.
- Collected representative pavement, base, soil, and rock samples from the explorations. Classified the materials encountered in the explorations and maintained a detailed log of each exploration.

- Completed laboratory analyses on disturbed and undisturbed soil samples collected from the explorations as follows:
  - Thirty-nine moisture content determinations in general accordance with ASTM D2216
  - Two Atterberg limits tests in general accordance with ASTM D4318
  - Six particle-size analyses in general accordance with ASTM D1140
  - Two unconfined compression tests in general accordance with ASTM D7012
- Analyzed traffic information provided by DKS Associates and estimated pavement design ESALs.
- Analyzed subsurface and DCP results to determine pavement support characteristics.
- Provided recommendations for pavement repair and rehabilitation in existing pavement areas.
- Provided recommendations for new and widening pavement structures.
- Provided this geotechnical report for the project that includes the following:
  - Geotechnical engineering construction recommendations for site preparation, structural fill compaction criteria, and wet/dry weather earthwork procedures
  - Geotechnical engineering recommendations for utility trenching, including rock excavation information
  - Geotechnical and pavement engineering material recommendations
  - Foundation recommendations for the pump station

## 3.0 SITE CONDITIONS

### *3.1 GEOLOGIC SETTING*

The site is located in the northwestern portion of the Portland Basin physiographic province, which is bound by the Tualatin Mountains to the west and south and the Cascade Range to the east and north. The near-surface geologic unit is mapped as catastrophic mixed grain flood deposits left by the Missoula and Bonneville floods. Based on the development history of the riverbank and exploration results, undocumented fill material overlies the alluvial deposits. The fill and alluvium are underlain by basalt flows belonging to the Sentinel Bluffs member of the CRBG, which is also the surface geologic unit for the bluff bordering the western edge of the site. The Miocene aged (20 million to 10 million years ago) CRBG is a series of basalt flows that originated from southeastern Washington and northeastern Oregon (Evarts, 2004). The Sentinel Bluffs basalt flows are reported to be up to 300 feet thick and are considered the geologic basement unit for this report.

### *3.2 SURFACE CONDITIONS*

#### 3.2.1 General

Within the limits of our investigation, South 1st Street is an AC-paved collector road with one lane in each direction. Road edges are curbed and consists of on-site parking in both directions. South 6th Street is an AC-paved collector road with one lane in each direction. The eastbound road edge is curbed and the westbound road edge is unimproved. South 2nd Street is an AC-paved local road with one lane in each direction. Road edges are unimproved and consists of gravel parking and driveways. Strand Street is an AC-paved local road with one lane in each direction. Road edges are predominantly curbed and consists of on-site parking stalls in both directions.

The planned new road sections for Strand Street, South 1st Street, and a short segment connecting the two are in an existing undeveloped area generally surfaced with gravel and existing AC. The ground surface in the undeveloped area is relatively flat with a very gentle slope to the east toward the steeper bank at the edge of the Columbia River. Elevations generally range between approximately 25 and 30 feet for the flat, undeveloped area of the site.

#### 3.2.2 Pavement Distress

We performed a generalized visual survey for pavement distress along the existing pavement section within the scope of our investigation on May 13, 2021. The survey did not evaluate ride roughness or friction. We observed intermittent low severity fatigue cracking and low severity thermal cracking on all the paved sections.

### *3.3 SUBSURFACE CONDITIONS*

We explored subsurface conditions at the proposed development site by drilling 13 borings (B-1 through B-13) to a maximum depth of 32 feet BGS and drilling five borings (C-1 through C-5) through the existing pavement to a maximum depth of 16.5 feet BGS. The approximate locations of the explorations are shown on Figure 2. The exploration logs and a detailed description of the field explorations are presented in Appendix A.

#### 3.3.1 Pavement Thicknesses

Eight out of 13 borings (B-5 and B-7 through B-13) in undeveloped areas were drilled on an AC surface comprised of 2.5 to 10.0 inches of AC. A summary of the pavement thickness observed in C-1 through C-5 in the existing road sections is presented in Table 1. We encountered AC ranging in thickness from 3.5 to 6.5 inches and aggregate base ranging in thickness from 12.5 to 25.5 inches. Cores on cracks indicate full-depth cracking at boring C-1 and C-2. We observed a delaminated layer in the boring C-2 at a depth of 1.5 inches.

**Table 1. Existing Pavement Thickness**

| Boring Number | Street | Thickness (inches) | | AC Crack Depth (inches) | Delamination Depth (inches) |
|---|---|---|---|---|---|
| | | AC | Aggregate Base | | |
| C-1 | South 1st Street | 4.5 | 12.5 | 0 to 4.5 | NP |
| C-2 | South 1st Street | 4.5 | 25.5 | 0 to 4.5 | 1.5 |
| C-3 | South 6th Street | 4.0 | 12.0[1] | NP | NP |
| C-4 | South 2nd Street | 3.5 | 14.5 | NP | NP |
| C-5 | Strand Street | 6.5 | 23.5 | NP | NP |

1. Aggregate base capped at 12.0 inches

#### 3.3.2 Fill

We observed fill, including pavement materials or shoulder aggregate, at all borings. In general, the fill below the pavement consists of gravel and sand with silt to depths ranging up to 16.5 feet BGS (the maximum depth explored). Silt, clay, and organic fill was also encountered as detailed below. We observed medium dense to very dense gravel (B-3, B-4, B-6 through B-12, and C-3), loose to medium dense sand (B-1 through B-3, B-6 through B-11, and B-13), very soft silt (B-6), very soft to stiff clay (C-1 and C-2), or organic fill (B-3 and B-4). In addition to the above, we encountered an approximately 7-inch-thick concrete slab at 2 feet BGS in B-7.

#### 3.3.3 Silt and Clay

Very soft to stiff silt and clay was encountered below the pavement and fill layers at some locations (B-1, B-3, B-6 through B-8, B-11, B-13, and C-5). The sand and gravel content for the silt and clay generally varies between trace and with sand and/or gravel.

#### 3.3.4 Sand and Gravel

We observed very loose, silty sand in one boring (B-10) at a depth of 15 feet BGS to the depth explored of 16.5 feet BGS. In general, however, the native material below the silt and clay consists of medium dense to very dense weathered basalt gravel. We encountered refusal SPT blow counts (greater than 50 blow per 6 inches) in the very dense gravel at varying depths at B-1 through B-8 and C-1. Based on rock coring conducted in other borings, the very dense basalt gravel quickly transitions to basalt or represents the top of the basalt.

#### 3.3.5 Basalt

We cored basalt from depths of approximately 4.5 to 12.5 feet BGS in boring C-2, 1.5 to 7.5 feet BGS in C-4, 4 to 10 feet BGS in C-5, and 17 to 32 feet BGS in B-13. The basalt is generally medium hard to hard (R3 to R4) and moderately to very intensely fractured. Photographs of the recovered rock cores are presented in Appendix A. Unconfined compression testing indicates an unconfined compression strength of approximately 5,500 psi for a sample at 8.5 feet BGS from C-5 and 12,000 psi for a sample at 27.5 feet BGS from B-13. The results of the unconfined compression tests are presented in Appendix B. The basalt is generally shallow near the western edge of the vacant site area and increases with depth towards the river. The depths to refusal blow counts in very dense weathered basalt gravel or basalt are indicated on Figure 2.

#### 3.3.6 Groundwater

Mud rotary drilling methods prevented the measurement of groundwater in some borings. Where direct-push drilling methods were used, groundwater was encountered at depths between 3.5 feet to greater than 16.5 feet BGS. The groundwater was encountered approximately 1.6 to 6.5 feet above the depth of refusal blow counts in the very dense weathered basalt gravel or basalt, where present. Based on the results of our explorations, groundwater perches above the basalt where it is shallower and likely transitions from the shallow depths to the adjacent level of the Columbia River where the basalt becomes deeper. The Columbia River has an ordinary low-water elevation of 2.9 feet and an ordinary high-water elevation of 14.6 feet near the site.

#### 3.3.7 Laboratory Testing

In addition to the moisture content testing, we completed six particle-size analyses and two Atterberg limits tests on samples collected from the explorations. Descriptions of the laboratory procedures and test results are presented in Appendix A.

### *3.4 DCP TESTING*

We conducted DCP testing at all boring locations on the existing pavement and 7 out of 13 borings within the new development on May 13, 2021. We conducted tests in general accordance with ASTM D6951 and summarized the data by plotting depth of penetration versus blow count. Plots of the summarized DCP test data are presented in Appendix C. We used the data and method described in Appendix C to estimate resilient modulus. A summary of the estimated subgrade resilient modulus at each test location is presented in Table 2.

**Table 2. Estimated Subgrade Resilient Modulus Based on DCP Testing**

| Boring Number | Resilient Modulus (psi) |
|---|---|
| C-1 | 4,180 |
| C-2 | NA (refusal) |
| C-3 | NA (refusal) |
| C-4 | NA (refusal) |
| C-5 | 17,200 |
| B-1 | 17,110 |
| B-3 | 17,070 |
| B-5 | 20,200 |
| B-6 | 18,600 |
| B-7 | 8,120 |
| B-9 | 16,190 |
| B-10 | 36,080 |

### *3.5 PAVEMENT DESIGN*

We used the guidance presented in the *ODOT Pavement Design Guide* (ODOT, 2019; herein referred to as the ODOT guide) and the AASHTO *Guide for Design of Pavement Structures* (AASHTO, 1993; herein referred to as the AASHTO guide) in preparing our pavement designs. The subgrade resilient modulus is based on subsurface explorations and DCP testing. Traffic loading is based on the ADT forecast for the year 2042 provided by DKS Associates. We evaluated designs based on 15- and 20-year pavement service. Descriptions of our input parameters and the recommended pavement designs are summarized below.

#### 3.5.1 Design ESAL Values

We estimated the design ESAL values based on the ADT forecast for the year 2042 provided by DKS Associates for the proposed road segment. The estimated 15- and 20-year design ESAL values are 136,000 and 181,000, respectively. Details of our method for calculating ESALs and our calculation sheet are presented in Appendix D.

#### 3.5.2 Subgrade Resilient Modulus

As noted in Table 2, the resilient modulus of the subgrade is highly variable based on DCP testing. In addition, some of the DCP tests met with refusal within the fill material or shallow gravel. Accordingly, we recommend a design resilient modulus value of 8,000 psi to account for the variable subgrade conditions.

### 3.5.3 Structural Layer Coefficients for Existing Pavement

We used observations during pavement coring to estimate the layer coefficient for the existing AC in conjunction with Table 5.2 in Part III, Section 5.4.5 in the AASHTO design guide. We used the base layer modulus of 13,000 psi to estimate the layer coefficients for the base layers using Figure 2.6 in Part II, Section 2.3.5 in the AASHTO design guide. The existing AC and base layer coefficients are estimated to be 0.34 and 0.06, respectively.

### 3.5.4 Required Structural Number

We used the procedure in the AASHTO guide to determine the required structural number for new pavement comprised of new AC overlying new aggregate base overlying the existing subgrade along the project. We calculated required 15- and 20-year structural numbers of 2.38 and 2.49, respectively, based on the estimated 15- and 20-year ESAL values of 136,000 and 181,000, respectively, a subgrade resilient modulus value of 8,000 psi, and the other design parameters discussed in the following section. We recommend pavement design using a structural number of 2.49.

### 3.5.5 Other Design Parameters

Other pavement design parameters used in our analysis are as recommended by the ODOT guide and/or the AASHTO guide. These input parameters are summarized as follows:

- Reliability of 85 percent for the collector road section and a reliability of 80 percent for the local road section
- Overall standard deviation value of 0.49
- Initial and terminal serviceability values of 4.2 and 2.5, respectively
- Structural layer coefficients of 0.42 and 0.10 for new AC and new aggregate base, respectively

## 4.0 CONCLUSIONS AND RECOMMENDATIONS

### *4.1 DISCUSSION*

Based on the results of our explorations, laboratory testing, and analyses, it is our opinion that the proposed improvements can be constructed, provided the recommendations in this report are followed. The primary geotechnical considerations are as follows:

- Groundwater was encountered at relatively shallow depths between 3.5 feet to greater than 16.5 feet BGS. Groundwater perches on the shallow basalt, where present, and likely corresponds closely with the adjacent level of the Columbia River where basalt is deeper. Sand fill was encountered in many of the explorations and native sand is also present. Considering the relatively shallow groundwater conditions and sand soil, sloughing, caving, and “running sand” will likely occur if groundwater is encountered in excavations. Accordingly, the contractor should expect to shore excavations with a positive pressure system where groundwater is encountered. External dewatering wells may also be required in addition to shoring to maintain the stability of excavations.

- We encountered very dense, weathered basalt gravel or basalt at shallow depths in many of the borings. Based on rock coring in other borings, the very dense basalt gravel quickly transitions to basalt or represents the top of the basalt. The basalt is generally shallow near the western edge of the vacant site area and increases with depth toward the Columbia River. The depths to refusal blow counts in very dense, weathered basalt gravel or basalt are indicated on Figure 2. The basalt is generally medium hard to hard and moderately to very intensely fractured. Shallow excavations into the basalt may be possible with large earthwork equipment. If excavations in the basalt cannot be ripped with large excavation equipment, rock excavation methods such as pneumatic hammering, drilling and hydraulic splitting, or drilling and blasting will be necessary. We recommend the project budget include a contingency for some rock excavation.

- We encountered fill in most of the explorations to depths of up to 16.5 feet BGS at the locations explored. The fill is variable in nature and contains silt and clay, gravel, sand, and concrete. Difficult excavation may be encountered where oversize material, such as cobbles, boulders, or debris, is encountered during excavations in the fill. Excavation volumes for utility trenches may also be greater than typical due to sloughing, caving, and the need to remove oversize materials.

- The surficial soil is highly variable due to the past use and fill within the area. If not carefully executed, site preparation, grading, utility trench work, and roadway excavation in the fine-grained or silty soil, where present, can create extensive soft areas and significant subgrade repair costs can result.

- Based on the sand fill materials encountered in our explorations and anticipated groundwater depths, there is a risk of liquefaction from design-level seismic events for most of the site. We estimate the liquefaction-induced settlement and lateral spreading potential from a design-level seismic event at the planned pump station on the western side of South 1st Street, near our borings B-8 and B-13, will both be less than 1 inch considering the depth to basalt and distance to the river. We recommend the lightly loaded buildings for restrooms and/or the planned pump station can be supported on conventional shallow foundations constructed on minimum 12-inch-thick granular pads.

The following sections present specific geotechnical recommendations for design and construction of the proposed improvement project.

### *4.2 EROSION CONTROL*

When exposed, the soil at this site can easily be eroded by wind and water; therefore, erosion control measures should be carefully planned and in place before construction begins. Existing pavement and aggregate base should be left in place, wherever feasible, to protect the ground surface. Measures employed to reduce erosion include the use of silt fences, hay bales, buffer zones of natural growth, sedimentation ponds, and granular haul roads.

### *4.3 SITE PREPARATION*

#### 4.3.1 General

Earthwork operations should be planned and executed to minimize subgrade disturbance. Managing construction traffic and protection of exposed soil subgrades is the responsibility of the contractor.

#### 4.3.2 Demolition

The limits of the required demolition should be determined by the project engineer, although they should include all improvements that will impede construction of the improvements. Demolished material should be transported off site for disposal, except as noted in the "Structural Fill" section.

Within structural areas (new pavement, foundations, or fills), excavations resulting from removing existing foundations, tanks, and other subsurface elements should be backfilled with compacted engineered fill that meets the criteria outlined in the "Structural Fill" section and as described in OSSC 00310 (Removal of Structures and Obstructions). In addition, the bottom of the excavations should expose firm subgrade before filling. The sides of the excavations should be cut into firm material and sloped at a minimum gradient of 1H:1V to allow for more uniform compacting of backfill. Utility lines abandoned under new structural components should be completely removed or (with written approval) grouted full if left in place. Soft or loose soil encountered in existing utility line excavations should be removed and replaced with structural fill.

#### 4.3.3 Subgrade Evaluation

Upon completion of stripping (if required) and prior to the placement of any foundation, fill, or pavement, the exposed subgrade should be evaluated by proof rolling. The subgrade should be proof rolled with a fully loaded dump truck or similar heavy rubber-tire construction equipment to identify soft, loose, or unsuitable areas. The proof roll should be completed in conformance with the specifications provided in OSSC 00330.43 (Earthwork Compaction Requirements). Qualified personnel should observe proof rolling to evaluate yielding of the ground surface. Areas that appear to be too wet and soft to support proof rolling equipment should be evaluated by probing and prepared in accordance with the recommendations for wet weather construction presented in the "Construction Considerations" section.

#### 4.3.4 Construction Considerations

The fine-grained and silty soil, where present at the site, is easily disturbed when wet. If not carefully executed, site preparation, utility trench work, and roadway excavation will create extensive soft areas and significant subgrade repair costs will result. The construction methods and schedule should be carefully considered with respect to preventing trafficking on the subgrade to reduce the need to over-excavate disturbed or softened soil. Existing asphalt and rock surfaces should remain in place as long as possible to carry construction traffic. Where fine-grained and silty soil subgrades are present during wet weather or when the moisture contents are more than a few percentage points above optimum, traffic should be limited to track-mounted equipment or granular haul roads and staging areas should be used.

The thickness of the granular material for haul roads and staging areas will depend on the amount and type of construction traffic and should be selected by the contractor. In general, a 12-inch-thick mat of granular material is sufficient for light staging areas but is generally not expected to be adequate to support heavy equipment or truck traffic. The granular mat for haul roads and areas with repeated heavy construction traffic typically needs to be increased up to 18 inches. The actual thickness of haul roads and staging areas should be based on the contractor's approach to site development and the amount and type of construction traffic. The imported granular material should meet the requirements provided in the "Structural Fill" section. Stabilization material may be used as a substitute, provided the top 4 inches of material consists of imported granular material. The requirements for stabilization material are provided in the "Structural Fill" section. A geotextile can be placed as a barrier between silty subgrade material and imported granular material in areas of repeated construction traffic for additional support.

## *4.4 EXCAVATION*

### 4.4.1 General

The contractor should be aware of, and become familiar with, applicable local, state, and federal safety regulations, including current OSHA excavation and trench safety standards. Excavations shall be completed in conformance with the relevant sections of the OSSC, including, but not limited to, OSSC 00330 (Earthwork), OSSC 00400 (Drainage and Sewers), and OSSC 00500 (Bridges). Construction site safety, including the means, methods, and sequencing of construction operations, is the sole responsibility of the contractor. The information provided below is general in nature and should not be relied upon by the contractor during construction without their own evaluation of excavation stability.

Considering the relatively shallow groundwater conditions and sand soil, sloughing, caving, and "running sand" will likely occur if groundwater is encountered in excavations. Accordingly, the contractor should expect to shore excavations with a positive pressure system where groundwater is encountered. External dewatering wells may also be required in addition to shoring to maintain the stability of excavations.

The soil at the site should be readily excavatable with conventional grading equipment, with the exception of the basalt. The basalt is generally medium hard to hard and moderately to very intensely fractured. Shallow excavations into the basalt may be possible with large earthwork equipment. If excavations in the basalt cannot be ripped with large excavation equipment, rock excavation methods such as pneumatic hammering, drilling and hydraulic splitting, or drilling and blasting will be necessary. We recommend the project budget include a contingency for some rock excavation.

Debris and oversize materials may also be encountered in the existing fill. Excavations into these materials may result in localized difficult excavation. Also, the presence of these materials may cause temporary excavations to cave or slough, resulting in greater than anticipated backfill quantities.

### 4.4.2 Trenches

Temporary construction excavations in the soil should stand vertical for short periods of time to a depth of approximately 4 feet, provided groundwater seepage, cobbles, boulders, or debris 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 1½H:1V and groundwater seepage does not occur. Sloughing, caving, and "running sand" will likely occur for excavations below the groundwater table. If seepage is encountered, the walls of the trench will likely need to be shored with a positive pressure system and the area dewatered using external dewatering wells to maintain stability. 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.

### 4.4.3 Dewatering

Dewatering systems are best designed by the contractor. Because of the tendency for sand and sandy soil to "run," dewatering measures will likely require well points or pump wells located outside of the trench excavation. However, it may be possible to use a sump located within the trench excavations to dewater isolated zones of perched water or shallow limited excavations below the water table.

Flow rates for dewatering are likely to vary depending on location, soil type, and the season during which the excavation occurs. Dewatering systems should be capable of adapting to variable flows. If groundwater is encountered and the base of excavations does not extend down to very dense gravel or basalt, we recommend over-excavating the subgrade by 12 to 18 inches and placing stabilization rock in the base. Specifications for stabilization material are provided in the "Structural Fill" section.

## *4.5 PERMANENT SLOPES*

Permanent cut or fill slopes should not exceed a gradient of 2H:1V, unless specifically evaluated for stability. Slopes that will be maintained by mowing should not be constructed steeper than 3H:1V. 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.

## *4.6 STRUCTURAL FILL*

### 4.6.1 General

A variety of material may be used as structural fill at the site. Fill should only be placed over subgrade that has been prepared in conformance with the "Site Preparation" section. Structural fill should meet the specifications provided in OSSC 00330 (Earthwork), OSSC 00400 (Drainage and Sewers), and OSSC 02600 (Aggregates), depending on the application. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill are provided below.

### 4.6.2 On-Site Soil

The near-surface soil at the site consists primarily of sand and gravel with variable silt content. The granular soil at the site should be suitable for use as general fill, provided it is properly moisture conditioned, free of organic material and particles over 6 inches in diameter, and meets the specifications provided in OSSC 00330.12 (Borrow Material).

Drying will likely be required to achieve adequate compaction for the on-site soil, except during the dry season when wetting may be needed. It may not be possible to adequately moisture condition (dry) the soil for use as structural fill during the wet season or wet weather conditions. When used as structural fill, native soil should be placed in lifts with a maximum uncompacted thickness of 6 to 8 inches and compacted to not less than 95 percent of the maximum dry density, as determined by AASHTO T 99.

#### 4.6.3 Imported Granular Material

Imported granular material used as structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.14 (Selected Granular Backfill) or OSSC 00330.15 (Selected Stone Backfill). The imported granular material should also be angular, should be fairly well graded between coarse and fine material, should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have at least two mechanically fractured faces.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 8 inches and compacted to not less than 95 percent of the maximum dry density, as determined by AASHTO T 99. During the wet season or when wet subgrade conditions exist, the initial lift should be approximately 12 to 18 inches in uncompacted thickness and should be compacted by rolling with a smooth-drum roller without using vibratory action.

Where the imported granular material is placed atop a fine-grained subgrade, a geotextile should be placed as a barrier between the native soil subgrade and the imported granular material. Subgrade geotextile should conform to OSSC Table 02320-1 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.

#### 4.6.4 Stabilization Material

Stabilization material should consist of pit- or quarry-run rock, crushed rock, or crushed gravel and should meet the specifications provided in OSSC 00330.16 (Stone Embankment Material). In addition, 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 18 inches thick and be compacted to a firm condition.

Where the stabilization material is used to stabilize soft subgrade beneath construction haul roads or staging areas, a geotextile should be placed as a barrier between the soil subgrade and the imported granular material. Subgrade geotextile should conform to OSSC Table 02320-1 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

### 4.6.5 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 well-graded granular material with a maximum particle size of 1 inch and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve and meet the specifications provided in OSSC 00405.13 (Pipe Zone Material). The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by AASHTO T 99, 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 well-graded granular material with a maximum particle size of 2½ inches and less than 7 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet OSSC 00405.14 (Trench Backfill; Class B, C, or D). This material should be compacted to at least 95 percent of the maximum dry density, as determined by AASHTO T 99, or as required by the pipe manufacturer or local building department.

### 4.6.6 Aggregate Base

Imported granular material used as aggregate base should be clean, crushed rock or crushed gravel and sand that are well graded. The aggregate base should meet the gradation defined in OSSC 00640 (Aggregate Base and Shoulders), with the exception that the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, a maximum particle size of 1½ inches, and 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 AASHTO T 99.

### 4.6.7 Existing AC Pavement, Concrete, and Aggregate Base

AC pavement, concrete, and aggregate base from the existing roadways can be used in general structural fill, provided it is environmentally suitable, contains no particles greater than 6 inches in diameter, is thoroughly mixed so that there are no voids, and is allowed by the jurisdiction. This material should only be used at depths greater than 3 feet below the finished subgrade in general fill areas and at least 3 feet above the pipe zone in trenches. The recycled material should meet the specifications provided in OSSC 00330.12 (Borrow Material) and other appropriate specifications.

### 4.6.8 AC

The AC should be ½-inch, Level 2 ACP according to OSSC 00744 (Asphalt Concrete Pavement). If lift thicknesses outside this range are requested, we recommend additional consultation and communication between the City and the design team.

### 4.6.9 Geotextile Fabric

#### *4.6.9.1 Subgrade Geotextile*

The subgrade geotextile should conform to OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

#### *4.6.9.2 Pavement Geotextile*

Paving geotextile placed across widening should be 12 to 24 inches wide and be self-adhesive. General pavement geotextile properties should meet or exceed the properties in OSSC Table 02320-6 for Pavement Overlay Geotextile.

## *4.7 DRAINAGE*

During grading of the roadways, the contractor should be responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface. Surface runoff should not be allowed to flow over the face of the exposed soil.

## *4.8 PAVEMENT*

### 4.8.1 Overview

Based on the results of pavement borings, DCP testing, and analyses, it is our opinion that the existing pavement section needs improvement to accommodate forecasted 20-year design traffic loading. Rehabilitation through grind and inlay to remove some of the existing surface distresses and improve capacity is feasible; however, reflection cracking is expected to develop within the pavement life. If a slight grade increase can be accommodated, then an overlay layer can significantly improve pavement life. The pavement layers recommended below should conform to the specifications presented in the "Structural Fill" section.

### 4.8.2 Rehabilitation

For rehabilitation of the existing pavement section on the paved City-owned roadways, we recommend a grind and inlay depth of 2.0 inches to remove most of the existing cracks. However, based on crack depths observed during our explorations, reflection cracks will develop within the life of the pavement. If a slight grade increase can be accommodated, we recommend an additional 1.0-inch-thick overlay to significantly improve the design life.

**Grind and Inlay – 10- to 15-year design life**

- 2.0-inch cold plane milling
- 2.0-inch-thick, ½-inch ACP, Level 2, PG 64-22

**Option – Grind and Overlay – 1.0-inch grade increase – 20-year design life**

- 2.0-inch cold plane milling
- 3.0-inch-thick, ½-inch ACP, Level 2, PG 64-22

### 4.8.3 New Construction

For the proposed road section in the new development and for any existing sections that require widening, we recommend a new pavement section of 5.0 inches of AC over 6.0 inches of aggregate base. The widening surface lift and the inlay of the existing pavement should be completed at the same time with either no surface paving joint or a paving joint offset a minimum of 12 inches from the widening joint. Our recommended construction thickness for new and widening sections are as follows:

**Widening/New Construction (5.0 inches of AC over 6.0 inches of aggregate base)**

- 2.0 inches of Level 3, ½-inch ACP, PG 64-22 (surface course)
- 3.0 inches of Level 3, ½-inch ACP, PG 64-22 (base course)
- Pavement overlay geotextile (at connection to existing AC, see below)
- 6.0 inches of aggregate base
- Stabilization aggregate (if required)
- Subgrade geotextile

The pavement overlay geotextile should be placed on the AC surface underlying the wearing course over joints between existing and new pavement. Joints should be located outside of areas of wheel wander and preferably either at middle of lane or at pavement striping locations.

### *4.9 FOUNDATION SUPPORT*

Based on the sand fill materials encountered in our explorations and anticipated groundwater depths, there is a risk of liquefaction from design-level seismic events for most of the site. Considering the presumed or observed depth to basalt of 15.4 to 17 feet BGS and subsurface and groundwater conditions encountered in borings B-8 and B-13, we anticipate liquefaction-induced settlement from a design-level seismic event near the planned pump station on the western side of the planned South 1st Street extension will be less than 1 inch. Considering the distance of greater than 400 feet to the river and limited zone of potentially liquefiable soil below the groundwater depth, which was encountered at 9 feet BGS in boring B-8 in early May 2021, lateral spreading at the planned pump station location is also expected to be less than 1 inch. We recommend small, lightly loaded buildings for restrooms and/or the planned pump station (near our borings B-8 and B-13) can be supported on conventional shallow foundations constructed on minimum 12-inch-thick granular pads. A thorough evaluation of the liquefaction-induced settlements and lateral spreading movements at different locations across the site was beyond the scope of our study.

#### 4.9.1 Shallow Foundations

Small, lightly loaded buildings for restrooms and/or the planned pump station on the western side of the South 1st Street extension or non-building structures in other areas of the site for which liquefaction-induced settlement and associated lateral spreading is acceptable without presenting a life and safety hazard can be supported on conventional shallow foundations bearing on minimum 12-inch-thick granular pads. Deeper excavations for thicker granular pads may be necessary if deleterious materials are encountered. The granular pads should extend a minimum of 6 inches beyond the footing perimeter for every foot excavated below the base of the footings. The granular pads should be constructed using imported granular material as described in the "Structural Fill" section.

Continuous and isolated spread footings should be at least 16 and 20 inches wide, respectively. The bottoms of exterior footings should be at least 18 inches below the lowest adjacent exterior grade. The bottoms of interior footings should be established at least 12 inches below the base of the slab. Footings established on granular pads as recommended above should be sized based on an allowable bearing pressure of 2,000 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 can be increased by one-half for short-term loads such as those resulting from wind or seismic forces.

Based on our analysis and experience with similar soil, total post-construction consolidation-induced settlement under static conditions should be less than 1 inch, with differential settlement of less than ½ inch between footings. This does not include liquefaction-induced settlement that may occur during the design earthquake.

### 4.9.2 Resistance to Sliding

Lateral loads on foundations can be resisted by passive earth pressure on the sides of the structure and by friction on the base. Our analysis indicates that the available passive earth equivalent fluid pressure for footings confined by on-site soil and structural fill is 325 pcf. Typically, the movement required to develop the available passive resistance may be relatively large; therefore, we recommend using a reduced passive equivalent fluid pressure of 250 pcf. Adjacent floor slabs, pavement, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. In addition, in order to rely on passive resistance, a minimum of 10 feet of horizontal clearance must exist between the face of the footings and adjacent downslopes. For foundations in contact with imported granular material, a coefficient of friction equal to 0.40 may be used when calculating resistance to sliding.

### 4.9.3 Seismic Parameters

Seismic design is prescribed by ASCE 7-16. Table 3 presents the site design parameters prescribed by ASCE 7-16 for the site assuming structures are supported as recommended in this section and the risk of liquefaction-induced settlement and lateral spreading is acceptable. Due to the presence of liquefiable soil, the Site Class is F; however, the design parameters for Site Class D provided below can be used per ASCE 7-16, provided the fundamental period of the structure is 0.5 second or less.

**Table 3. Seismic Design Parameters**

| Parameter | Short Period ($T_s$ = 0.2 second) | 1 Second Period ($T_1$ = 1.0 second) |
|---|---|---|
| MCE Spectral Acceleration, S | $S_s$ = 0.830 g | $S_1$ = 0.399 g |
| Site Class | F* | |
| Site Coefficient, F | $F_a$ = 1.20 | $F_v$ = 1.90 |
| Adjusted Spectral Acceleration, $S_M$ | $S_{MS}$ = 0.996 g | $S_{M1}$ = 0.758 g |
| Design Spectral Response Acceleration Parameters, $S_D$ | $S_{DS}$ = 0.664 g | $S_{D1}$ = 0.505 g |

* The above parameters provided for Site Class D can be used, provided the structure has a fundamental period of 0.5 second or less per ASCE 7-16 Section 20.3.1 and the seismic response coefficient ($C_s$) is determined according to the exception in ASCE 7-16 Section 11.4.8 or else a site-specific response analysis will be required.

### 4.9.4 Floor Slabs

Satisfactory subgrade support for building floor slabs supporting up to 100 psf floor loading can be obtained, provided the subgrade is prepared in accordance with the "Site Preparation" section. A minimum 6-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade to assist as a capillary break. The floor slab base rock should be crushed rock or crushed gravel and sand meeting the requirements for aggregate base outlined in the "Structural Fill" section. The aggregate base should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. Floor slab base rock contaminated with excessive fines (greater than 5 percent by dry weight passing the U.S. Standard No. 200 sieve) should be replaced.

Flooring manufacturers often require vapor barriers to protect flooring and flooring adhesives. Many flooring manufacturers will warrant their product only if a vapor barrier is installed according to their recommendations. Selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team. We can provide additional information to assist you with your decision.

## 5.0 OBSERVATION OF CONSTRUCTION

Satisfactory earthwork and pavement performance depends to a large degree on the 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 determine 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 preparation, final proof rolling of the pavement subgrade and base rock, and AC placement and compaction, and performing laboratory compaction and field moisture-density tests.

## 6.0 LIMITATIONS

We have prepared this report for use by the City of St. Helens, Otak, and the design and construction teams for the proposed project. The 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 sites.

Exploration observations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata, pavement, 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 improvement 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, 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 of our services 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, expressed 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

Shashwath Sreedhar, E.I.T., Ph.D.
Technical Specialist/Engineer

Shawn M. Dimke, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
63114PE
OREGON
March 14, 2006
SHAWN DIMKE
EXPIRES: 12/31/23

## REFERENCES

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ASCE, 2016. *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. ASCE Standard ASCE/SEI 7-016.

Evarts, R. C., 2004. *Geologic Map of the Saint Helens Quadrangle, Columbia County, Oregon, and Clark and Cowlitz Counties, Washington.* U.S. Geological Survey Scientific Investigations Map 2834, scale: 1:24,000.

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

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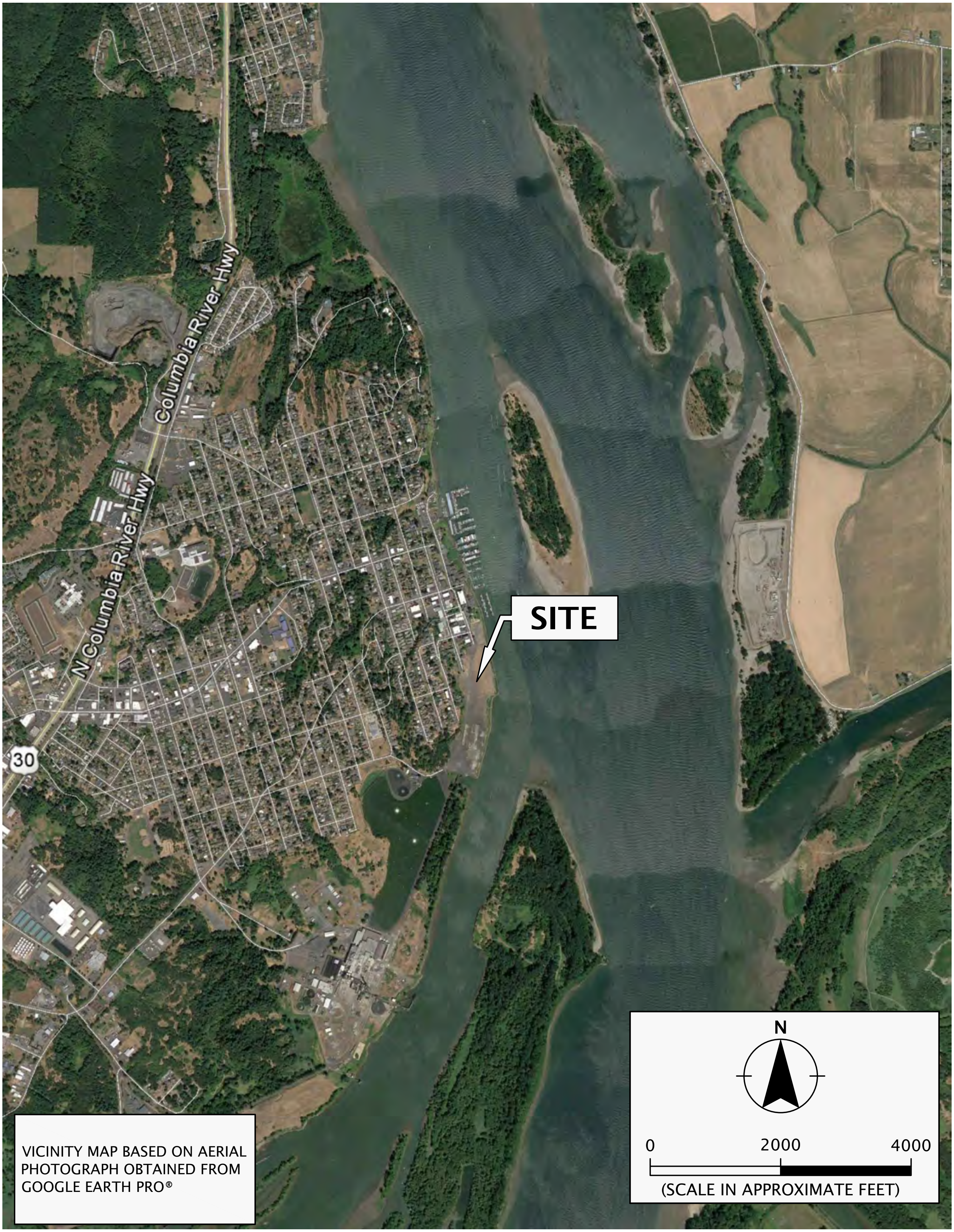
# FIGURES




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

| | | | |
|---|---|---|---|
| NV5 | STHELENS-3-01 | VICINITY MAP | |
| | JANUARY 2022 | 1ST AND STRAND STREETS<br>ST HELENS, OR | FIGURE 1 |

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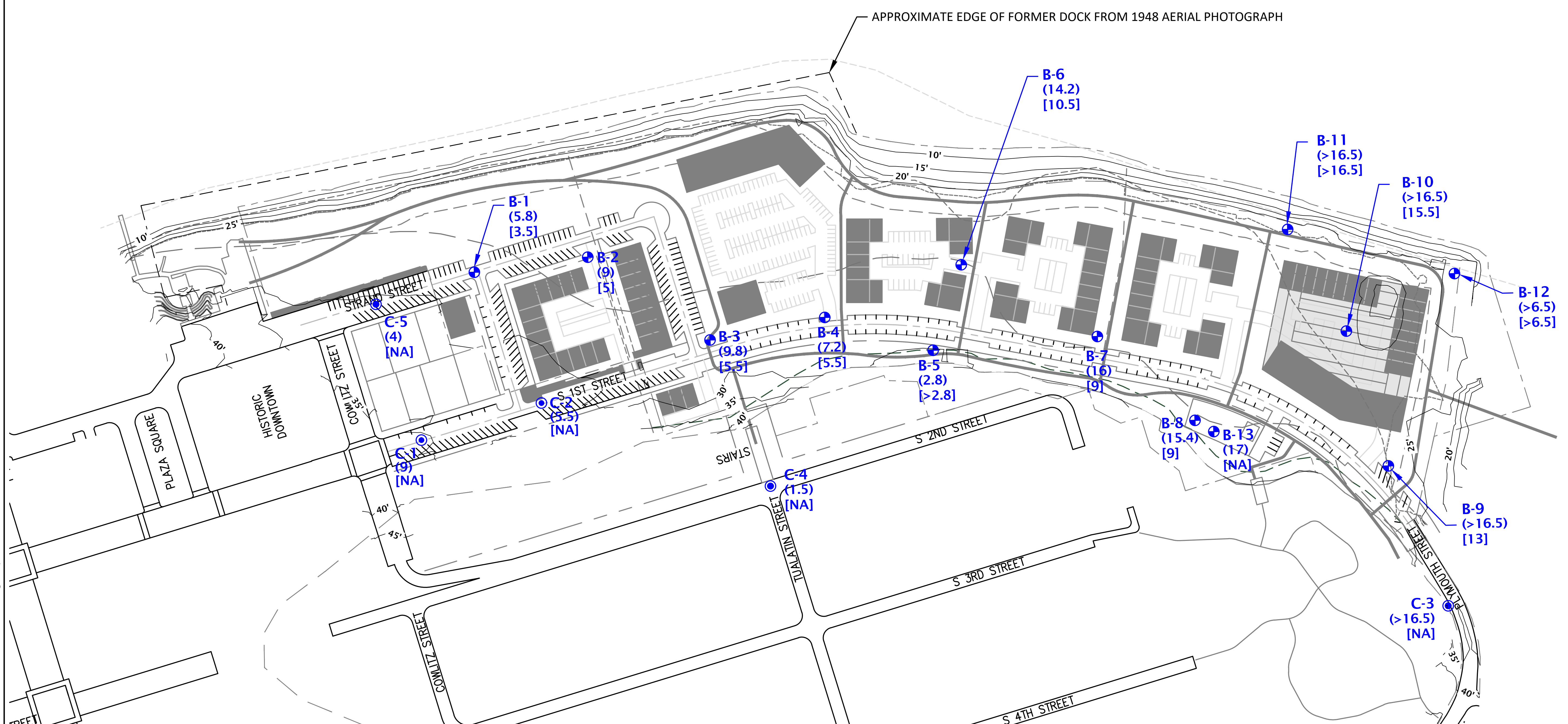



**LEGEND:**

- B-1 BORING
- C-1 PAVEMENT CORE BORING
- (5.8) DEPTH TO PRACTICAL DRILLING REFUSAL ON BASALT (FEET BGS)
- [3.5] DEPTH TO GROUNDWATER DURING DRILLING (FEET BGS)
- EXISTING TOPOGRAPHY (ELEVATIONS RELATIVE TO NORTH AMERICAN VERTICAL DATUM 1988)

SITE PLAN BASED ON DRAWING OBTAINED FROM OTAK ON JUNE 24, 2021.

**SITE PLAN**

1ST AND STRAND STREETS
ST HELENS, OR

STHELENS-3-01

JANUARY 2022

NV5

**FIGURE 2**

# APPENDIX A

## APPENDIX A

## FIELD EXPLORATIONS

### *GENERAL*

We explored subsurface conditions at the site by drilling 13 borings (B-1 through B-13) and drilling 5 borings (C-1 through C-5) with surface pavement cores between May 3 and 12, 2021 and on July 26, 2021. The borings were drilled to depths between 2.8 and 32 feet BGS by Western States Soil Conservation, Inc. of Hubbard, Oregon, under the supervision of NV5 personnel. The borings were completed using direct-push, mud rotary, pavement core drilling, and HQ core drilling methods as indicated on the logs.

We chose the locations of the explorations based on information provided by the design team. The locations of the explorations were determined using a GPS application on a mobile phone. Some locations were adjusted slightly relative to nearby surrounding features. This information should be considered accurate only to the degree implied by the methods used. Approximate exploration locations are shown on Figure 2. The exploration logs are presented in this appendix.

### *SOIL AND ROCK SAMPLING*

Samples were collected from the borings using a 1½-inch-inside diameter, split-spoon sampler (SPT sampler). The split-spoon sampling was conducted in general accordance with ASTM D 1586. The 1½-inch-inside diameter, split-spoon samplers were driven into the soil with 140-pound automatic trip hammer free falling 30 inches. The samplers were driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the boring logs, unless otherwise noted. Representative grab samples were also collected from the Geoprobe® sleeve of the direct-push borings. Sampling methods and intervals are shown on the exploration logs.

Rock was cored continuously using HQ core drilling methods in general accordance with ASTM D2113. Percent core recovery and RQD are noted on the boring logs. RQD is determined by summing the length of intact pieces of core longer than 4 inches and dividing by the length of the core advance. Photographs of the rock cores are presented in this appendix.

The average efficiency of the automatic SPT hammers used by Western States Soil Conservation, Inc. are 78.4 percent for rig #1, 85.6 percent for rig #3, 69.2 percent for rig #4, and 82.2 percent for rig #8. Rig #8 was used to advance the SPT samples for borings B-1 through B-12, C-1, and C-2. Rig #4 was used to advance the SPT samples for borings C-3 and C-4. Rig #1 was used to advance the SPT samples for boring C-5. Rig# 3 was used to advance the SPT samples for boring B-13. The calibration testing results are presented at the end of this appendix.

### *SOIL AND ROCK CLASSIFICATION*

The soil and rock samples were classified in the field in accordance with the “Exploration Key” (Table A-1), “Soil Classification System” (Table A-2), and “Rock Classification System” (Table A-3), which are included in this appendix. The exploration logs indicate the depths at which the soil and rock 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

Laboratory testing was conducted on select soil samples to confirm field classifications and determine the index engineering properties and strength characteristics. Descriptions of the testing completed are presented below.

### *MOISTURE CONTENT*

We tested the natural moisture content of select soil samples 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.

### *ATTERBERG LIMITS TESTING*

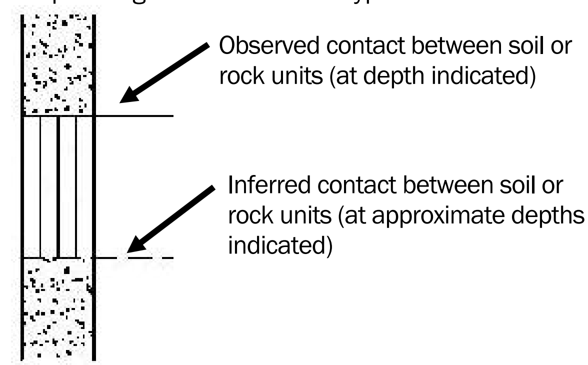
The plastic limit and liquid limit (Atterberg limits) of select soil samples were determined in accordance with ASTM D4318. The Atterberg limits and the plasticity index were completed to aid in the classification of the soil. The plastic limit is defined as the moisture content (in percent) where the 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.

### *PARTICLE-SIZE ANALYSIS*

Particle-size analysis were 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.

| 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




| 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** |

| HARDNESS | DESCRIPTION |
|---|---|
| Extremely soft (R0)<br>Very soft (R1)<br>Soft (R2)<br>Medium hard (R3)<br>Hard (R4)<br>Very hard (R5) | Indented by thumbnail<br>Can be peeled by pocket knife or scratched with finger nail<br>Can be peeled by a pocket knife with difficulty<br>Can be scratched by knife or pick<br>Can be scratched with knife or pick only with difficulty<br>Cannot be scratched with knife or sharp pick |
| **WEATHERING** | **DESCRIPTION** |
| Decomposed<br>Predominantly decomposed<br>Moderately weathered<br>Slightly weathered<br>Fresh | Rock mass is completely decomposed<br>Rock mass is more than 50% decomposed<br>Rock mass is decomposed locally<br>Rock mass is generally fresh<br>No discoloration in rock fabric |
| **JOINT SPACING** | **DESCRIPTION** |
| Very close<br>Close<br>Moderate close<br>Wide<br>Very wide | Less than 2 inches<br>2 inches to 1 foot<br>1 foot to 3 feet<br>3 feet to 10 feet<br>Greater than 10 feet |
| **FRACTURING** | **FRACTURE SPACING** |
| Very intensely fractured<br>Intensely fractured<br>Moderately fractured<br>Slightly fractured<br>Very slightly fractured<br>Unfractured | Chips and fragments with a few scattered short core lengths<br>0.1 foot to 0.3 foot with scattered fragments intervals<br>0.3 foot to 1 foot with most lengths 0.6 foot<br>1 foot to 3 feet<br>Greater than 3 feet<br>No fractures |
| **HEALING** | **DESCRIPTION** |
| Not healed<br>Partly healed<br>Moderately healed<br>Totally healed | Discontinuity surface, fractured zone, sheared material or filling not re-cemented<br>Less than 50% of fractured or sheared material<br>Greater than 50% of fractured or sheared material<br>All fragments bonded |

| | | |
|---|---|---|
| NV5 | **ROCK CLASSIFICATION SYSTEM** | **TABLE A-3** |

| DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100) | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|---|
| 0.0 | | Medium dense, light gray SAND with silt (SP-SM); moist, sand is medium - **FILL**. | | | | | |
| 2.5 | | orange; wet at 3.5 feet | | | | ● 18 | 3.5 feet, during drilling |
| | | with gravel at 4.0 feet | | | | | |
| 5.0 | | Medium stiff, gray CLAY (CH), minor to with gravel, minor sand; wet. | 4.5 | | | ● ▲ 3-50/3" | |
| | | Very dense, dark gray GRAVEL with clay (GP-GC); moist to wet (weathered basalt). | 5.5 | | | | Surface elevation was not measured at the time of exploration. |
| | | Exploration terminated at a depth of 5.8 feet due to refusal. | 5.8 | | | | |
| 7.5 | | Hammer efficiency factor is 82.2 percent. | | | | | |
| 10.0 | | | | | | | |
| 12.5 | | | | | | | |
| 15.0 | | | | | | | |
| 17.5 | | | | | | | |
| 20.0 | | | | | | | |

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: J. Hook COMPLETED: 05/03/21

BORING METHOD: direct push (see document text) BORING BIT DIAMETER: 2 1/2 inches

NV5

STHELENS-3-01

JANUARY 2022

**BORING B-1**

1ST AND STRAND STREETS
ST. HELENS, OR

**FIGURE A-1**

| DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100) | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|---|
| 0.0 | | AGGREGATE BASE (12.0 inches). | | | | | |
| | | Medium dense, light gray SAND with silt (SP-SM), trace gravel and organics (woody debris); moist, sand is medium - **FILL**. | 1.0 | | | | |
| 2.5 | | | | P200 | | 14 | P200 = 6% |
| 5.0 | | very loose; wet at 5.0 feet | | | | 2 | ◁ 5.0 feet, during drilling |
| 7.5 | | | | | | 50/6" | Heaving sand at 7.5 feet. |
| | | Very dense, dark gray GRAVEL with clay (GP-GC), trace sand; wet (weathered basalt). | 8.0 | | | | |
| | | Exploration terminated at a depth of 9.0 feet due to refusal.<br><br>Hammer efficiency factor is 82.2 percent. | 9.0 | | | | Surface elevation was not measured at the time of exploration. |
| 10.0 | | | | | | | |
| 12.5 | | | | | | | |
| 15.0 | | | | | | | |
| 17.5 | | | | | | | |
| 20.0 | | | | | | | |

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: J. Hook COMPLETED: 05/03/21

BORING METHOD: direct push (see document text) BORING BIT DIAMETER: 2 1/2 inches

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100) | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|
| 0.0 | Medium dense, light gray-brown, silty GRAVEL with sand (GM); moist, gravel is fine and angular - **FILL**. | | | | | |
| 2.5 | | | | | | |
| | Medium dense, light gray SAND with silt (SP-SM); moist, sand is fine to medium - **FILL**. | 3.0 | | | 20 | ⊻ 5.5 feet, during drilling |
| 5.0 | loose at 5.0 feet | | | | | |
| | Soft, gray to gray-brown ORGANIC SILT/WOOD (OL), minor sand; wet - **FILL**. | 5.5 | | | 3 | |
| 7.5 | Stiff, dark brown CLAY with sand and gravel (CL), minor silt and organics (woody debris); wet. | 7.0 | | | 13 | |
| | Very dense, dark gray GRAVEL with clay (GP-GC); wet (weathered basalt). | 9.5 | | | 50/3" | |
| 10.0 | Exploration terminated at a depth of 9.8 feet due to refusal. | 9.8 | | | | Surface elevation was not measured at the time of exploration. |
| | Hammer efficiency factor is 82.2 percent. | | | | | |
| 12.5 | | | | | | |
| 15.0 | | | | | | |
| 17.5 | | | | | | |
| 20.0 | | | | | 0 50 100 | |

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: J. Hook COMPLETED: 05/03/21

BORING METHOD: direct push (see document text) BORING BIT DIAMETER: 2 1/2 inches

NV5

STHELENS-3-01

JANUARY 2022

**BORING B-3**

1ST AND STRAND STREETS
ST. HELENS, OR

**FIGURE A-3**

| DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100) | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|---|
| 0.0 | | Dense, gray to dark gray-brown, silty GRAVEL with sand (GM), trace debris (metal) and organics (woody debris); moist - **FILL**. | | | | | |
| 2.5 | | | | | | 34 | ▽ 5.5 feet, during drilling |
| 5.0 | | very loose at 5.0 feet | | | | | |
| | | Light brown WOOD/SAWDUST - **FILL**. | 5.5 | | | 2 | |
| | | Very dense, dark gray GRAVEL with clay (GP-GC), trace to minor sand; wet (weathered basalt). | 7.0 | | | 50/2" | Surface elevation was not measured at the time of exploration. |
| 7.5 | | Exploration terminated at a depth of 7.2 feet due to refusal. | 7.2 | | | | |
| | | Hammer efficiency factor is 82.2 percent. | | | | | |
| 10.0 | | | | | | | |
| 12.5 | | | | | | | |
| 15.0 | | | | | | | |
| 17.5 | | | | | | | |
| 20.0 | | | | | | 0 50 100 | |

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: J. Hook COMPLETED: 05/03/21

BORING METHOD: direct push (see document text) BORING BIT DIAMETER: 2 1/2 inches

| | | |
|---|---|---|
| NV5 | STHELENS-3-01 | **BORING B-4** |
| | JANUARY 2022 | 1ST AND STRAND STREETS ST. HELENS, OR — **FIGURE A-4** |

| DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100) | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|---|
| 0.0 | | ASPHALT CONCRETE (8.0 inches). | | | | | |
| | | Very dense, light gray GRAVEL with silt and sand (GP-GM); moist (weathered basalt). | 0.7 | | | | |
| 2.5 | | | | | | 50/1" | |
| | | Exploration terminated at a depth of 2.8 feet due to refusal. Hammer efficiency factor is 82.2 percent. | 2.8 | | | | Surface elevation was not measured at the time of exploration. |
| 5.0 | | | | | | | |
| 7.5 | | | | | | | |
| 10.0 | | | | | | | |
| 12.5 | | | | | | | |
| 15.0 | | | | | | | |
| 17.5 | | | | | | | |
| 20.0 | | | | | | | |

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: J. Hook COMPLETED: 05/04/21

BORING METHOD: direct push (see document text) BORING BIT DIAMETER: 2 1/2 inches

NV5

STHELENS-3-01

JANUARY 2022

**BORING B-5**

1ST AND STRAND STREETS
ST. HELENS, OR

**FIGURE A-5**

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | BLOW COUNT / MOISTURE CONTENT % | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 0.0 | Very dense, light gray GRAVEL with silt and sand (GP-GM); moist (aggregate base) - **FILL**. | | | | |
| 2.5 | Medium dense, light gray-brown SAND (SP), trace silt; moist, sand is fine to medium - **FILL**. | 2.5 | | 14 | |
| 5.0 | | | | 11 | |
| 7.5 | very loose to loose, orange-brown; moist to wet at 8.0 feet | | P200 | 4 | P200 = 4% |
| | Very soft, dark brown SILT (ML), some clay and organics (woody debris), minor sand; moist - **FILL**. | 9.5 | | | |
| 10.0 | Very soft, gray CLAY (CL), minor silt, trace to minor sand, trace organics (woody debris); wet. | 10.5 | | 1 | 10.5 feet, during drilling |
| 12.5 | | | | | |
| | Very dense, dark gray GRAVEL with clay (GP-GC); moist to wet (weathered basalt). | 14.0 | | 50/2" | Surface elevation was not measured at the time of exploration. |
| | Exploration terminated at a depth of 14.2 feet due to refusal. | 14.2 | | | |
| 15.0 | Hammer efficiency factor is 82.2 percent. | | | | |
| 17.5 | | | | | |
| 20.0 | | | | | |

Legend: ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: J. Hook COMPLETED: 05/04/21

BORING METHOD: direct push (see document text) BORING BIT DIAMETER: 2 1/2 inches

NV5 | STHELENS-3-01 | **BORING B-6**

JANUARY 2022 | 1ST AND STRAND STREETS ST. HELENS, OR | **FIGURE A-6**

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION / DEPTH | TESTING | BLOW COUNT / MOISTURE CONTENT % (0–50–100) | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 0.0 | ASPHALT CONCRETE (2.5 inches). | 0.2 | | | |
| | Dense, gray-brown GRAVEL with silt and sand (GP-GM); moist (aggregate base) - **FILL**. | | | | |
| 2.5 | | | | 50/5" | |
| | CONCRETE SLAB. | 2.9 | | | |
| | Medium dense, light gray-brown GRAVEL with silt and sand (GP-GM); moist - **FILL**. | 3.5 | | | |
| 5.0 | | | | 24 | |
| | Loose, light gray SAND with silt (SP-SM), trace gravel; moist, sand is fine - **FILL**. | 7.0 | | | |
| 7.5 | | | | 7 | |
| | Very soft, gray-brown SILT (ML), some clay, trace to minor sand; wet, interbeds of silty SAND. | 9.5 | | | ▽ 9.0 feet, during drilling |
| 10.0 | | | ATT | 1 | Recovery at 10.0 feet is from Geoprobe sleeve, not SPT.<br>LL = 52%<br>PL = 29% |
| 12.5 | | | | | |
| 15.0 | Very dense, dark gray GRAVEL with clay (GP-GC); moist to wet (weathered basalt). | 15.0 | | 6-50/6" | |
| | Exploration terminated at a depth of 16.0 feet due to refusal.<br><br>Hammer efficiency factor is 82.2 percent. | 16.0 | | | Surface elevation was not measured at the time of exploration. |
| 17.5 | | | | | |
| 20.0 | | | | | |

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC%

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: J. Hook COMPLETED: 05/04/21

BORING METHOD: direct push (see document text) BORING BIT DIAMETER: 2 1/2 inches

NV5

STHELENS-3-01

JANUARY 2022

**BORING B-7**

1ST AND STRAND STREETS
ST. HELENS, OR

**FIGURE A-7**

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | BLOW COUNT / MOISTURE CONTENT % | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 0.0 | ASPHALT CONCRETE (8.0 inches). | | | | |
| | Dense, light gray GRAVEL with silt and sand (GP-GM); moist (aggregate base) - **FILL**. | 0.7 | | | |
| | Medium dense, light gray SAND with silt (SP-SM); moist, sand is fine to medium - **FILL**. | 2.0 | | | |
| 2.5 | | | | 11 | |
| 5.0 | travel gravel at 5.0 feet | | | 18 | |
| 7.5 | with gravel at 7.5 feet | | | 17 | |
| | without gravel at 8.5 feet | | | | |
| | wet at 9.0 feet | | | | 9.0 feet, during drilling |
| 10.0 | | | PP | 3 | PP = 0.5 tsf |
| | Soft, light gray CLAY (CH), minor organics, trace silt and sand; wet, clay has high plasticity. | 11.0 | | | |
| 12.5 | | | | | |
| | Very dense, gray, clayey GRAVEL (GC), minor sand; wet (weathered basalt). | 14.5 | | | |
| 15.0 | | | | 50/5" | |
| | Exploration terminated at a depth of 15.4 feet due to refusal.<br><br>Hammer efficiency factor is 82.2 percent. | 15.4 | | | Surface elevation was not measured at the time of exploration. |
| 17.5 | | | | | |
| 20.0 | | | | | |

Legend: ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0 50 100)

DRILLED BY: Western States Soil Conservation, Inc.  LOGGED BY: J. Hook  COMPLETED: 05/04/21

BORING METHOD: direct push (see document text)  BORING BIT DIAMETER: 2 1/2 inches

NV5

STHELENS-3-01

JANUARY 2022

**BORING B-8**

1ST AND STRAND STREETS
ST. HELENS, OR

**FIGURE A-8**

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT ● MOISTURE CONTENT % (0–50–100) | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|
| 0.0 | ASPHALT CONCRETE (8.0 inches). | | | | | |
| | Medium dense, gray GRAVEL with silt and sand (GP-GM); moist, gravel is angular (aggregate base) - **FILL**. | 0.7 | | | | |
| 2.5 | Medium dense, light gray SAND with silt (SP-SM); moist, sand is fine to medium - **FILL**. | 2.5 | | ■ | 26 | |
| 5.0 | loose to medium dense at 5.0 feet | | | ■ | 10 | |
| 7.5 | Loose, light gray SAND (SP), trace silt and organics (woody debris); moist - **FILL**. | 7.5 | | ■ | 5 | |
| 10.0 | without organics at 10.0 feet | | P200 | ■ | 8 | P200 = 5% |
| 12.5 | | | | | | ◁ 13.0 feet, during drilling |
| 15.0 | trace organics (woody debris); wet at 15.0 feet | | | ■ | 6 | |
| | Loose, black, silty GRAVEL with organics (woody debris) (GM); wet, gravel is fine. | 16.0 | | | | |
| | Exploration completed at a depth of 16.5 feet.<br><br>Hammer efficiency factor is 82.2 percent. | 16.5 | | | | Surface elevation was not measured at the time of exploration. |
| 17.5 | | | | | | |
| 20.0 | | | | | | |

RQD% / CORE REC%

DRILLED BY: Western States Soil Conservation, Inc.   LOGGED BY: J. Hook   COMPLETED: 05/04/21

BORING METHOD: direct push (see document text)   BORING BIT DIAMETER: 2 1/2 inches

| NV5 | STHELENS-3-01 | **BORING B-9** | |
|---|---|---|---|
| | JANUARY 2022 | 1ST AND STRAND STREETS<br>ST. HELENS, OR | **FIGURE A-9** |

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION / DEPTH | TESTING | BLOW COUNT / MOISTURE CONTENT % | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 0.0 | ASPHALT CONCRETE (6.0 inches). | | | | |
| | Dense, light gray GRAVEL with silt and sand (GP-GM); moist (aggregate base) - **FILL**. | 0.5 | | | |
| | Medium dense, light gray SAND with silt (SP-SM); moist, sand is fine - **FILL**. | 2.0 | | 21 | |
| 5.0 | loose to medium dense, light gray-brown at 5.0 feet | | | 10 | |
| 7.5 | Medium dense, light gray SAND (SP), trace silt; moist - **FILL**. | 7.5 | P200 | 14 | P200 = 5% |
| 10.0 | | | | 13 | |
| 12.5 | | | | | |
| 15.0 | Very loose, gray-brown, silty SAND (SM); wet. | 15.0 | | 2 | ▽ 15.5 feet, during drilling |
| | with organics (woody debris) at 16.0 feet | | | | |
| | Exploration completed at a depth of 16.5 feet. | 16.5 | | | Surface elevation was not measured at the time of exploration. |
| 17.5 | Hammer efficiency factor is 82.2 percent. | | | | |
| 20.0 | | | | | |

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC%

0 50 100

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: J. Hook COMPLETED: 05/04/21

BORING METHOD: direct push (see document text) BORING BIT DIAMETER: 2 1/2 inches

NV5

STHELENS-3-01

JANUARY 2022

**BORING B-10**

1ST AND STRAND STREETS
ST. HELENS, OR

**FIGURE A-10**

| DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|---|
| 0.0 | | ASPHALT CONCRETE (6.0 inches). | | | | | |
| | | Loose, light gray-brown, silty GRAVEL with sand (GM); moist, gravel is angular - **FILL**. | 0.5 | | | | |
| 2.5 | | | | | | 7 | Recovery at 2.5 feet is from Geoprobe sleeve, not SPT. |
| | | Very loose, gray-brown GRAVEL with silt and sand (GP-GM); moist, gravel is angular - **FILL**. | 4.5 | | | | |
| 5.0 | | | | | | 3 | |
| | | Very loose to loose, light brown, silty SAND (SM), minor gravel; moist, sand is fine - **FILL**. | 7.0 | | | | |
| 7.5 | | | | | | 4 | Recovery at 7.5 feet is from Geoprobe sleeve, not SPT. |
| 10.0 | | | | | | 2 | |
| 12.5 | | | | | | | |
| 15.0 | | | | | | | P200 = 66% |
| | | Soft to medium stiff, light brown SILT (ML), minor sand; moist. | 15.5 | P200 | | 4 | |
| | | Exploration completed at a depth of 16.5 feet.<br>Hammer efficiency factor is 82.2 percent. | 16.5 | | | | Surface elevation was not measured at the time of exploration. |
| 17.5 | | | | | | | |
| 20.0 | | | | | | | |

0 50 100

DRILLED BY: Western States Soil Conservation, Inc.   LOGGED BY: J. Hook   COMPLETED: 05/04/21

BORING METHOD: direct push (see document text)   BORING BIT DIAMETER: 2 1/2 inches

| NV5 | STHELENS-3-01 | **BORING B-11** | |
|---|---|---|---|
| | JANUARY 2022 | 1ST AND STRAND STREETS<br>ST. HELENS, OR | **FIGURE A-11** |

| DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100) | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|---|
| 0.0 | | ASPHALT CONCRETE (8.0 inches). | | | | | |
| | | Medium dense, gray-brown, silty GRAVEL with sand and cobbles (GM); moist - **FILL**. | 0.7 | | | | |
| 2.5 | | | | | | 14 | |
| 5.0 | | | | | | 12 | |
| | | Exploration terminated at a depth of 6.5 feet due to practical refusal in gravel and cobbles. | 6.5 | | | | Surface elevation was not measured at the time of exploration. |
| 7.5 | | Hammer efficiency factor is 82.2 percent. | | | | | |
| 10.0 | | | | | | | |
| 12.5 | | | | | | | |
| 15.0 | | | | | | | |
| 17.5 | | | | | | | |
| 20.0 | | | | | | | |

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: J. Hook COMPLETED: 05/04/21

BORING METHOD: direct push (see document text) BORING BIT DIAMETER: 2 1/2 inches

| | | |
|---|---|---|
| NV5 | STHELENS-3-01 | **BORING B-12** |
| | JANUARY 2022 | 1ST AND STRAND STREETS<br>ST. HELENS, OR |

**FIGURE A-12**

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | BLOW COUNT | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 0.0 | ASPHALT CONCRETE (10.0 inches). | | | | |
| | AGGREGATE BASE (12.0 inches). | 0.8 | | | |
| | Dense, gray SAND with silt (SP-SM), trace organics; moist, sand is fine to medium - **FILL**. | 1.8 | | | |
| 2.5 | | | | 33 | |
| 5.0 | trace gravel at 5.0 feet | | | 34 | |
| 7.5 | medium dense, without organics at 7.5 feet | | | 23 | |
| 10.0 | | | P200 | 21 | P200 = 7% |
| 12.5 | | | | | |
| | Soft, gray with black mottled CLAY (CL), trace sand and silt; moist to wet, clay has low to medium plasticity. | 13.0 | | | |
| 15.0 | | | | 3 | |
| | Medium hard to hard (R3-R4), gray BASALT; slightly weathered, moderately to intensely fractured [jointed, 45-90°, very narrow, iron oxide staining, planar, smooth to rough, not healed], aphanitic, vesicular (5%) (Columbia River Basalt). | 17.0 | | | |
| 17.5 | | | | | Driller Comment: rock, hard drilling at 17.0 feet. |
| | | | | | Switch to HQ rock drilling at 19.5 feet. |
| 20.0 | | | | | |

Legend: ▲ BLOW COUNT; ● MOISTURE CONTENT %; RQD%; CORE REC% (scale 0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: H. Herinckx
COMPLETED: 07/26/21

BORING METHOD: mud rotary and HQ core drilling (see document text)
BORING BIT DIAMETER: 4 7/8 inches

NV5

STHELENS-3-01

JANUARY 2022

**BORING B-13**

1ST AND STRAND STREETS
ST. HELENS, OR

**FIGURE A-13**

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | INSTALLATION AND COMMENTS |
|---|---|---|---|---|
| 20.0 | (continued from previous page) | | | |
| 22.0 | hard (R4); [clay infill and quartz mineralization, planar and stepped, not healed to partly healed] vesicular (2%) at 22.0 feet | | | |
| 27.0 | hard to very hard (R4-R5); [silt infill, planar to curved, not healed] at 27.0 feet | | UC | UC = 12,058 psi |
| 32.0 | Exploration completed at a depth of 32.0 feet.<br><br>Hammer efficiency factor is 85.6 percent. | 32.0 | | Surface elevation was not measured at the time of exploration. |

Legend: ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: H. Herinckx
COMPLETED: 07/26/21

BORING METHOD: mud rotary and HQ core drilling (see document text)
BORING BIT DIAMETER: 4 7/8 inches

NV5

STHELENS-3-01

JANUARY 2022

**BORING B-13**
(continued)

1ST AND STRAND STREETS
ST. HELENS, OR

**FIGURE A-13**

| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | BLOW COUNT / MOISTURE CONTENT % | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|
| 0.0 | ASPHALT CONCRETE (1.0 inch). | 0.1 | | | |
| | OIL MAT (3.5 inches). | 0.4 | | | |
| | AGGREGATE BASE (12.5 inches). | | | | |
| | Very soft, gray-brown CLAY (CL), minor silt and sand, trace to minor gravel; moist to wet - **FILL**. | 1.4 | | | |
| 2.5 | | | ATT | 1 | LL = 38%<br>PL = 18% |
| 5.0 | | | | 27 | Switch to 3 7/8-inch mud rotary drilling at 5.0 feet. |
| | Medium dense, gray-brown, clayey GRAVEL (GC), minor sand; moist. | 5.5 | | | |
| | Very dense, dark gray GRAVEL with clay (GP-GC), minor sand; moist (weathered basalt). | 6.5 | | | |
| 7.5 | | | | 50/5" | |
| | Exploration terminated at a depth of 9.0 feet due to broken tri-cone bit in hole.<br><br>Hammer efficiency factor is 82.2 percent. | 9.0 | | | CORE DETAILS:<br>No patch observed.<br>Cored on moderate fatigue crack.<br><br>Surface elevation was not measured at the time of exploration. |
| 10.0 | | | | | |
| 12.5 | | | | | |
| 15.0 | | | | | |
| 17.5 | | | | | |
| 20.0 | | | | | |

Legend: ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: J. Hook COMPLETED: 05/03/21

BORING METHOD: core drill/direct-push and mud rotary (see document text) BORING BIT DIAMETER: 5 1/2 inches/2 1/2 inches and 3 7/8 inches

STHELENS-3-01

JANUARY 2022

**BORING C-1**

1ST AND STRAND STREETS
ST. HELENS, OR

**FIGURE A-14**

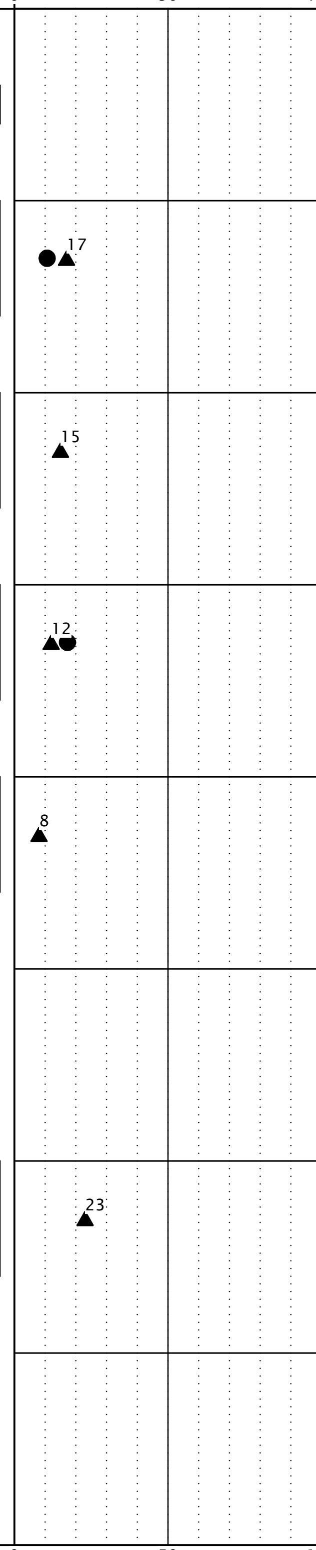
| DEPTH FEET | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100) | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|
| 0.0 | ASPHALT CONCRETE (4.5 inches). | | | | | |
| | AGGREGATE BASE (25.5 inches). | 0.4 | | | | |
| 2.5 | Stiff, dark gray CLAY with gravel (CL), trace to minor sand; moist, gravel is angular - **FILL**. | 2.5 | | | 9 | |
| | Soft to medium hard (R2-R3), gray BASALT; slightly to moderately weathered, intensely fractured (jointed, 0-20° and 60-80°, very narrow, clay infill, stepped to irregular, rough, partly healed), aphanitic, vesicular along flow bands (Columbia River Basalt). | 4.5 | | | | Switch to HQ rock drilling at 4.5 feet. |
| 5.0 | Dense, dark brown, clayey GRAVEL (GC); moist, gravel is angular (weathered basalt). | 5.0 | | | | |
| | Medium hard (R3), light gray BASALT; moderately fractured [planar to curved, smooth to rough, moderately to totally healed]. | 5.5 | | | | |
| 7.5 | | | | | | |
| 10.0 | | | | | | |
| 12.5 | Exploration completed at a depth of 12.5 feet. Hammer efficiency factors are 82.2 percent. | 12.5 | | | | CORE DETAILS: No patch observed. Cored on moderate fatigue crack. Surface elevation was not measured at the time of exploration. |
| 15.0 | | | | | | |
| 17.5 | | | | | | |
| 20.0 | | | | | | |

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: J. Hook COMPLETED: 05/12/21

BORING METHOD: core drill/mud rotary and HQ core drilling (see document text) BORING BIT DIAMETER: 6 inches/3 7/8 inches

| | | | |
|---|---|---|---|
| NV5 | STHELENS-3-01 | **BORING C-2** | |
| | JANUARY 2022 | 1ST AND STRAND STREETS ST. HELENS, OR | **FIGURE A-15** |

| DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100) | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|---|
| 0.0 | | ASPHALT CONCRETE (4.0 inches). | | | | | |
| | | Medium dense, gray-brown GRAVEL with silt and sand (GP-GM); moist, gravel is angular - **FILL**. | 0.3 | | | | |
| 2.5 | | | | | | 17 | |
| 5.0 | | | | | | 15 | |
| 7.5 | | | | | | 12 | |
| 10.0 | | loose at 10.0 feet | | | | 8 | |
| 12.5 | | | | | | | |
| 15.0 | | medium dense at 15.0 feet | | | | 23 | |
| | | Exploration completed at a depth of 16.5 feet. | 16.5 | | | | CORE DETAILS: Cored through patch. No crack at core. |
| 17.5 | | Hammer efficiency factor is 69.2 percent. | | | | | Surface elevation was not measured at the time of exploration. |
| 20.0 | | | | | | | |




DRILLED BY: Western States Soil Conservation, Inc.  LOGGED BY: J. Hook  COMPLETED: 05/05/21

BORING METHOD: core drill/mud rotary (see document text)  BORING BIT DIAMETER: 6 inches/3 7/8 inches

| | | |
|---|---|---|
| STHELENS-3-01 | **BORING C-3** | |
| JANUARY 2022 | 1ST AND STRAND STREETS ST. HELENS, OR | **FIGURE A-16** |

| DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100) | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|---|
| 0.0 | | ASPHALT CONCRETE (3.5 inches). | 0.3 | | | | |
| | | AGGREGATE BASE (14.5 inches). | | | | | |
| | | Medium hard (R3), gray BASALT; slightly weathered, intensely to very intensely fractured [jointed, 0-10° and 80-90°, very narrow, iron oxide staining, stepped, rough, not healed], aphanitic (Columbia River Basalt). | 1.5 | | | | Switch to HQ rock drilling at 1.5 feet. |
| 2.5 | | | | | | | |
| 5.0 | | | | | | | |
| 7.5 | | Exploration completed at a depth of 7.5 feet.<br><br>Hammer efficiency factor is 69.2 percent. | 7.5 | | | | CORE DETAILS:<br>No patch observed.<br>No crack at core.<br><br>Surface elevation was not measured at the time of exploration. |
| 10.0 | | | | | | | |
| 12.5 | | | | | | | |
| 15.0 | | | | | | | |
| 17.5 | | | | | | | |
| 20.0 | | | | | | 0 50 100 | |

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: J. Hook COMPLETED: 05/05/21

BORING METHOD: core drill/mud rotary and HQ core drilling (see document text) BORING BIT DIAMETER: 6 inches/3 7/8 inches

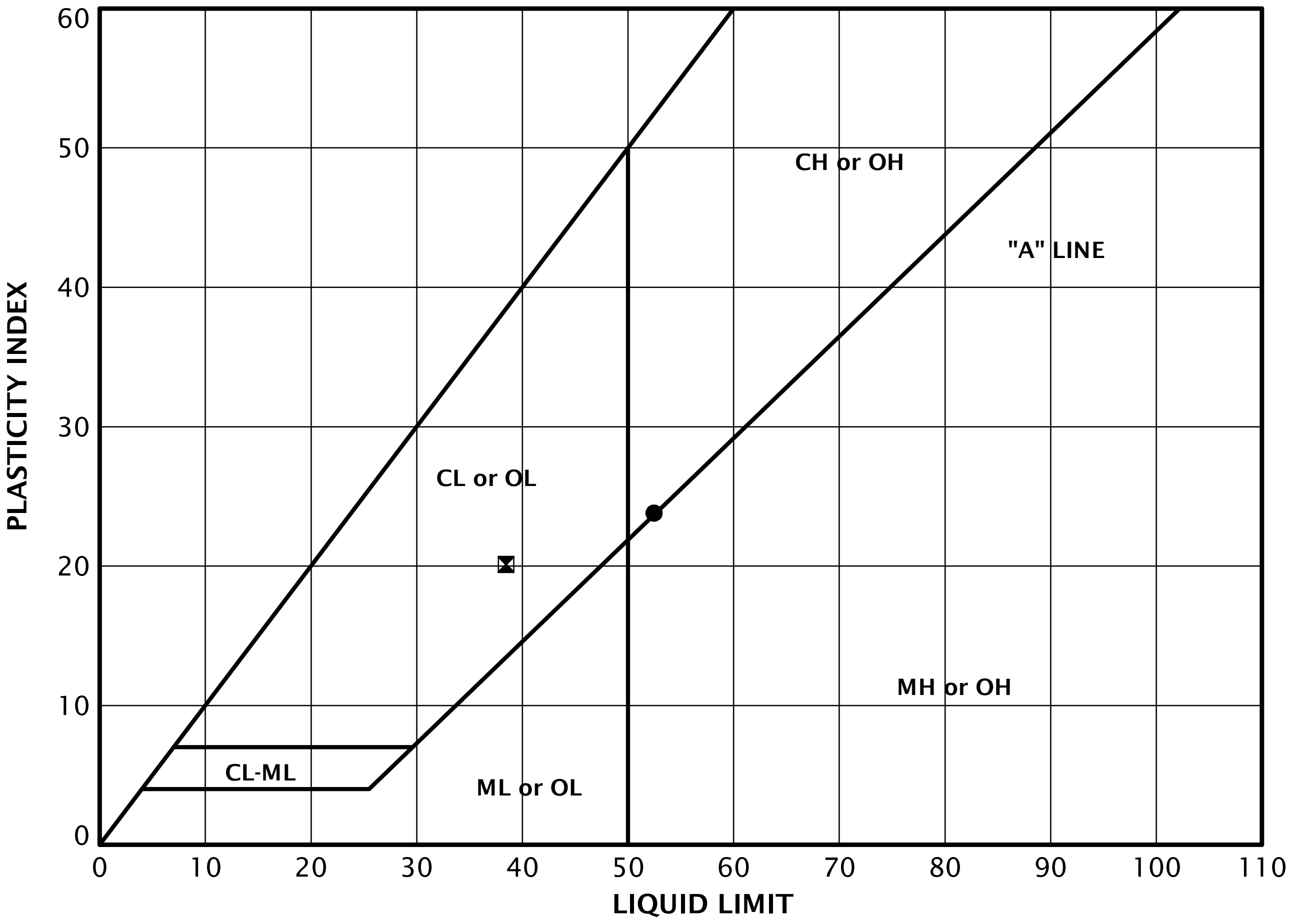
| NV5 | STHELENS-3-01 | BORING C-4 | |
|---|---|---|---|
| | JANUARY 2022 | 1ST AND STRAND STREETS<br>ST. HELENS, OR | FIGURE A-17 |

| DEPTH FEET | GRAPHIC LOG | MATERIAL DESCRIPTION | ELEVATION DEPTH | TESTING | SAMPLE | ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100) | INSTALLATION AND COMMENTS |
|---|---|---|---|---|---|---|---|
| 0.0 | | ASPHALT CONCRETE (3.25 inches). | 0.3 | | | | |
| | | OIL MAT (3.25 inches). | 0.5 | | | | |
| | | AGGREGATE BASE (23.5 inches). | | | | | |
| 2.5 | | Soft to medium stiff, brown-black CLAY (CL), some organics (woody debris), minor sand; moist. | 2.5 | | | 4 | |
| 5.0 | | Medium hard (R3), gray BASALT; slightly weathered, intensely fractured (jointed, 80-100° and 0-20°, very narrow, oxide and clay infill, undulating, smooth to rough, partly healed), aphanitic, vesicular along flow bands (Columbia River Basalt). | 4.0 | | | | Switch to HQ rock drilling at 5.0 feet. |
| 7.5 | | slightly fractured [totally healed] at 8.5 feet | | UC | | | UC = 5,518 psi |
| 10.0 | | Exploration completed at a depth of 10.0 feet.<br><br>Hammer efficiency factor is 78.4 percent. | 10.0 | | | | CORE DETAILS:<br>No patch observed.<br>No crack at core.<br><br>Surface elevation was not measured at the time of exploration. |
| 12.5 | | | | | | | |
| 15.0 | | | | | | | |
| 17.5 | | | | | | | |
| 20.0 | | | | | | 0 50 100 | |

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: J. Hook COMPLETED: 05/12/21

BORING METHOD: core drill/mud rotary and HQ core drilling (see document text) BORING BIT DIAMETER: 6 inches/3 7/8 inches

| NV5 | STHELENS-3-01 | BORING C-5 | |
|---|---|---|---|
| | JANUARY 2022 | 1ST AND STRAND STREETS<br>ST. HELENS, OR | FIGURE A-18 |




| KEY | EXPLORATION NUMBER | SAMPLE DEPTH (FEET) | MOISTURE CONTENT (PERCENT) | LIQUID LIMIT | PLASTIC LIMIT | PLASTICITY INDEX |
|---|---|---|---|---|---|---|
| ● | B-7 | 10.0 | 44 | 52 | 29 | 23 |
| ⧗ | C-1 | 2.5 | 29 | 38 | 18 | 20 |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |



NV5

STHELENS-3-01

JANUARY 2022

## ATTERBERG LIMITS TEST RESULTS

1ST AND STRAND STREETS
ST. HELENS, OR

**FIGURE A-19**

| 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 | | 18 | | | | | | | |
| B-1 | 5.0 | | 20 | | | | | | | |
| B-2 | 2.5 | | 15 | | | | 6 | | | |
| B-3 | 2.5 | | 5 | | | | | | | |
| B-3 | 2.6 | | 7 | | | | | | | |
| B-3 | 5.0 | | 323 | | | | | | | |
| B-3 | 7.5 | | 22 | | | | | | | |
| B-4 | 2.5 | | 8 | | | | | | | |
| B-4 | 5.0 | | 145 | | | | | | | |
| B-6 | 2.5 | | 6 | | | | | | | |
| B-6 | 5.0 | | 6 | | | | | | | |
| B-6 | 7.5 | | 9 | | | | 4 | | | |
| B-6 | 10.0 | | 60 | | | | | | | |
| B-7 | 2.5 | | 10 | | | | | | | |
| B-7 | 5.0 | | 10 | | | | | | | |
| B-7 | 7.5 | | 7 | | | | | | | |
| B-7 | 10.0 | | 44 | | | | | 52 | 29 | 23 |
| B-7 | 15.0 | | 19 | | | | | | | |
| B-8 | 5.0 | | 8 | | | | | | | |
| B-8 | 10.0 | | 19 | | | | | | | |
| B-9 | 2.5 | | 7 | | | | | | | |
| B-9 | 7.5 | | 7 | | | | | | | |
| B-9 | 10.0 | | 13 | | | | 5 | | | |
| B-10 | 5.0 | | 7 | | | | | | | |
| B-10 | 7.5 | | 7 | | | | 5 | | | |
| B-10 | 10.0 | | 8 | | | | | | | |
| B-10 | 15.0 | | 29 | | | | | | | |

LAB SUMMARY - GDI-NV5 STHELENS-3-01-B1_13-C1_5.GPJ GDI_NV5.GDT PRINT DATE: 1/5/22:KT

NV5

STHELENS-3-01

JANUARY 2022

**SUMMARY OF LABORATORY DATA**

1ST AND STRAND STREETS
ST. HELENS, OR

**FIGURE A-20**

| 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-11 | 5.0 | | 5 | | | | | | | |
| B-11 | 15.0 | | 41 | | | | 66 | | | |
| B-13 | 5.0 | | 13 | | | | | | | |
| B-13 | 10.0 | | 15 | | | | 7 | | | |
| B-13 | 15.0 | | 51 | | | | | | | |
| C-1 | 2.5 | | 29 | | | | | 38 | 18 | 20 |
| C-2 | 1.0 | | 5 | | | | | | | |
| C-2 | 2.5 | | 39 | | | | | | | |
| C-3 | 2.5 | | 11 | | | | | | | |
| C-3 | 7.5 | | 17 | | | | | | | |
| C-5 | 1.0 | | 4 | | | | | | | |
| C-5 | 2.5 | | 69 | | | | | | | |

CORE LOCATION C-1.


CORE C-1.

CORE LOCATION C-2.


CORE C-2.

StHelens-3-01-FA21_A29-CPH.docx Print Date: 1/5/22

| NV5 | STHELENS-3-01 | CORE LOCATION AND CORE PHOTOGRAPHS | |
|---|---|---|---|
| | JANUARY 2022 | 1ST AND STRAND STREETS<br>ST. HELENS, OR | FIGURE A-22 |

CORE LOCATION C-3.

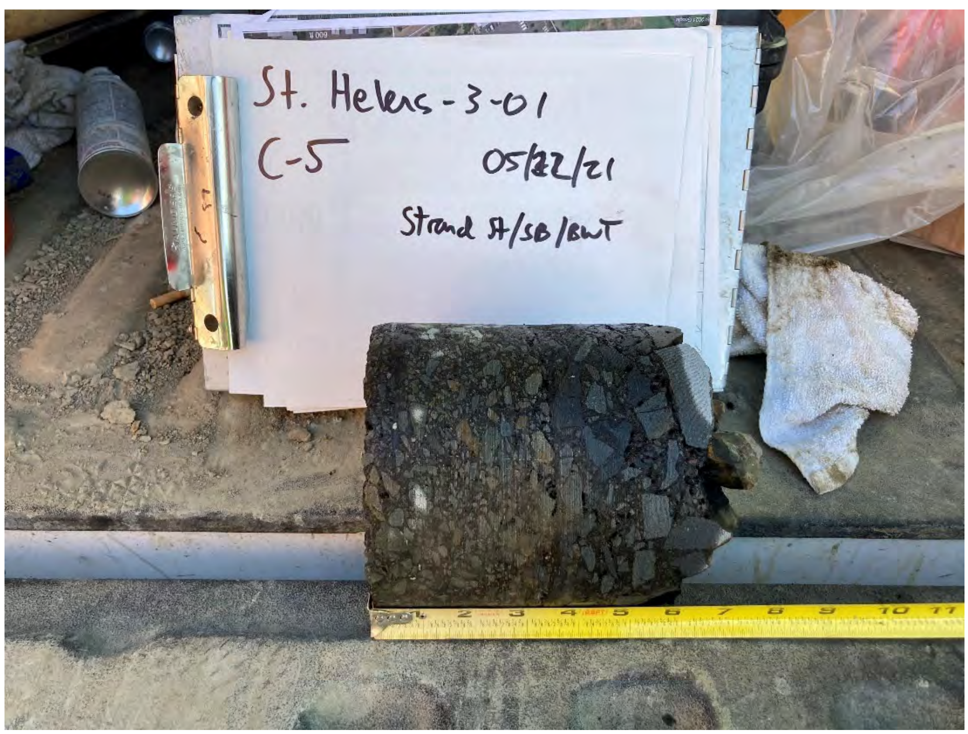

CORE C-3.


CORE LOCATION C-4.


CORE C-4.

| | | | |
|---|---|---|---|
| NV5 | STHELENS-3-01 | **CORE LOCATION AND CORE PHOTOGRAPHS** | |
| | JANUARY 2022 | 1ST AND STRAND STREETS<br>ST. HELENS, OR | **FIGURE A-24** |


CORE LOCATION C-5.




CORE C-5.


BORING B-13. RUN 1. 19.5 TO 22 FEET BGS.


BORING B-13. RUN 2. 22 TO 27 FEET BGS.



| | | | |
|---|---|---|---|
| NV5 | STHELENS-3-01 | ROCK CORE PHOTOGRAPHS | |
| | JANUARY 2022 | 1ST AND STRAND STREETS<br>ST. HELENS, OR | FIGURE A-26 |


BORING B-13. RUN 3. 27 TO 32 FEET BGS.


BORING C-2. RUN 1. 3.5 TO 7.5 FEET BGS.

BORING C-2. RUN 2. 7.5 TO 12.5 FEET BGS.

BORING C-4. RUN 1. 1.5 TO 4.5 FEET BGS.

| NV5 | STHELENS-3-01 | ROCK CORE PHOTOGRAPHS | |
|---|---|---|---|
| | JANUARY 2022 | 1ST AND STRAND STREETS<br>ST. HELENS, OR | FIGURE A-28 |


BORING C-4.  RUN 2.  4.5 TO 7.5 FEET BGS.


BORING C-5.  RUN 1.  5 TO 10 FEET BGS.

| | | | |
|---|---|---|---|
| NV5 | STHELENS-3-01 | **ROCK CORE PHOTOGRAPHS** | |
| | JANUARY 2022 | 1ST AND STRAND STREETS<br>ST. HELENS, OR | **FIGURE A-29** |

**Summary of SPT Test Results**

Project: WSSC-8-05, Test Date: 4/13/2020

EMX: Maximum Energy

ETR: Energy Transfer Ratio - Rated

| Start Depth ft | Final Depth ft | **N Value** | **N60 Value** | Average EMX ft-lb | Average ETR % |
|---|---|---|---|---|---|
| 15.00 | 16.50 | **8** | **10** | 291.65 | 83.3 |
| 17.50 | 19.00 | **15** | **20** | 278.80 | 79.7 |
| 20.00 | 21.50 | **18** | **24** | 290.63 | 83.0 |
| 22.50 | 24.00 | **15** | **20** | 304.84 | 87.1 |
| 25.00 | 26.50 | **11** | **15** | 269.66 | 77.0 |
| | | | **Overall Average Values:** | 287.84 | 82.2 |
| | | | **Standard Deviation:** | 38.44 | 11.0 |
| | | | **Overall Maximum Value:** | 327.58 | 93.6 |
| | | | **Overall Minimum Value:** | 0.10 | 0.0 |

**Summary of SPT Test Results**

Project: WSSC-8-05, Test Date: 4/13/2020

EMX: Maximum Energy

ETR: Energy Transfer Ratio - Rated

| Start Depth ft | Final Depth ft | **N Value** | **N60 Value** | Average EMX ft-lb | Average ETR % |
|---|---|---|---|---|---|
| 40.00 | 41.50 | **23** | **32** | 307.27 | 87.8 |
| 42.50 | 44.00 | **24** | **34** | 294.99 | 84.3 |
| 45.00 | 46.50 | **28** | **39** | 296.53 | 84.7 |
| 47.50 | 49.00 | **19** | **27** | 296.50 | 84.7 |
| 50.00 | 51.50 | **15** | **21** | 305.07 | 87.2 |
| | | | **Overall Average Values:** | 299.63 | 85.6 |
| | | | **Standard Deviation:** | 7.50 | 2.1 |
| | | | **Overall Maximum Value:** | 320.59 | 91.6 |
| | | | **Overall Minimum Value:** | 281.10 | 80.3 |

**Summary of SPT Test Results**

Project: WSSC-8-05, Test Date: 4/13/2020

EMX: Maximum Energy

ETR: Energy Transfer Ratio - Rated

| Start Depth ft | Final Depth ft | **N Value** | **N60 Value** | Average EMX ft-lb | Average ETR % |
|---|---|---|---|---|---|
| 27.50 | 29.00 | **30** | **34** | 243.05 | 69.4 |
| 30.00 | 31.50 | **26** | **29** | 253.78 | 72.5 |
| 32.50 | 34.00 | **29** | **33** | 238.01 | 68.0 |
| 35.00 | 36.50 | **37** | **42** | 241.80 | 69.1 |
| 37.50 | 39.00 | **44** | **50** | 237.44 | 67.8 |
| | | | **Overall Average Values:** | 242.09 | 69.2 |
| | | | **Standard Deviation:** | 9.55 | 2.7 |
| | | | **Overall Maximum Value:** | 269.06 | 76.9 |
| | | | **Overall Minimum Value:** | 220.68 | 63.1 |

**Summary of SPT Test Results**

Project: WSSC-8-05, Test Date: 4/13/2020

EMX: Maximum Energy

ETR: Energy Transfer Ratio - Rated

| Start Depth ft | Final Depth ft | **N Value** | **N60 Value** | Average EMX ft-lb | Average ETR % |
|---|---|---|---|---|---|
| 15.00 | 16.50 | **11** | **14** | 242.93 | 69.4 |
| 22.50 | 24.00 | **11** | **14** | 277.29 | 79.2 |
| 25.00 | 26.50 | **18** | **23** | 292.05 | 83.4 |
| | | | **Overall Average Values:** | 274.48 | 78.4 |
| | | | **Standard Deviation:** | 27.96 | 8.0 |
| | | | **Overall Maximum Value:** | 354.67 | 101.3 |
| | | | **Overall Minimum Value:** | 228.22 | 65.2 |

# APPENDIX B

## APPENDIX B

## UNCONFINED COMPRESSIVE STRENGTH TEST RESULTS

Unconfined compression testing was conducted by ACS Testing of Tigard, Oregon, on two select rock core samples in general accordance with ASTM D7012. Test results are presented in this appendix.

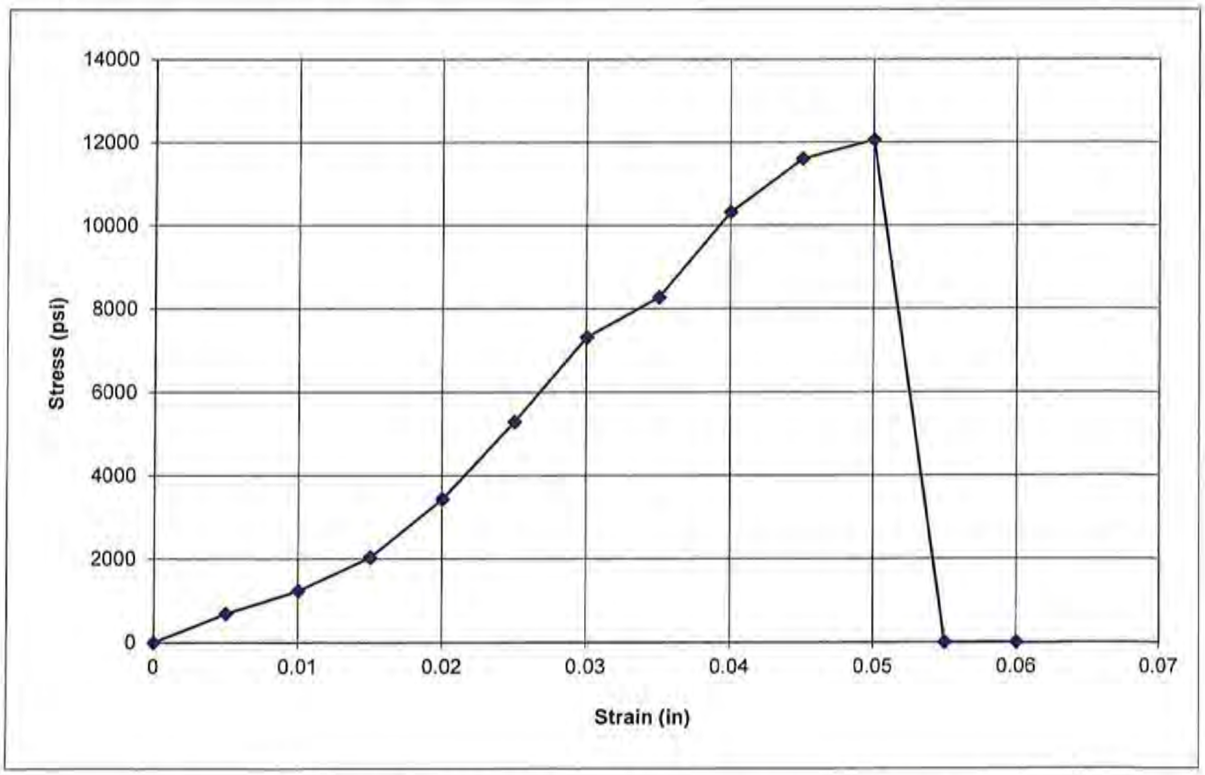
ACS Testing, Inc
7409 SW Tech Center Dr Ste 145
Tigard, OR 97223
PH: 503-443-3799 F: 503-620-2748

**NV5 - PORTLAND**
**9450 SW COMMERCE CIRCLE -STE 300**
**WILSONVILLE, OR 97070**

| | | | |
|---|---|---|---|
| **PROJECT:** | 2021 LAB SERVICES | **JOB NO:** | 21-L001 |
| **LOCATION:** | 1ST AND STRAND | **WORK ORDER NO:** | ST HELENS-3-0 |
| **MATERIAL:** | ROCK CORE | **LAB NO:** | 13424 |
| **SAMPLE SOURCE:** | B-13, 27.5' | **DATE RECEIVED:** | 08/02/21 |
| **SAMPLE PREP:** | SAW-CUT | **DATE SAMPLED:** | 07/26/21 |

**UNCONFINED COMPRESSION STRENGTH OF Rock Cores**
**APPLICABLE PORTIONS OF (ASTM D2938)**

| | | | |
|---|---|---|---|
| DIAMETER: | 2.40 in | MAXIMUM STRESS: | 12,058 psi |
| HEIGHT: | 4.24 in | AT STRAIN: | 1.18% |
| STRAIN RATE: | .005 inches/min. | | |
| DRY DENSITY: | 172.1 lb/cu.ft | | |
| MOISTURE: | 0.9% | | |




NOTE: VISIBLE CRACKING THROUGHOUT CORE UPON ARRIVAL WAS WORSENED DURING SAW-CUTTING
PHOTOS ARE ATTACHED

REVIEWED BY

DE/js

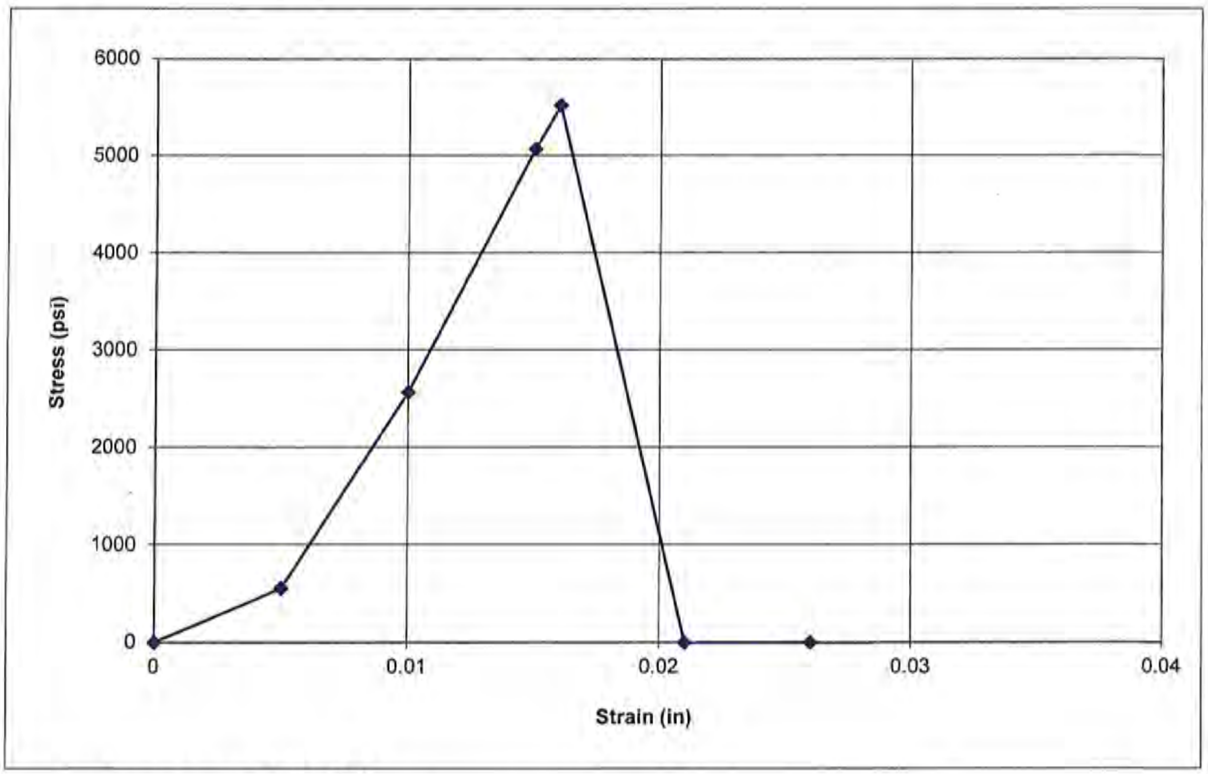
ACS Testing, Inc
7409 SW Tech Center Dr Ste 145
Tigard, OR 97223
PH: 503-443-3799 F: 503-620-2748

**NV5 - PORTLAND**
**9450 SW COMMERCE CIRCLE -STE 300**
**WILSONVILLE, OR 97070**

| | | | |
|---|---|---|---|
| **PROJECT:** | 2021 LAB SERVICES | **JOB NO:** | 21-L001 |
| **LOCATION:** | ST. HELENS - 3 - 01 | **WORK ORDER NO:** | |
| **MATERIAL:** | ROCK CORE | **LAB NO:** | 13268 |
| **SAMPLE SOURCE:** | C-5, 8.5' | **DATE RECEIVED:** | 05/24/21 |
| **SAMPLE PREP:** | SAW-CUT | | |

**UNCONFINED COMPRESSION STRENGTH OF Rock Cores**
**APPLICABLE PORTIONS OF (ASTM D2938)**

| | | | |
|---|---|---|---|
| DIAMETER: | 2.38 in | MAXIMUM STRESS: | 5,518 psi |
| HEIGHT: | 4.96 in | AT STRAIN: | 0.32% |
| STRAIN RATE: | .005 inches/min. | | |
| DRY DENSITY: | 169.7 lb/cu.ft | | |
| MOISTURE: | 0.9% | | |




DISTRIBUTED

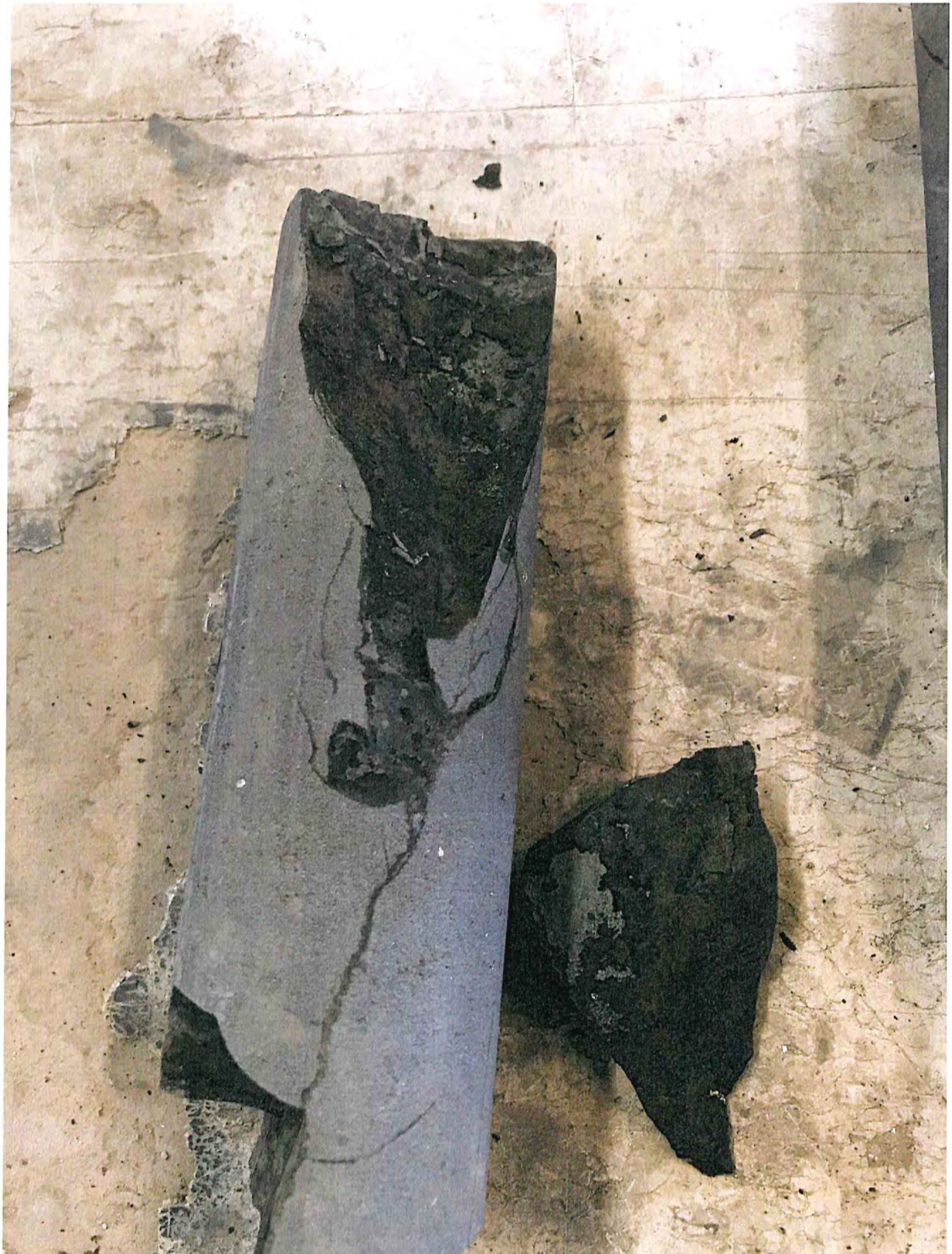
NOTE: VISIBLE CRACKING THROUGHOUT CORE

JUN 4 2021

REVIEWED BY Clais Ca

DE/js


C-2



# APPENDIX C

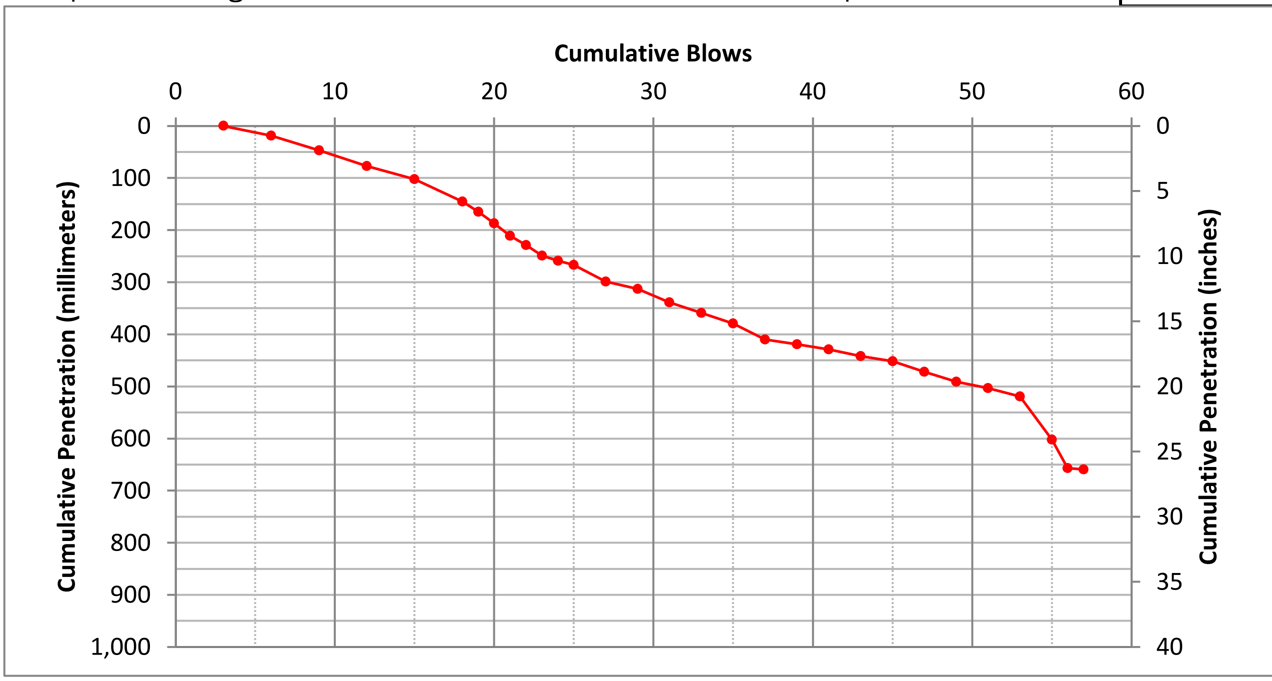
## APPENDIX C

## DCP TESTING

Using the summarized DCP test data, we visually assessed where slopes of the data are relatively constant and at which depths they change significantly. We used the first change in slope with depth to represent the transition from base to subgrade and the slope of the data to this point to estimate the base layer resilient modulus. We used the slope(s) of the data beyond this point to estimate the resilient modulus of subgrade strata. We used least squares regression to determine the slopes and the equation from the ODOT guide to estimate the moduli using a correction factor $C_f = 0.62$ for estimating the base layer resilient moduli and $C_f = 0.35$ for estimating the subgrade resilient moduli. We estimated an equivalent subgrade modulus using Odemark's Method of Equivalent Thickness in cases where we encountered more than one subgrade layer.

## DYNAMIC CONE PENETROMETER RESULTS - BORING C-1

| Layer | Layer Type and Location | Slope (mm/blow) | $C_f$ | $M_R$ (psi) |
|---|---|---|---|---|
| 1 | Aggregate base or subbase below AC | 11.0 | 0.62 | 11,920 |
| 2 | Subgrade below AC and aggregate base | 37.5 | 0.35 | 4,180 |
| 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:

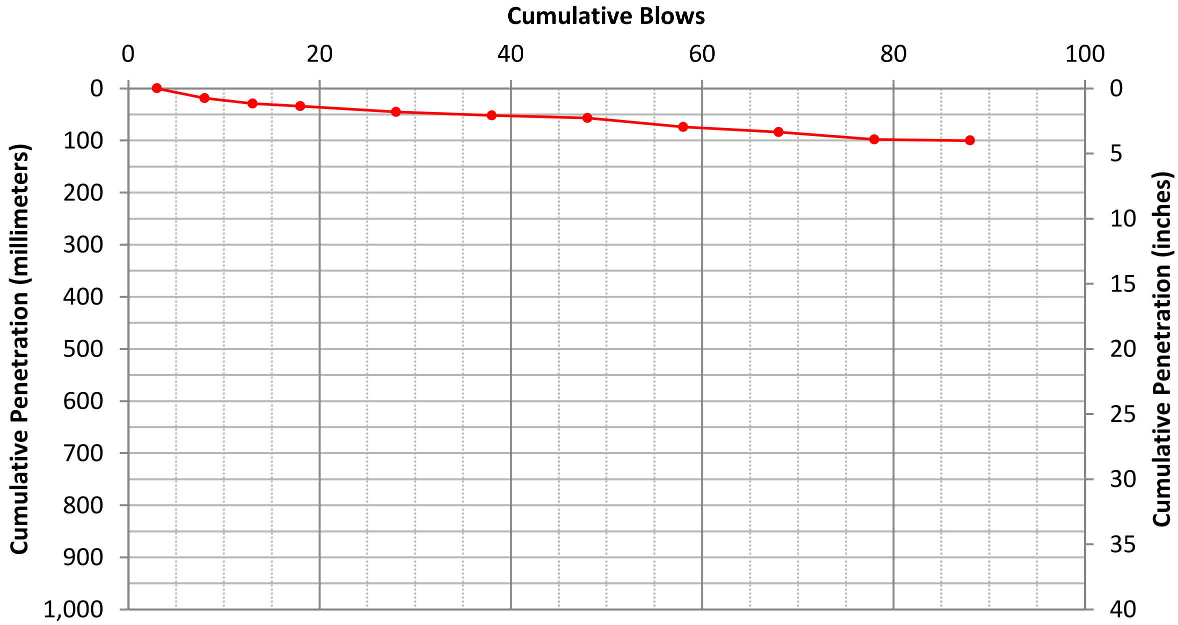
ODOT Pavement Design Guide, Pavement Services Unit, Oregon Department of Transportation, April 2011.

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 - BORING C-5

| Layer | Layer Type and Location | Slope (mm/blow) | $C_f$ | $M_R$ (psi) |
|---|---|---|---|---|
| 1 | No base material | --- | --- | --- |
| 2 | Subgrade below AC and aggregate base | 1.0 | 0.35 | 17,200 |
| 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:

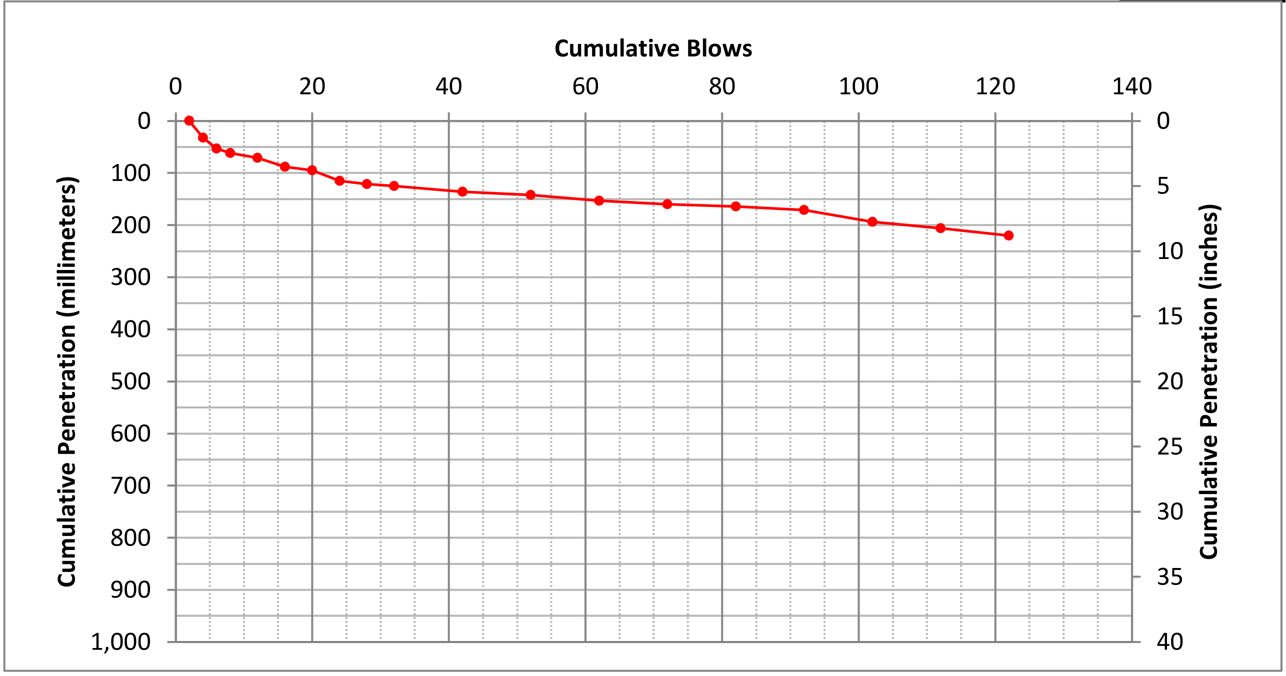

ODOT Pavement Design Guide, Pavement Services Unit, Oregon Department of Transportation, April 2011.

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 - BORING B-1

| Layer | Layer Type and Location | Slope (mm/blow) | $C_f$ | $M_R$ (psi) |
|---|---|---|---|---|
| 1 | Subgrade below AC and aggregate base | 1.0 | 0.35 | 17,110 |
| 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:

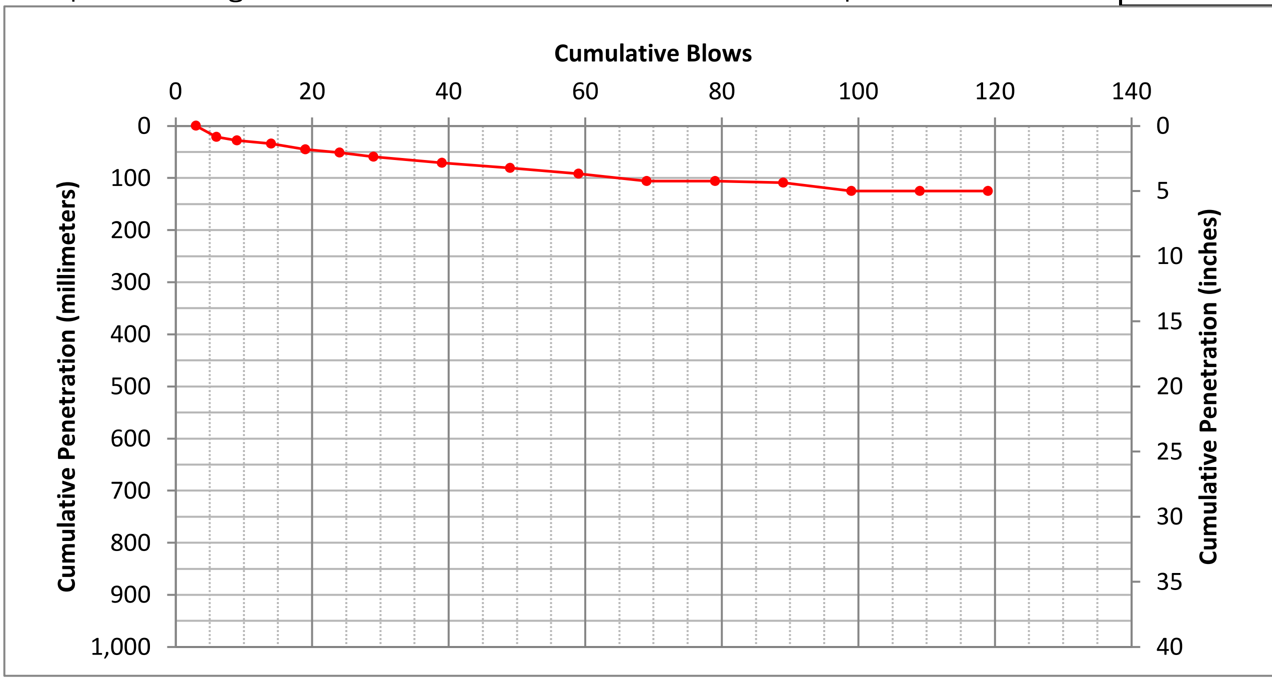

ODOT Pavement Design Guide, Pavement Services Unit, Oregon Department of Transportation, April 2011.

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 - BORING B-3

| Layer | Layer Type and Location | Slope (mm/blow) | $C_f$ | $M_R$ (psi) |
|---|---|---|---|---|
| 1 | Subgrade below AC and aggregate base | 1.0 | 0.35 | 17,070 |
| 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:

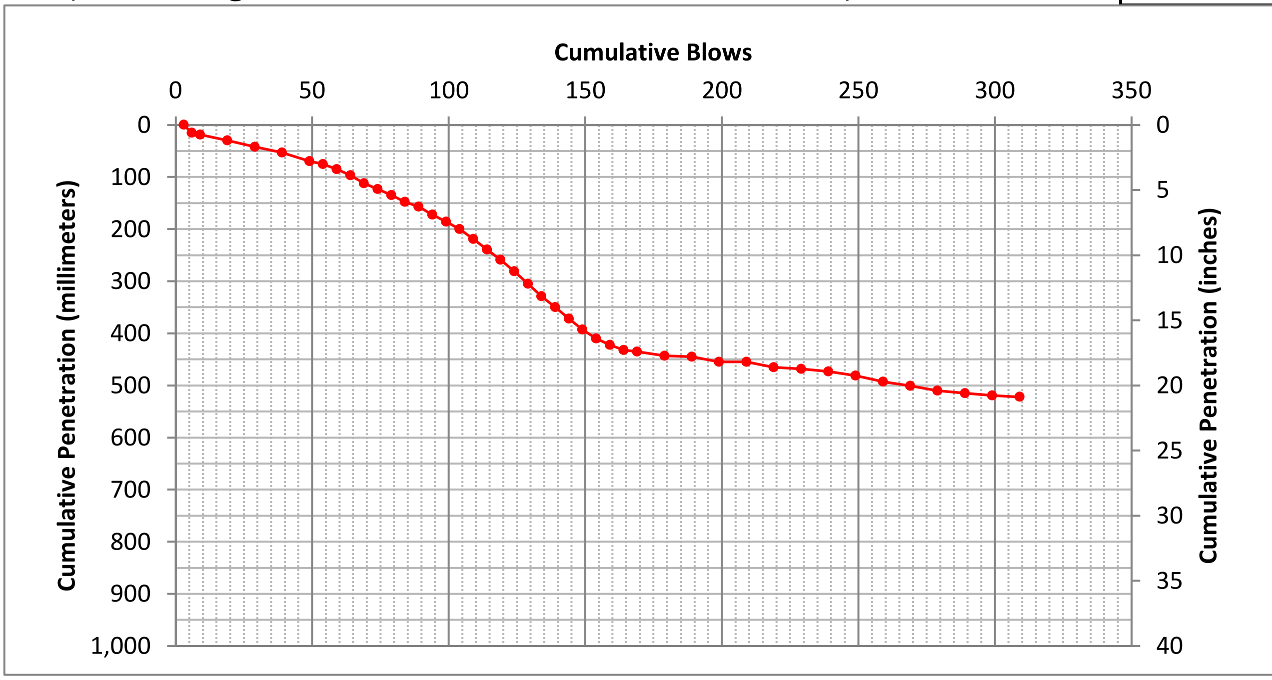

ODOT Pavement Design Guide, Pavement Services Unit, Oregon Department of Transportation, April 2011.

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 - BORING B-5

| Layer | Layer Type and Location | Slope (mm/blow) | $C_f$ | $M_R$ (psi) |
|---|---|---|---|---|
| 1 | Aggregate base or subbase below AC | 2.7 | 0.62 | 20,510 |
| 2 | Subgrade below AC and aggregate base | 0.7 | 0.35 | 20,200 |
| 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:

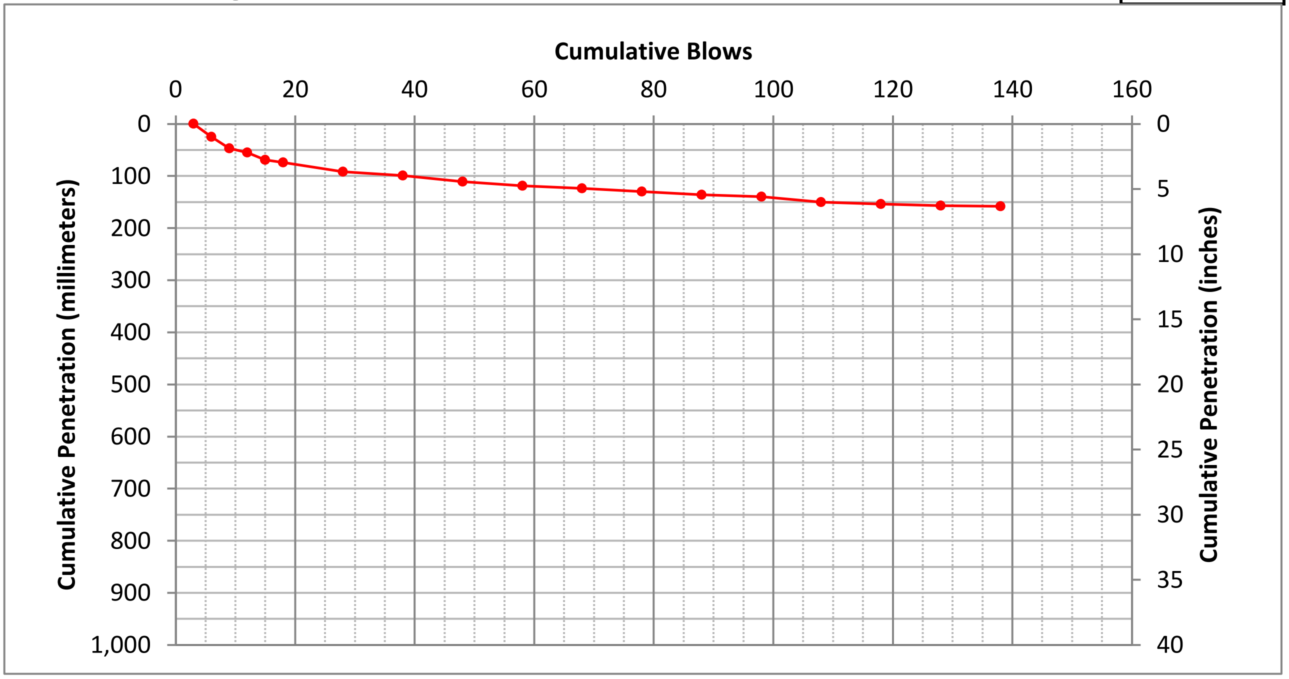

ODOT Pavement Design Guide, Pavement Services Unit, Oregon Department of Transportation, April 2011.

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 - BORING B-6

| Layer | Layer Type and Location | Slope (mm/blow) | $C_f$ | $M_R$ (psi) |
|---|---|---|---|---|
| 1 | Subgrade below AC and aggregate base | 0.8 | 0.35 | 18,600 |
| 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, April 2011.

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 - BORING B-7

| Layer | Layer Type and Location | Slope (mm/blow) | $C_f$ | $M_R$ (psi) |
|---|---|---|---|---|
| 1 | Subgrade below AC and aggregate base | 6.8 | 0.35 | 8,120 |
| 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:

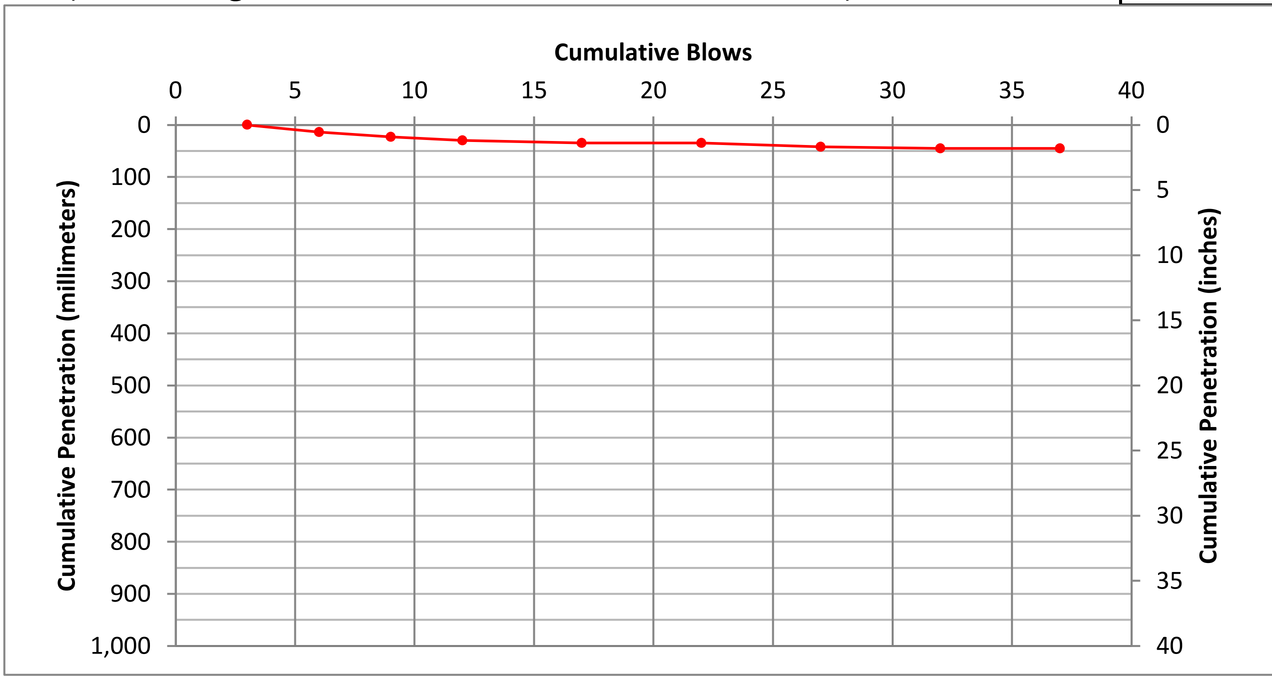

ODOT Pavement Design Guide, Pavement Services Unit, Oregon Department of Transportation, April 2011.

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 - BORING B-9

| Layer | Layer Type and Location | Slope (mm/blow) | $C_f$ | $M_R$ (psi) |
|---|---|---|---|---|
| 1 | Subgrade below AC and aggregate base | 1.2 | 0.35 | 16,190 |
| 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:

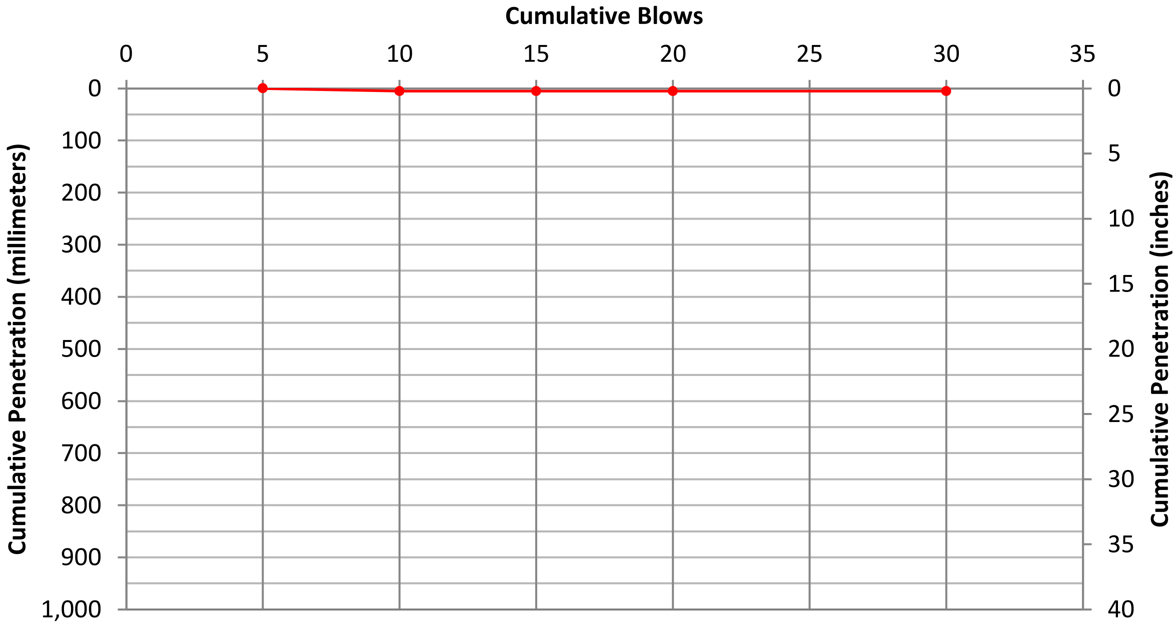

ODOT Pavement Design Guide, Pavement Services Unit, Oregon Department of Transportation, April 2011.

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 - BORING B-10

| Layer | Layer Type and Location | Slope (mm/blow) | $C_f$ | $M_R$ (psi) |
|---|---|---|---|---|
| 1 | Subgrade below AC and aggregate base | 0.1 | 0.35 | 36,080 |
| 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, April 2011.

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.

# APPENDIX D

## APPENDIX D

## TRAFFIC DATA

DKS Associates provided the forecast 2042 ADT value of 3,400 vehicles, with 2 percent heavy vehicles. We back-calculated the current 2021 ADT from the forecast value, assuming a growth rate of zero percent. We also assumed a reasonable breakup of vehicle classification counts for FHWA class 4 through 13 vehicles to achieve 2 percent heavy vehicles. We used the ESAL conversion factors from the ODOT guide to convert the daily counts to annual ESALs. Using these annual ESALs as baseline values, we forecasted ESALs for each vehicle class without any growth resulting in the 15- and 20-year ESALs assuming construction will occur in 2022. Average counts and our calculation sheet are presented in this appendix.

**TABLE D-1**
**ESAL Calculation: Proposed Extensions for 1st and Strand Streets**
**Traffic volumes according to information provided by DKS Associates**

| | | | |
|---|---|---|---|
| **Year of Traffic Count** | 2021 | **Pavement Type** | Flexible |
| **Average Daily Traffic** | 3,400 | **Construction Year**[1] | 2022 |
| **One-way or Two-way** | Two-way | **Lane Distribution Factor** | 100 |
| **Compound Growth Rate (%)** | 0.00 | **Percent Heavy Trucks** | 2.00 |

[1]Assumes pavement put into service in the following year

| FHWA Classification | Average Daily Traffic by Classification in 2021 | Conversion Factor[2] | ESALs in 2021 |
|---|---|---|---|
| 4 | 7.1 | 135.3 | 966 |
| 5 | 35.7 | 57.2 | 2,042 |
| 6 | 10.7 | 156.2 | 1,673 |
| 7 | 7.1 | 416.4 | 2,973 |
| 8 | 3.6 | 139.2 | 497 |
| 9 | 3.6 | 256.3 | 915 |
| 10 | 0 | 308.6 | 0 |
| 11 | 0 | 331.7 | 0 |
| 12 | 0 | 300.3 | 0 |
| 13 | 0 | 570.4 | 0 |

[2]Directional Factor = 55 percent

| | |
|---|---|
| Total ESALs in 2021 | 9,065 |
| ESALs in Construction Year (2022) | 9,065 |

| Year | ESALs | Cumulative ESALs[3] | Year | ESALs | Cumulative ESALs[3] |
|---|---|---|---|---|---|
| 2023 (1) | 9,065 | 18,131 | 2048 (26) | 9,065 | 244,768 |
| 2024 (2) | 9,065 | 27,196 | 2049 (27) | 9,065 | 253,833 |
| 2025 (3) | 9,065 | 36,262 | 2050 (28) | 9,065 | 262,899 |
| 2026 (4) | 9,065 | 45,327 | 2051 (29) | 9,065 | 271,964 |
| 2027 (5) | 9,065 | 54,393 | 2052 (30) | 9,065 | 281,030 |
| 2028 (6) | 9,065 | 63,458 | 2053 (31) | 9,065 | 290,095 |
| 2029 (7) | 9,065 | 72,524 | 2054 (32) | 9,065 | 299,161 |
| 2030 (8) | 9,065 | 81,589 | 2055 (33) | 9,065 | 308,226 |
| 2031 (9) | 9,065 | 90,655 | 2056 (34) | 9,065 | 317,292 |
| 2032 (10) | 9,065 | 99,720 | 2057 (35) | 9,065 | 326,357 |
| 2033 (11) | 9,065 | 108,786 | 2058 (36) | 9,065 | 335,423 |
| 2034 (12) | 9,065 | 117,851 | 2059 (37) | 9,065 | 344,488 |
| 2035 (13) | 9,065 | 126,917 | 2060 (38) | 9,065 | 353,554 |
| 2036 (14) | 9,065 | 135,982 | 2061 (39) | 9,065 | 362,619 |
| 2037 (15) | 9,065 | 145,048 | 2062 (40) | 9,065 | 371,685 |
| 2038 (16) | 9,065 | 154,113 | 2063 (41) | 9,065 | 380,750 |
| 2039 (17) | 9,065 | 163,179 | 2064 (42) | 9,065 | 389,816 |
| 2040 (18) | 9,065 | 172,244 | 2065 (43) | 9,065 | 398,881 |
| 2041 (19) | 9,065 | 181,310 | 2066 (44) | 9,065 | 407,947 |
| 2042 (20) | 9,065 | 190,375 | 2067 (45) | 9,065 | 417,012 |
| 2043 (21) | 9,065 | 199,441 | 2068 (46) | 9,065 | 426,078 |
| 2044 (22) | 9,065 | 208,506 | 2069 (47) | 9,065 | 435,143 |
| 2045 (23) | 9,065 | 217,572 | 2070 (48) | 9,065 | 444,208 |
| 2046 (24) | 9,065 | 226,637 | 2071 (49) | 9,065 | 453,274 |
| 2047 (25) | 9,065 | 235,702 | 2072 (50) | 9,065 | 462,339 |

[3]Includes ESALs in construction year as per method in ODOT Pavement Design Guide

| 2-Year ESALs | 15-Year ESALs | 20-Year ESALs | 30-Year ESALs | 40-Year ESALs | 50-Year ESALs |
|---|---|---|---|---|---|
| 18,000 | 136,000 | 181,000 | 272,000 | 363,000 | 453,000 |

NV5

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