REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Broadway Field
1400 Broadway Street
Seaside, Oregon

For
Seaside School District
April 12, 2023

Project: SeasideSD-3-01

April 12, 2023

Seaside School District
1801 South Franklin Street
Seaside, OR 97138

Attention: Susan Penrod

Report of Geotechnical Engineering Services
Broadway Field
1400 Broadway Street
Seaside, Oregon
Project: SeasideSD-3-01

NV5 is pleased to present this report of geotechnical engineering services for the proposed Broadway Field project located at 1400 Broadway Street in Seaside, Oregon. Our services for this project were conducted in accordance with our proposal dated January 26, 2023.

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

Sincerely,

NV5

Scott McDevitt, P.E., G.E.
Principal Engineer

cc: Brian Hardebeck, Otak CPM
       Josh Modin, ZCS Engineering & Architecture

TAP:SPM:kt
Attachments
One copy submitted
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EXECUTIVE SUMMARY

We understand that the Seaside School District plans to renovate the existing locker rooms and construct a new softball field, new softball and football scoreboards, a new 20-foot-tall barrier net, and associated parking and utilities at the existing Broadway Field site. The following is a summary of our findings and recommendations for use in design and construction of the proposed improvements. This executive summary is limited to an overview of the project. We recommend that the report be referenced for a thorough description of the subsurface conditions and geotechnical recommendations for the project.

The primary geotechnical considerations for the project are summarized as follows:

- In our opinion, the proposed locker room improvements can be supported on conventional shallow footings bearing on firm, undisturbed native soil or structural fill overlying undisturbed native soil. The upper portion of the native sand was found to be loose. We recommend that all shallow foundation subgrade be compacted to a depth of at least 12 inches in accordance with the "Structural Fill" section.

- The barrier net supports and scoreboards should be supported on deep foundations. Lateral resistance design parameters for deep foundations are provided in the "Drilled Pier Foundations" section.

- The shallow groundwater and sand will make excavations prone to caving, sloughing, and "running sands." Excavation sidewalls should be sloped at 1H:1V or flatter or braced with shoring. In addition, a minimum of 12 inches of stabilization material should be placed at the base of excavations if groundwater is encountered.

- The soil present at the site may be susceptible to disturbance from construction equipment during periods of wet weather or when the subgrade is saturated. If not carefully executed, site earthwork can create soft areas and repair costs can result. Subgrade protection by means of granular haul roads and working pads should be considered when the subgrade is wet of its optimum moisture content.

- Field infiltration testing results were highly variable and groundwater was encountered as shallow as 5 feet BGS. Permitting agencies typically recommend at least 5 feet of separation between the base of the infiltration facility and groundwater. If on-site stormwater disposal is implemented, we recommend that infiltration occur in the upper 2 to 3 feet of the ground surface.

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ACRONYMS AND ABBREVIATIONS

AC	asphalt concrete
ACP	asphalt concrete pavement
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BGS	below ground surface
DOGAMI	Oregon Department of Geology and Mineral Industries
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
MCE	maximum considered earthquake
OSHA	Occupational Safety and Health Administration
OSSC	2021 Oregon Standard Specifications for Construction
pcf	pounds per cubic foot
pci	pounds per cubic inch
PG	performance grade
psf	pounds per square foot
psi	pounds per square inch
SPT	standard penetration test
USGS	U.S. Geological Survey

1.0 INTRODUCTION

NV5 is pleased to submit this report of geotechnical engineering services for the proposed Broadway Field project located at 1400 Broadway Street in Seaside, Oregon. Figure 1 shows the site relative to existing topographic and physical features. Figure 2 shows the proposed site layout and the approximate locations of our explorations. Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

2.0 PROJECT UNDERSTANDING

Based on the information provided, it is our understanding that the Seaside School District plans to renovate the existing locker rooms and construct a new softball field, new softball and football scoreboards, a new 20-foot-tall barrier net, and associated parking and utilities at the existing Broadway Field site. We understand that continuous footings and isolated footings will have maximum loads of 1.5 kips per lineal foot and 10 kips, respectively. Based on existing site grades, we have assumed permanent on-site cuts and fills will be less than 2 feet.

3.0 PURPOSE AND SCOPE

The purpose of our scope was to explore subsurface conditions at the site and provide geotechnical engineering recommendations for design and construction of the proposed improvements. Specifically, we conducted the following scope of services:

- Reviewed readily available, published geologic data and our in-house files for existing information on subsurface conditions in the site vicinity.
- Coordinated and managed the field explorations, including utility locates and scheduling subcontractors and NV5 field staff.
- Explored subsurface conditions at the site by drilling ten borings to depths between 5.9 and 26.5 feet BGS using mud rotary, direct-push, and hollow-stem auger drilling techniques.
- Collected soil samples for laboratory testing and maintained a detailed log of subsurface conditions encountered in the explorations.
- Performed field infiltration testing in three borings to evaluate feasibility of on-site stormwater disposal.
- Conducted the following laboratory testing on select samples from the explorations:
    - Sixteen moisture content determinations in general accordance with ASTM D2216
    - Seven particle-size analyses in general accordance with ASTM D1140
- Provided the results of infiltration testing and general recommendations for on-site stormwater disposal.
- Provided recommendations for site preparation, grading and drainage, compaction criteria for both on-site and imported materials, fill type for imported material, use of on-site soil, and wet weather earthwork.
- Evaluated groundwater conditions at the site and provided general recommendations for site drainage.
- Provided foundation support recommendations for the proposed improvements, including preferred foundation type, allowable capacity, settlement estimates, and lateral response parameters.

- Provided recommendations for design and construction of concrete slab-on-grade structures, including an anticipated value for subgrade modulus.
- Provided seismic design coefficients in accordance with ASCE 7-16.
- Evaluated liquefaction potential at the site.
- Provided general recommendations for the construction of AC pavement for on-site parking areas, including subbase, base course, and AC paving thickness.
- Prepared this report documenting our explorations, findings, conclusions, and recommendations.

4.0 SITE CONDITIONS

4.1 GEOLOGIC CONDITIONS

The site is located on the Northern Oregon Coastal Plain that resides on the western flank of the Coast Range physiographic province. The Northern Oregon Coastal Plain is composed of a series of marine terraces flanked by ocean beaches to the west and Coast Range uplands to the east. The marine terraces represent wave-cut platforms formed on marine bedrock and contain early Pleistocene deposits left during past high sea level stands. The marine terraces have been tectonically uplifted and faulted to their present position and deeply weathered and incised by coastal streams. The site is located on the east margin of the Necanicum River flood plain, which flows north from the Coast Range and onto the Coastal Plain.

The surficial geologic unit at the site is mapped as Quaternary alluvial deposits comprised of unconsolidated clay, silt, sand, and gravel (Schlicker et al., 1973). The alluvium is underlain by Oligocene to Miocene (35 million to 20 million years before present) sedimentary bedrock consisting of tuffaceous siltstone with minor sandstone and claystone units.

4.2 SURFACE CONDITIONS

The site is currently occupied by Broadway Field, which is part of the Sunset Recreation Center. The development area includes the landscaped field north of the existing buildings and the existing football and baseball fields to the northeast. While the sports fields are covered by synthetic turf, the ground surface in the improvement areas is covered with grass and paved with AC. The overall site is bordered by residential properties to the north, Neawanna Creek to the east, Broadway Street to the south, and the Oregon Coast Highway to the west. The site is relatively level with the exception of the Neawanna Creek bank on the east border of the site. The bank slopes gently (estimated 10H:1V or flatter) downward toward the creek to the east.

4.3 SUBSURFACE CONDITIONS

Subsurface conditions were explored by drilling ten borings (B-1 through B-10) to depths between 5.9 and 26.5 feet BGS using mud rotary, direct-push, and hollow-stem auger drilling techniques. To supplement the data collected from our explorations at the site, we also reviewed subsurface information from past NV5 project sites in the vicinity.

The locations of our explorations are shown on Figure 2. The exploration logs and laboratory testing results are presented in the Appendix.

4.3.1 Soil Conditions

In general, subsurface conditions at the site generally consist of native sand with varying proportions of silt and gravel and occasional cobbles to the maximum depths explored, with exception of B-9 where basalt bedrock was encountered beneath the sand. The sand particles are generally fine to medium grained and the gravel/cobbles particles are generally rounded to subangular. SPT results indicate that the near-surface sand is generally loose and becomes medium dense to dense at depths below approximately 5 to 7 feet BGS. Boring B-10 encountered dense to very dense sand with gravel at 15 feet BGS. Laboratory testing indicates the moisture content of the sand ranged between 5 and 33 percent at the time of our explorations, and fines content ranges from 4 to 11 percent.

Weathered basalt bedrock was encountered at 16.5 feet BGS in B-9 and extends to 16.9 feet BGS where the boring met refusal, likely on intact basalt bedrock. The weathered basalt was classified as very dense, black gravel with silt and sand. Boring B-1 was also terminated due to refusal on what potentially could be basalt bedrock at a depth of 5.9 feet BGS.

4.3.2 Groundwater

Groundwater was observed at depths between 5 and 8 feet BGS in borings completed with hollow-stem augers and the direct-push borings (B-3, B-5, B-7, and B-8). The drilling fluid in borings completed using mud rotary drilling methods precluded direct measurement of groundwater levels. Groundwater conditions are expected to fluctuate in response to seasonal changes, tidal changes, prolonged rainfall, changes in surface topography, and other factors not observed in this study.

4.4 INFILTRATION TESTING

Infiltration testing was conducted in borings B-1, B-2, and B-4 at depths between 2.5 and 3 feet BGS using the encased falling head procedure. Results of the field infiltration testing and laboratory testing on samples below the infiltration testing depth are summarized in Table 1.

Table 1. Infiltration Test Results

Exploration	Depth (feet BGS)	Material	Observed Infiltration Rate (inches per hour)	Percent Fines[1]
B-1	3	Sand	>200	4
B-2	2.5	Sand	7.5	5
B-4	3	Sand	88	4

1. Percent passing the U.S. Standard No. 200 sieve

4.5 GEOLOGIC HAZARDS

4.5.1 Liquefaction and Lateral Spreading

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Silty soil with low plasticity is moderately susceptible to liquefaction under relatively higher levels of ground shaking.

DOGAMI mapping indicates that the site is located in an area with a moderate risk of liquefaction (DOGAMI, 2023). We completed a liquefaction analysis to evaluate the possible magnitude of liquefaction settlement in loose to medium dense sand layers encountered in the explorations. While our explorations only extended to a maximum depth of 26.5 feet BGS, geologic mapping and our prior experience in the site area indicates that medium dense to very dense sand extends to the basalt bedrock. Basalt bedrock was encountered at depths of 5.9 and 16.5 feet BGS in two of our explorations. Based on our analysis, the risk of liquefaction is low to moderate. In the event that liquefaction occurs in loose zones of native sand, we anticipate that settlement experienced at the ground surface will likely be less than 2 inches, with differential settlement up to approximately 1 inch over a distance of 50 feet.

Lateral spreading is a liquefaction-related seismic hazard and occurs on gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face, such as a riverbank. Liquefied soil adjacent to an open face can flow toward the open face, resulting in lateral ground displacement. In the event of the design-level earthquake, lateral spreading may occur along the Neawanna Creek bank, but we expect that ground displacement in the improvement areas will be negligible.

4.5.2 Fault Rupture

The closest mapped fault is an unnamed fault mapped approximately 7.7 miles to the west (Personius, 2002). The fault is part of the Cascadia fold and fault belt. Since faults are not mapped beneath the site, we conclude that the probability of surface fault rupture beneath the site is low.

4.5.3 Tsunami

The site is located within the tsunami hazard zone for the Cascadia subduction zone event according to DOGAMI's Tsunami Inundation Map (DOGAMI, 2023).

5.0 DESIGN

5.1 SHALLOW FOUNDATIONS

5.1.1 General

We anticipate that improvements for the locker rooms and other ancillary structures, such as bleachers, will be established on shallow foundations. Based on the results of our explorations, laboratory testing, and analysis, it is our opinion that the site soil should be capable of supporting improvements on conventional spread footings or concrete pads (bleachers). The upper portion of the native sand was found to be loose. We recommend that all shallow foundation subgrade be compacted to a depth of at least 12 inches in accordance with the "Structural Fill" section. During periods of wet weather, the sand subgrade may be difficult to compact adequately and may need to be replaced with imported granular material. We recommend a minimum 12-inch removal depth.

As discussed in the “Liquefaction and Lateral Spreading” section, up to 2 inches of liquefaction-induced settlement is possible during the design-level earthquake. Shallow foundations will be subject to up to 2 inches of total liquefaction-induced settlement and up to 1 inch of differential settlement over a distance of 50 feet. ASCE 7-16 indicates that ground improvement and foundation ties are not required based on the anticipated differential settlement magnitude. The structural engineer should verify that project-specific settlement tolerances are met.

Additional recommendations for concrete slabs that support structures are provided in the “Concrete Slabs” section. Recommendations for deep foundations are provided in the “Deep Foundations” section.

5.1.2 Dimensions and Capacities

Continuous wall and isolated spread footings should be at least 16 and 20 inches wide, respectively. The bottom of exterior footings should be at least 18 inches below the lowest adjacent exterior grade. The bottom of interior footings should be established at least 12 inches below the base of the slab.

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

5.1.3 Settlement

Shallow foundations with bearing pressures up to 2,500 psf should experience post-construction settlement of less than 1 inch. Differential settlement of up to one-half of the total settlement magnitude can be expected between adjacent footings with similar loads. We expect much of the settlement will occur during construction as loads are applied. This does not include potential settlement from liquefaction, as discussed previously.

5.1.4 Resistance to Sliding

Lateral loads on footings can be resisted by passive earth pressure on the sides of structures and by friction on the base of footings. Our analysis indicates the available passive earth pressure for footings confined by on-site soil and structural fill is 300 pcf, modeled as an equivalent fluid pressure. 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 slabs, pavement, or the upper 12-inch depth of unpaved areas should not be considered when calculating passive resistance. In addition, in order to rely on passive resistance, a minimum of 10 feet of horizontal clearance must exist between the face of the footings and any adjacent down slopes.

For footings in contact with imported granular material or native soil, a coefficient of friction equal to 0.40 may be used when calculating resistance to sliding.

5.1.5 Subgrade Evaluations

All footing subgrade should be evaluated by a member of our geotechnical staff. Observations should also evaluate whether all deleterious material, organic material, unsuitable fill, prior topsoil zones, and disturbed subgrade (if present) have been removed and native soil subgrade has not dried excessively. Localized deepening of footing excavations may be required to penetrate debris, fill, disturbed, dried, or deleterious material, if encountered.

5.2 DEEP FOUNDATIONS

We recommend that the barrier net supports and the scoreboards be established on deep foundations to resist overturning moments. We recommend that drilled concrete piers be used for deep foundation support. Recommendations for drilled piers are presented below.

5.2.1 Drilled Pier Foundations

A drilled pier foundation system will likely consist of concrete piers drilled open-hole into the native sand. We recommend that drilled piers be embedded at least 5 feet below finished grade and proportioned using a net allowable end bearing pressure of 5 kips per square foot. We expect that the depth of foundations will be determined based on lateral loads, torsion, and uplift capacity. Uplift capacity is derived from side friction and the weight of the pier. We recommend that side friction be computed using a uniform adhesion value of 250 psf. This value includes a safety factor of 2.0. The dead weight of the pier can be added to the frictional capacity without reducing by a safety factor.

We estimate that settlement of drilled piers due to static loading will be ½ inch or less, provided the pier excavation is prepared in accordance with the "Construction Considerations" section. This estimate does not include elastic compression of the piers, which is also expected to be small, or potential liquefaction-induced settlement.

5.2.1.1 Lateral Resistance Design Parameters

Lateral response of pier foundations should be estimated using the LPILE computer software program, or similar. The recommended soil parameters for development of p-y curves and use with LPILE are presented in Table 2. If a passive resistance value is used for design of deep foundations, we recommend using a value of 300 pcf, provided that up to 1 inch of lateral displacement is acceptable at the top of the foundation.

Table 2. LPILE Input Parameters

Depth (feet BGS)	LPILE Soil Type	Unit Weight* (pcf)	Friction Angle, ϕ	Static Soil Modulus, k* (pci)
0 to 7.5	Sand (Reese)	105(43)	31	50(43)
Greater than 7.5	Sand (Reese)	115(53)	33	100(70)

*Saturated unit weight and submerged k values in (). Assume groundwater at 5 feet BGS.

5.2.1.2 Construction Considerations

The base of the excavated pier cavity should be relatively free of excess debris resulting from pier excavation. This may require a cleanout barrel or bucket to be turned at the base of the excavation when the desired design depths are achieved.

We recommend careful observation of the drilled pier foundation installation be conducted by qualified personnel to verify that subsurface soil conditions are as anticipated. Drilled piers should be installed with suitable alignment tolerances. Drilled piers with steel reinforcement cages should be installed with a vertical alignment within 5 percent of plumb. Lateral alignment should be within tolerances determined by the design team.

Due to the shallow groundwater and presence of relatively clean sand, temporary steel casing and/or installation by the slurry method may be required to install the drilled piers. The base of the excavated pier cavity should be relatively free of excess debris resulting from pier excavation. This will require a cleanout barrel or bucket to be turned at the base of the excavation when the desired design depths are achieved. Cobbles in the sand soil may lead to difficult drilled pier excavations as they have the potential to “roll” around the auger and cause belling or caving of the pier sidewalls. A core barrel, mud bucket, or other enclosed auger has proved successful on other jobs for removing cobbles and boulders from pier excavations.

If a pier is poured in the “wet,” concrete must be placed at the bottom of the pier cavity using a tremie pipe. If water is not present in an excavation, concrete may be placed using the “free fall” method, provided a centralizer is used to ensure that the concrete does not contact the rebar cage on its flight to the pier bottom and “separation” of the concrete is prevented.

5.3 SEISMIC DESIGN CRITERIA

Site Class F is applicable for the site since the soil is vulnerable to liquefaction. ASCE 7-16 Section 20.3.1 requires a site-specific ground motion analysis be performed for structures with a fundamental period (T) greater than 0.5 second that have a site class of F. If the fundamental period of the structures is less than 0.5 second, they can be designed using the site class without regard to liquefaction.

We anticipate the structures at the site will have a fundamental period of less than 0.5 second and that seismic design parameters can be determined using Site Class D, provided exception 3 in ASCE 7-16 Section 11.4.8 is met. The site class determination is based on SPT blow counts from the borings and the assumption that basalt bedrock exists at depths of less than 50 feet BGS. If the fundamental period of a structure is greater than 0.5 second, a site-specific seismic analysis will be required. Table 3 provides seismic design parameters in accordance ASCE 7-16.

Table 3. Seismic Design Parameters

Parameter	Short Period (T_s)	1 Second Period (T_1)
MCE Spectral Acceleration, S	S_s = 1.295 g	S_1 = 0.680 g
Site Class	D*	
Site Coefficient, F	F_a = 1.0	F_v = 1.7
Adjusted Spectral Acceleration, S_M	S_{MS} = 1.295 g	S_{M1} = 1.156 g
Design Spectral Response Acceleration Parameters, S_D	S_{DS} = 0.860 g	S_{D1} = 0.771 g

* The above parameters provided for Site Class D can be used, provided structures have 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.

5.4 CONCRETE SLABS

Satisfactory subgrade support for concrete slab-on-grade structures with maximum distributed loads of 100 pcf can be obtained, provided the subgrade is prepared in accordance with the "Site Preparation" section. A modulus of reaction of 125 pci can be used for slabs-on-grade constructed on subgrade prepared as recommended in the "Site Preparation" section. A minimum 6-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade to provide uniform support and assist as a capillary break. The slab base rock should be crushed rock or crushed gravel and sand meeting the requirements outlined in the "Structural Fill" section. The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. Concrete slab base rock contaminated with excessive fines during construction (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.

5.5 PAVEMENT

Pavement for on-site driveways and parking stalls should be installed on subgrade prepared as described in the "Site Preparation" section. Our pavement recommendations are based on the following assumptions:

- Resilient moduli of 4,500 psi and 20,000 psi were estimated for the firm, undisturbed subgrade and aggregate base, respectively.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.

- Reliability of 75 percent and standard deviation of 0.45.
- Structural coefficients of 0.42 and 0.10 for the AC and aggregate base, respectively.
- A 20-year design life.
- Truck traffic consists of occasional two- and three-axle vehicles, such as delivery and garbage trucks.
- Fire access will consist of an imposed fire apparatus load of 80,000 pounds on an infrequent basis.

Design traffic loads were not available at the time of this report. NV5 performed pavement analyses for assumed loads based on our experience with similar developments. If design traffic exceeds our assumed maximum loads, we should be contacted for additional recommendations. Our recommended pavement sections for two speculative loading scenarios are provided in Table 4. The recommended pavement sections with subgrade prepared as recommended are suitable to support an occasional 80,000-pound fire truck.

Table 4. Minimum Standard Pavement Thicknesses

Pavement Area	Traffic Levels		Pavement Thicknesses[1] (inches)	
	Cars per Day	Trucks per Day	AC	Aggregate Base
Light Traffic	500	0	2.5	6.0
Heavy Traffic	500	5	3.0	8.0

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

All thicknesses are intended to be the minimum acceptable. Design of the recommended pavement section is based on the assumption that construction will be completed during an extended period of dry weather. Wet weather construction could require an increased thickness of aggregate base. In addition, the pavement sections recommended above are for support of post-construction design traffic.

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

5.6 TURF FIELD

We anticipate that the new softball field will be of synthetic turf material. The turf should be installed in accordance with the manufacturer's recommendations over a drainage layer, as discussed in the "Drainage" section. Subgrade should be prepared in accordance with the "Construction" section. The greatest demand on the subgrade will be during construction, when earthwork equipment performs grading work. Subgrade protection will be important to the long-term performance of the field, especially during the wet season. We recommend that a subsurface drainage system be considered below the field, as discussed in the "Drainage" section.

5.7 DRAINAGE

5.7.1 Temporary

During work at the site, the contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface. During rough and finished grading of the site, the contractor should keep all pads and subgrade free of ponding water.

5.7.2 Surface

The ground surface at finished pads should be sloped away from their edges at a minimum 2 percent gradient for a distance of at least 5 feet. Roof drainage from buildings should be directed into solid, smooth-walled drainage pipes that carry the collected water to the storm drain system.

5.7.3 Turf Drainage

The thickness of the aggregate base course for the turf field will likely be controlled by subgrade support during construction. Subsurface drainpipes can be installed below the turf to convey water to the stormwater disposal system or infiltration trenches may be suitable for on-site stormwater disposal. The turf base can consist of aggregate base or drain rock, as discussed in the “Structural Fill” section. In general, a minimum 6-inch-thick layer of drainage aggregate in conjunction with drainage lines (AdvanEdge or similar installed in a herringbone arrangement with a spacing of approximately 15 feet center-to-center) is required to convey water to perimeter drains. Additional thickness can be considered as a contingency for subgrade protection, as discussed in the “Subgrade Protection” section.

5.7.4 Stormwater Infiltration Systems

The results of our infiltration testing indicate that the on-site soil has highly variable infiltration capacity. In addition, groundwater was generally observed as shallow as 5 feet BGS in our explorations. Permitting agencies typically recommend at least 5 feet of separation between the base of the infiltration facility and groundwater. If on-site stormwater disposal is implemented, we recommend that infiltration occur in the upper 2 to 3 feet of the ground surface. Due to the relatively high groundwater, an overflow system should be considered for off-site stormwater disposal during periods of high groundwater. Based on the highly variable infiltration rates observed during field testing, we suggest applying a safety factor of at least 3 for design of infiltration systems. The infiltration system design engineer should determine and apply appropriate remaining correction factor values or factors of safety to account for the degree of in-system filtration, system maintenance, vegetation, potential for siltation, etc.

5.8 PERMANENT SLOPES

Permanent cut and fill slopes should not exceed 2H:1V. Access roads, concrete slabs, and pavement should be located at least 5 feet from the top of cut and fill slopes. The setback should be increased to 10 feet for buildings. The 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.

6.0 CONSTRUCTION

6.1 SITE PREPARATION

6.1.1 Stripping and Grubbing

In existing landscaped areas, the near-surface root zone should be stripped and removed from the site in all proposed improvement areas and for a 5-foot margin surrounding such areas. Based on our subsurface explorations, the depth of stripping will be approximately 2 inches, although greater stripping depths may be encountered in isolated areas. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaped areas.

6.1.2 Subgrade Improvement

The near-surface soil across the majority of the site consists of loose sand with varying proportions of silt. Loose sand may provide poor support for pavement and slabs. We recommend the subgrade in slab and pavement areas be improved by compacting the upper 12 inches of the on-site soil to meet structural fill requirements.

As discussed in the “Structural Fill” section, the native soil may need to be dried to compact adequately. While compaction of the subgrade is the best option for subgrade improvement, it may not be possible during periods of persistent wet weather. If the on-site soil is too wet to achieve adequate compaction, it should be replaced with imported granular material.

6.1.3 Subgrade Observation

After completion of stripping/topsoil removal and subgrade improvement, and before fill, slabs, or pavement is placed, 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. A member of our geotechnical staff should observe proof rolling to evaluate yielding of the ground surface. When the subgrade is above the optimum moisture content for compaction and during wet weather, subgrade evaluation should be performed by probing with a foundation probe rather than proof rolling. Areas that appear soft or loose should be removed and replaced with structural fill in accordance with subsequent sections of this report.

6.2 SUBGRADE PROTECTION

The near-surface soil present on this site can become disturbed and may not be able to support construction traffic when saturated. If not carefully executed, site preparation, utility trench work, and excavation can create soft areas and repair costs can result. Earthwork planning, regardless of the time of year, should include considerations for minimizing subgrade disturbance.

When the subgrade is saturated, site preparation may need to be accomplished using track-mounted equipment loading into trucks supported on granular haul roads, working blankets, or existing pavement. Based on our experience, at least 8 to 12 inches of granular material are typically required for light staging areas and at least 12 to 18 inches of granular material for haul roads subject to repeated heavy equipment traffic. We recommend that imported granular material for haul roads and working blankets consist of durable crushed rock that is well graded and has less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. The granular material should be placed in a single lift and the surface compacted until well keyed. Although we have presented typical recommendations for haul roads and working blankets, the actual thickness and material should be determined by the contractor based on their sequencing of the project and the type and frequency of construction equipment.

6.3 EXCAVATION

6.3.1 General

Groundwater was encountered between 5 and 8 feet BGS in our explorations. Caving may be experienced at all excavation depths. If groundwater is encountered in excavations, sloughing, caving, and "running sands" will likely occur. Accordingly, the contractor should expect to flatten excavations or shore excavations, as described below, where water is encountered. In addition to safety considerations, caving and loss of ground will increase backfill volumes and can result in damage to adjacent structures or utilities.

6.3.2 Excavation Slopes and Trench Shoring

The shallow groundwater and sand will make excavations prone to caving, sloughing, and "running sands." Excavation sidewalls should be sloped at 1H:1V or flatter. Excavations should be flattened to 1.5H:1V or flatter if excessive sloughing occurs. Approved temporary shoring is recommended where slopes are not possible. If box shoring is used, it should be understood by the contractor that box shoring is a safety feature used to protect workers and does not prevent caving. If excavations are left open for extended periods, caving of the sidewalls may occur. The presence of caved material will limit the ability to properly backfill and compact the trenches. The contractor should be prepared to fill voids between the box shoring and the sidewalls of the trenches with sand or gravel before caving occurs.

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

6.3.3 Dewatering

We anticipate that a sump located within the trench excavation will likely be sufficient to remove up to 2 feet of accumulated water, depending on the amount and persistence of water seepage and the length of time for which the trench is left open. More intensive dewatering will be necessary for excavations that extend more than approximately 2 feet below groundwater. 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 and fine-grained soil are present in the base of the utility trench excavation, we recommend over-excavating the trench by 12 to 18 inches and placing trench stabilization material in the base. Placement of geotextile separation fabric may also be necessary prior to placing the stabilization material of very soft to soft soils.

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

6.3.4 Safety

All excavations should be made in accordance with applicable OSHA and state regulations. While we have described certain approaches to utility trench excavations in the foregoing discussion, the contractor should be responsible for selecting the excavation and dewatering methods, monitoring the trench excavations for safety, and providing shoring as required to protect personnel and adjacent areas.

6.4 MATERIALS

6.4.1 Structural Fill

Structural fill should only be placed over subgrade that has been prepared in conformance with the “Site Preparation” section. A variety of materials may be used as structural fill at the site. However, all material used as structural fill should be free of organic material and other deleterious material and, in general, should consist of particles no larger than 6 inches in diameter, 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.

6.4.1.1 On-Site Soil

The near-surface soil at the site is sand with varying proportions of silt. The native soil can be used for structural fill, provided it can be adequately moisture conditioned and is free of debris, organic material, and particles over 6 inches in diameter.

We estimate the optimum moisture content for compaction to be approximately 10 to 18 percent for the on-site soil. Optimum compaction typically occurs within 3 percent of optimum moisture. Moisture conditioning (drying) may be required to use the on-site sand following periods of precipitation. The on-site sand may be suitable for use as structural fill during periods of light to moderate precipitation, depending on the fines content. It may not be possible to use the on-site sand as structural fill during heavy precipitation.

When used as structural fill, the on-site sand should be placed in lifts with a maximum uncompacted thickness of 8 to 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

6.4.1.2 Imported Granular Material

Imported granular material used during periods of wet weather, for building pad subgrade, and for staging areas should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The imported granular material 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 a minimum of two mechanically fractured faces.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 8 to 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

6.4.1.3 Trench Backfill

Trench backfill for the utility pipe base and pipe zone should consist of durable, well-graded, granular material containing no organic material or other deleterious material; should have a maximum particle size of ¾ inch; and should have less than 8 percent by dry weight passing the U.S. Standard No. 200 sieve.

Backfill for the pipe base and to the springline of the pipe should be placed in maximum 12-inch-thick lifts and compacted to not less than 90 percent of the maximum dry density, as determined by ASTM D1557, or as recommended by the pipe manufacturer. Backfill above the springline of the pipe should be placed in maximum 12-inch-thick lifts and compacted to not less than 92 percent of the maximum dry density, as determined by ASTM D1557. Trench backfill located within 2 feet of finish subgrade elevation should be placed in maximum 12-inch-thick lifts and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

6.4.1.4 Concrete Slab Base Rock

Imported durable, granular material placed beneath concrete slabs should be clean crushed rock or crushed gravel and sand that is fairly well graded between coarse and fine. The granular material should have a maximum particle size of 1½ inches, 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 surfaces. The imported base rock should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

6.4.1.5 Stabilization Material

Stabilization material used to create haul roads for construction traffic or at the base of unstable trench subgrade should consist of pit- or quarry-run rock or crushed rock. The material should have a maximum particle size of 6 inches and less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, should have at least two mechanically fractured faces, and should be free of organic material and other deleterious material. Stabilization material should be placed in lifts between 12 and 18 inches thick and compacted to a firm condition with a smooth-drum roller without using vibratory action.

6.4.1.6 Drain Rock

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

6.4.2 Geotextile Fabric

6.4.2.1 Geotextile Separation Fabric

A separation geotextile fabric can be placed as a barrier between fine-grained subgrade and granular material in staging areas, haul road areas, or in areas of repeated construction traffic. The subgrade geotextile should meet the requirements in OSSC 02320 (Geosynthetics) for subgrade geotextiles and be installed in conformance with OSSC 00350 (Geosynthetic Installation).

6.4.2.2 Geotextile Drainage Fabric

Drain rock and other granular material used for subsurface drains should be wrapped in a geotextile fabric that meets the specifications provided in OSSC 00350 (Geosynthetic Installation) and OSSC 02320 (Geosynthetics) for drainage geotextiles and installed in conformance with OSSC 00350 (Geosynthetic Installation).

6.4.3 Conventional Pavement Materials

6.4.3.1 AC

The AC should be Level 2, ½-inch, dense ACP as described in OSSC 00744 (Asphalt Concrete Pavement) and compacted to 92 percent of the specific gravity of the mix, as determined by ASTM D2041. Minimum and maximum lift thicknesses for ½-inch, dense ACP are 2 and 3.5 inches, respectively. ACP should be placed at the minimum ground surface temperatures described in OSSC 00744.40 (Season and Temperature Limitations). Asphalt binder should be performance graded and conform to PG 64-22. The binder grade should be adjusted depending on the aggregate gradation and amount of recycled asphalt and/or recycled asphalt shingles in the contractor's mix design submittal.

6.4.3.2 Pavement Aggregate Base

The crushed aggregate base rock should consist of ¾- or 1½-inch-minus material meeting the requirements in OSSC 00641 (Aggregate Subbase, Base, and Shoulders), with the exception that the crushed base rock should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. The crushed base rock should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557.

6.5 EROSION CONTROL

Earthwork is feasible during the rainy season, provided proper erosion control procedures are implemented and the "Subgrade Protection" and "Structural Fill" sections are followed. The site soil is moderately susceptible to erosion; therefore, erosion control measures should be carefully planned and in place before construction begins. Surface water runoff should be collected and directed away from slopes to prevent water from running down the slope face. Erosion control measures (such as straw bales, sediment fences, and temporary detention and settling basins) should be used in accordance with local and state ordinances.

7.0 OBSERVATION OF CONSTRUCTION

Satisfactory pavement, earthwork, and foundation 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. NV5 should be retained to observe subgrade preparation, fill placement, foundation excavations, drainage system installation, and pavement placement and to review laboratory compaction and field moisture-density information.

Subsurface conditions observed during construction should be compared to those encountered during the subsurface explorations. Recognition of changed conditions requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

8.0 LIMITATIONS

We have prepared this report for use by the Seaside School District and members of the design team for the proposed project. The data and report can be used for bidding or estimating purposes, but the 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 or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

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

The scope 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, express or implied, should be understood.

◆ ◆ ◆

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

Sincerely,

NV5

Tyler A. Pierce, P.E.
Senior Project Engineer

Scott McDevitt, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
91210PE
OREGON
July 12, 2016
TYLER PIERCE
EXPIRES: 12/31/24

REFERENCES

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FIGURES

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

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SEASIDESD-3-01

APRIL 2023

VICINITY MAP

BROADWAY FIELD
SEASIDE, OR

FIGURE 1

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File Name: J:\S-Z\SeasideSD\SeasideSD-3\SeasideSD-3-01\Figures\CAD\SeasideSD-3-01-VM01.dwg | Layout: FIGURE 1

LEGEND:

— — — SITE BOUNDARY

B-1 ◐ BORING

SITE FEATURES

1. RELOCATED SOFTBALL FIELD
2. PARKING LOT
3. NEW SOFTBALL SCOREBOARD
4. NEW SOFTBALL SCOREBOARD
5. NEW NET BARRIER
6. NEW FOOTBALL SCOREBOARD

N

0 80 160

(SCALE IN FEET)

SITE PLAN BASED ON AERIAL PHOTOGRAPH DATED OCTOBER 18, 2018, OBTAINED FROM GOOGLE EARTH PRO.

FIGURE 2

SITE PLAN

BROADWAY FIELD
SEASIDE, OR

SEASIDESD-3-01

APRIL 2023

NV5

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APPENDIX

APPENDIX

FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at the site by drilling ten borings (B-1 through B-10) to depths between 5.9 and 26.5 feet BGS using mud rotary, direct-push, and hollow-stem auger drilling techniques. The borings were completed by Western States Soil Conservation, Inc. of Hubbard, Oregon, on March 2 and 3, 2023. The exploration logs are presented in this appendix.

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

SOIL SAMPLING

Samples were collected from the borings using a 1½-inch-inside diameter split-spoon (SPT) samplers in general accordance with ASTM D1586. The split-spoon samplers were driven into the soil with a 140-pound 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 exploration logs, unless otherwise noted. Sampling intervals are shown on the exploration logs.

The average efficiency of the automatic SPT hammer used by Western States Soil Conservation, Inc. was 74.3 percent. The calibration testing results are presented in this appendix.

SOIL CLASSIFICATION

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

LABORATORY TESTING

We visually examined soil samples collected from the explorations to confirm field classifications. We also performed the following laboratory testing to evaluate the engineering properties of the soil.

MOISTURE CONTENT

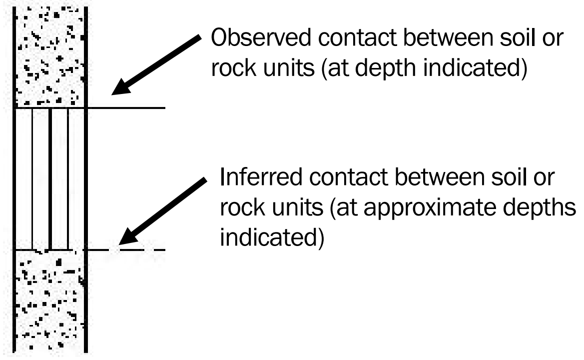
We determined the natural moisture content of select soil samples in general accordance with ASTM D2216. The natural moisture content is a ratio of the weight of the water to the dry weight of soil in a test sample expressed as a percentage. The test results are presented in this appendix.

PARTICLE-SIZE ANALYSIS

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

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

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EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOIL

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

CONSISTENCY - FINE-GRAINED SOIL

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

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

MOISTURE CLASSIFICATION

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

ADDITIONAL CONSTITUENTS

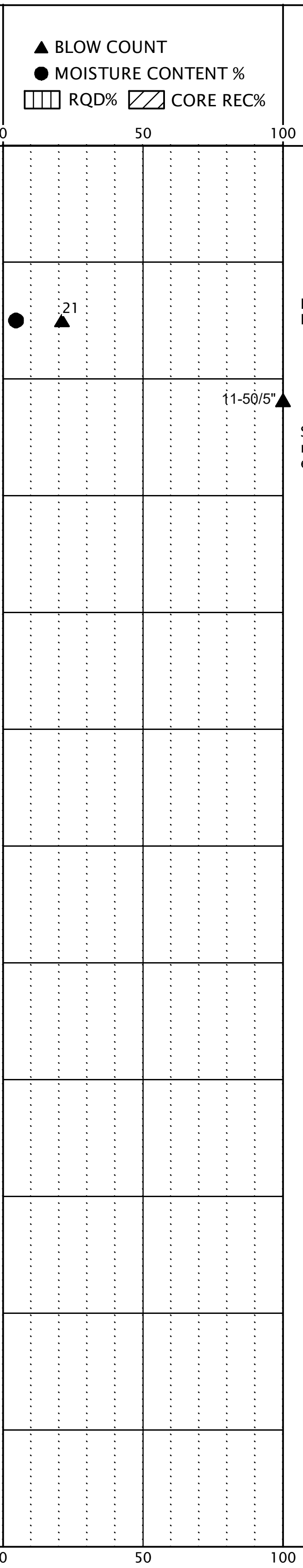
Secondary granular components or other materials such as organics, man-made debris, etc.

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

SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Dense, orange-brown SAND with cobbles (SP), trace silt; moist, sand is fine to medium, cobbles are approximately 40% (2-inch-thick root zone).			
2.5	medium dense, with gravel; gravel is fine to coarse and rounded to subangular at 3.0 feet		P200	Infiltration test at 3.0 feet. P200 = 4%
5.0	very dense at 5.0 feet Exploration terminated at a depth of 5.9 feet due to refusal.	5.9		Surface elevation was not measured at the time of exploration.
7.5	Hammer efficiency factor is 74.3 percent.			
10.0				
12.5				
15.0				
17.5				
20.0				
22.5				
25.0				
27.5				
30.0				



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: S. Freeman
COMPLETED: 03/02/23

BORING METHOD: hollow-stem auger (see document text)
BORING BIT DIAMETER: 8 inches

NV5

SEASIDESD-3-01

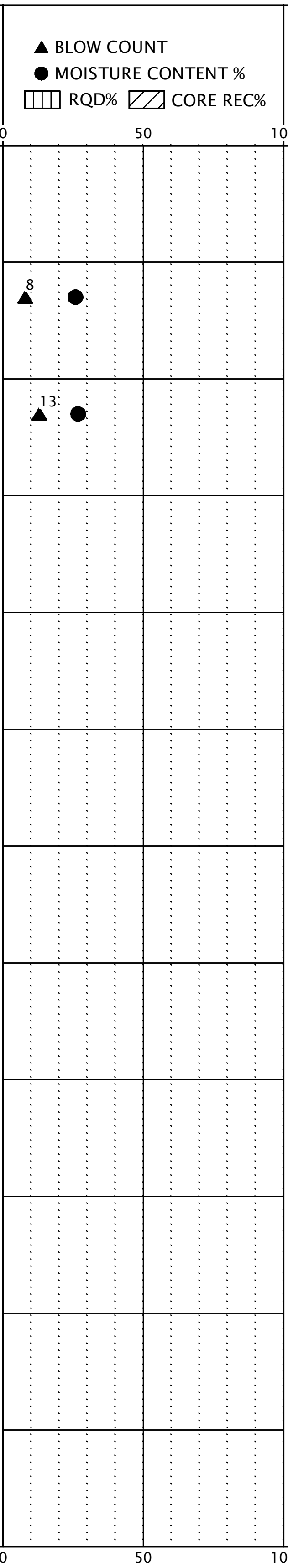
APRIL 2023

BORING B-1

BROADWAY FIELD
SEASIDE, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT	INSTALLATION AND COMMENTS
0.0	Loose, brown, silty SAND with cobbles (SM); moist, sand is fine to medium, cobbles are approximately 40% (2-inch-thick root zone). without cobbles at 2.0 feet					
2.5	Loose, brown SAND with silt (SP-SM); moist, sand is fine to medium.	2.5	P200	▮	8	Infiltration test at 2.5 feet. P200 = 5%
5.0	medium dense at 5.0 feet			▮	13	
6.5	Exploration completed at a depth of 6.5 feet. Hammer efficiency factor is 74.3 percent.	6.5				Surface elevation was not measured at the time of exploration.



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: S. Freeman

COMPLETED: 03/02/23

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

BORING BIT DIAMETER: 8 inches

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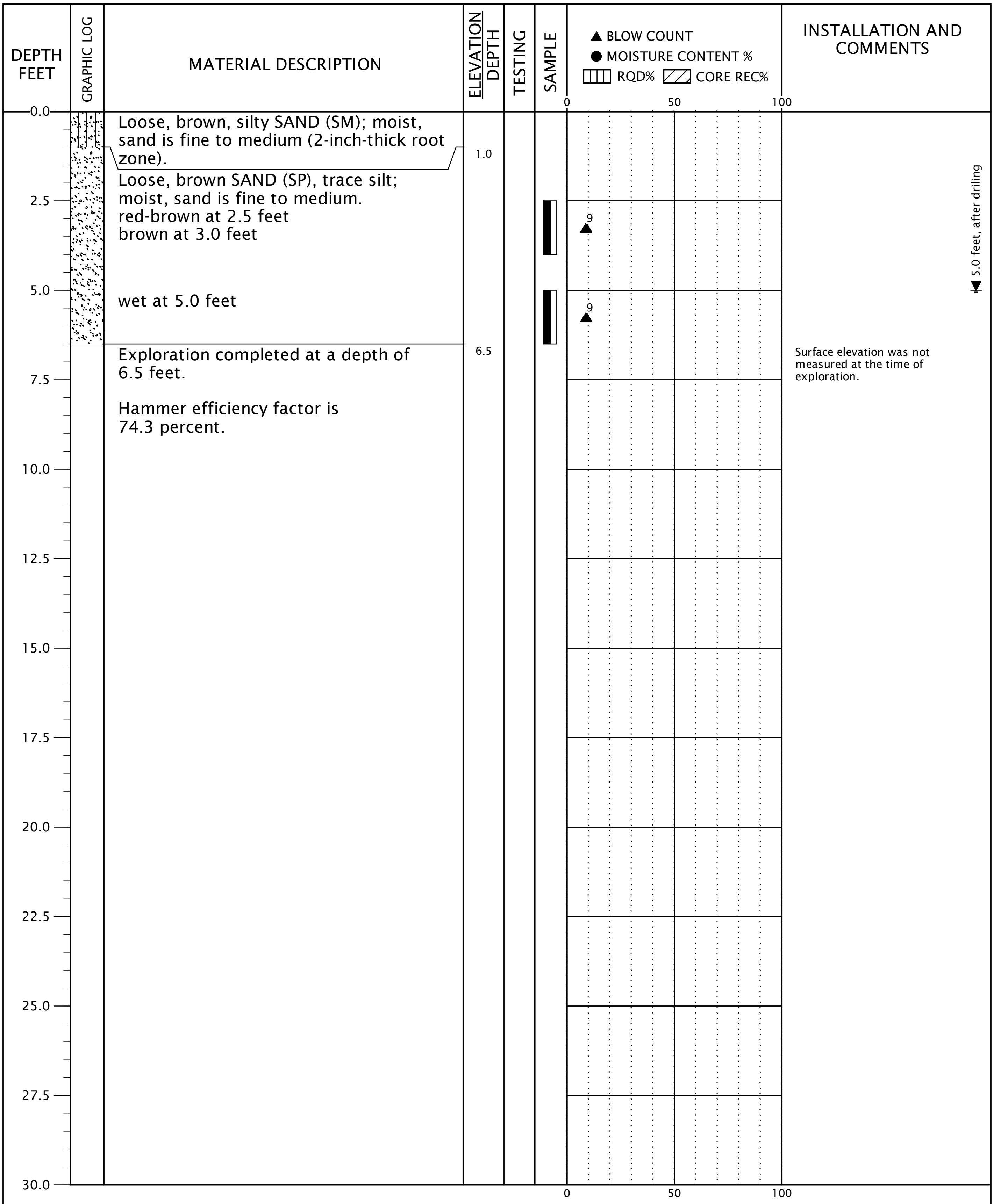
SEASIDESD-3-01

APRIL 2023

BORING B-2

BROADWAY FIELD
SEASIDE, OR

FIGURE A-2



Loose, brown, silty SAND (SM); moist, sand is fine to medium (2-inch-thick root zone).

Loose, brown SAND (SP), trace silt; moist, sand is fine to medium.
red-brown at 2.5 feet
brown at 3.0 feet

wet at 5.0 feet

Exploration completed at a depth of 6.5 feet.

Hammer efficiency factor is 74.3 percent.

Surface elevation was not measured at the time of exploration.

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: S. Freeman
COMPLETED: 03/02/23

BORING METHOD: hollow-stem auger (see document text)
BORING BIT DIAMETER: 8 inches

SEASIDESD-3-01

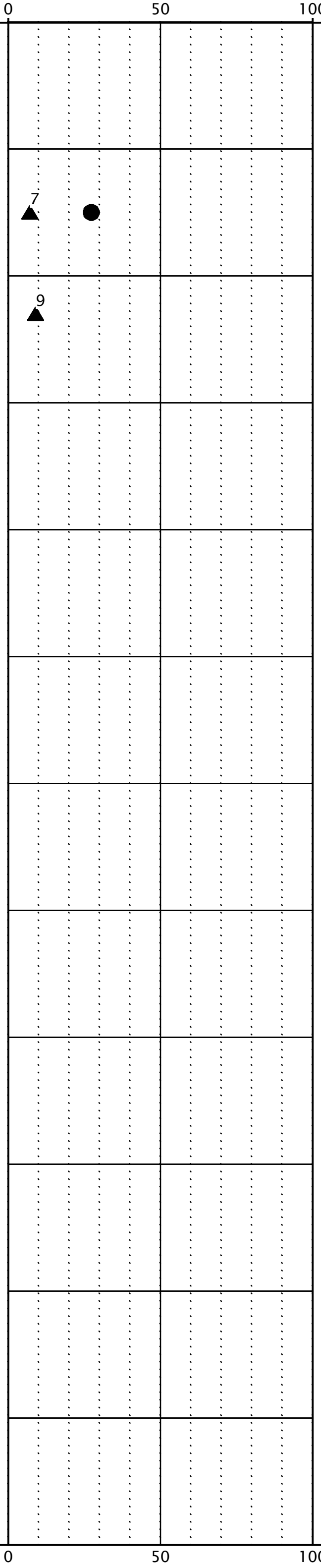
APRIL 2023

BORING B-3

BROADWAY FIELD
SEASIDE, OR

FIGURE A-3

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC%	INSTALLATION AND COMMENTS
0.0		Loose, brown, silty SAND (SM); moist, sand is fine to medium (2-inch-thick root zone).					
2.5		Loose, brown SAND with gravel and cobbles (SP), trace silt; moist, sand is fine to medium, gravel is fine to coarse and rounded, cobbles are approximately 25%. without gravel and cobbles at 3.0 feet	1.0	P200		7	Infiltration test at 3.0 feet. P200 = 4%
5.0						9	
7.5		Exploration completed at a depth of 6.5 feet. Hammer efficiency factor is 74.3 percent.	6.5				Surface elevation was not measured at the time of exploration.
10.0							
12.5							
15.0							
17.5							
20.0							
22.5							
25.0							
27.5							
30.0							



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: S. Freeman

COMPLETED: 03/02/23

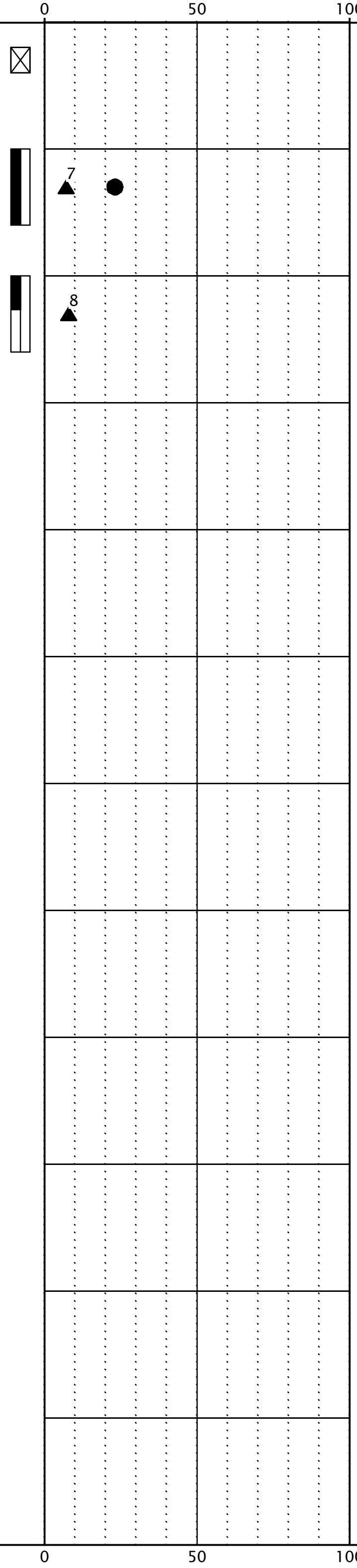
BORING METHOD: hollow-stem auger (see document text)

BORING BIT DIAMETER: 8 inches

NV5	SEASIDESD-3-01	**BORING B-4**
	APRIL 2023	BROADWAY FIELD SEASIDE, OR — **FIGURE A-4**

MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
Loose, brown, silty SAND (SM); moist, sand is fine to medium (2-inch-thick root zone).	1.0	
Loose, red-brown SAND (SP), trace silt; moist, sand is fine to medium. brown at 3.0 feet	6.5	▼ 5.0 feet, after drilling
Exploration completed at a depth of 6.5 feet. Hammer efficiency factor is 74.3 percent.		Surface elevation was not measured at the time of exploration.

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



DEPTH FEET: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0, 22.5, 25.0, 27.5, 30.0

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: S. Freeman
COMPLETED: 03/02/23

BORING METHOD: hollow-stem auger (see document text)
BORING BIT DIAMETER: 8 inches

NV5

SEASIDESD-3-01

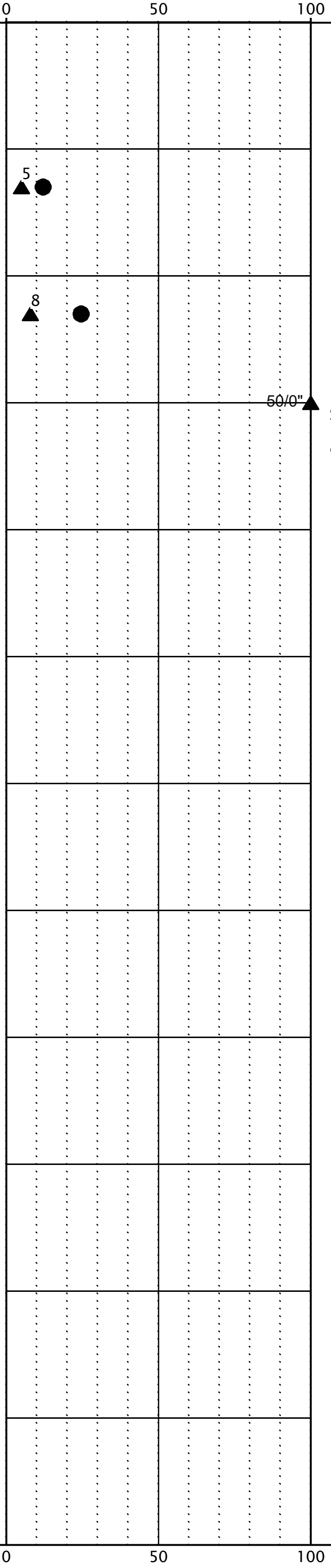
APRIL 2023

BORING B-5

BROADWAY FIELD
SEASIDE, OR

FIGURE A-5

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC%	INSTALLATION AND COMMENTS
0.0		Loose, orange-brown SAND (SP), trace silt; moist, sand is fine to medium.					
5.0		brown at 5.0 feet					
7.5		Exploration terminated at a depth of 7.5 feet due to possible utility conflict. Hammer efficiency factor is 74.3 percent.	7.5			50/0"	Surface elevation was not measured at the time of exploration.



Depth scale: 0.0, 2.5, 5.0, 7.5, 10.0, 12.5, 15.0, 17.5, 20.0, 22.5, 25.0, 27.5, 30.0

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: S. Freeman
COMPLETED: 03/02/23

BORING METHOD: direct push (see document text)
BORING BIT DIAMETER: 2 1/4 inches

NV5

SEASIDESD-3-01

APRIL 2023

BORING B-6

BROADWAY FIELD
SEASIDE, OR

FIGURE A-6

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
0.0	Dense, brown and gray GRAVEL with silt and sand (GP-GM), minor debris (metal); moist, gravel is fine to coarse and subangular to angular, sand is fine to coarse - **FILL**.				
2.5	Loose, red-brown to brown SAND with silt (SP-SM); moist, sand is fine to medium.	2.0		▲ 8	
5.0	brown; wet at 5.0 feet		P200	▲ 6 ●	P200 = 5% 5.5 feet, during drilling 8.0 feet, after drilling
7.5	medium dense at 7.5 feet			▲ 16	
10.0	moist at 10.0 feet			▲ 20	
12.5					
15.0	gray at 15.0 feet			▲ 20	
17.5	Exploration completed at a depth of 16.5 feet. Hammer efficiency factor is 74.3 percent.	16.5			Surface elevation was not measured at the time of exploration.
20.0					
22.5					
25.0					
27.5					
30.0					

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: S. Freeman
COMPLETED: 03/02/23

BORING METHOD: hollow-stem auger (see document text)
BORING BIT DIAMETER: 8 inches

NV5

SEASIDESD-3-01

APRIL 2023

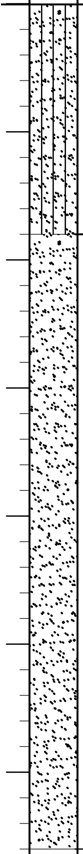
BORING B-7

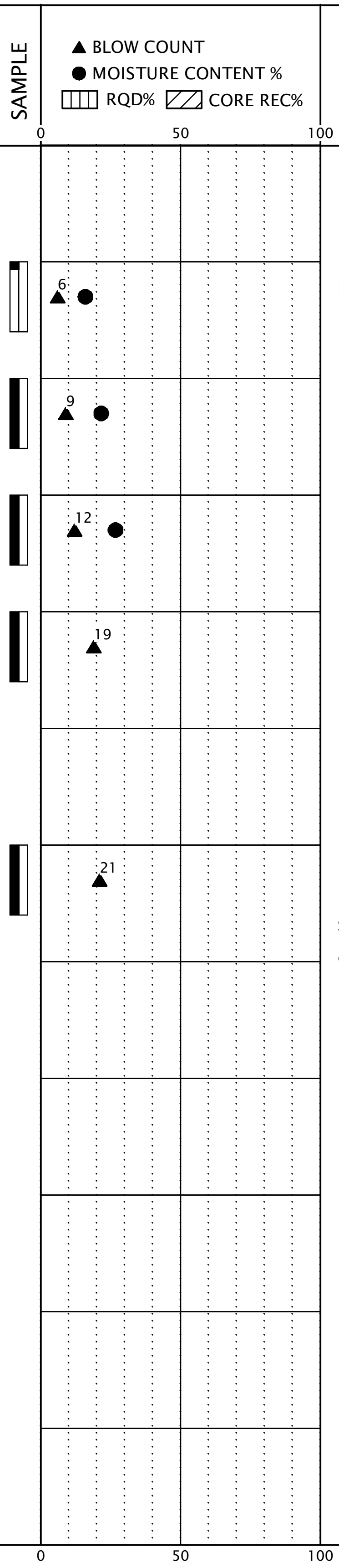
BROADWAY FIELD
SEASIDE, OR

FIGURE A-7

BORING LOG - NV5 - 1 PER PAGE SEASIDESD-3-01-B1_10.GPJ GDI_NV5.GDT PRINT DATE: 4/12/23:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	Loose, red-brown, silty SAND (SM); moist, sand is medium (2-inch-thick root zone).				
2.5			P200	6	P200 = 11%
	Loose, red-brown to brown SAND (SP), trace silt; moist, sand is fine to medium.	4.5			
5.0				9	▽ 6.4 feet, during drilling
7.5	medium dense, brown; wet at 7.5 feet			12	
10.0				19	
12.5					
15.0				21	
	Exploration completed at a depth of 16.5 feet. Hammer efficiency factor is 74.3 percent.	16.5			Surface elevation was not measured at the time of exploration.
17.5					
20.0					
22.5					
25.0					
27.5					
30.0					





DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: S. Freeman

COMPLETED: 03/02/23

BORING METHOD: direct push (see document text)

BORING BIT DIAMETER: 2 1/4 inches

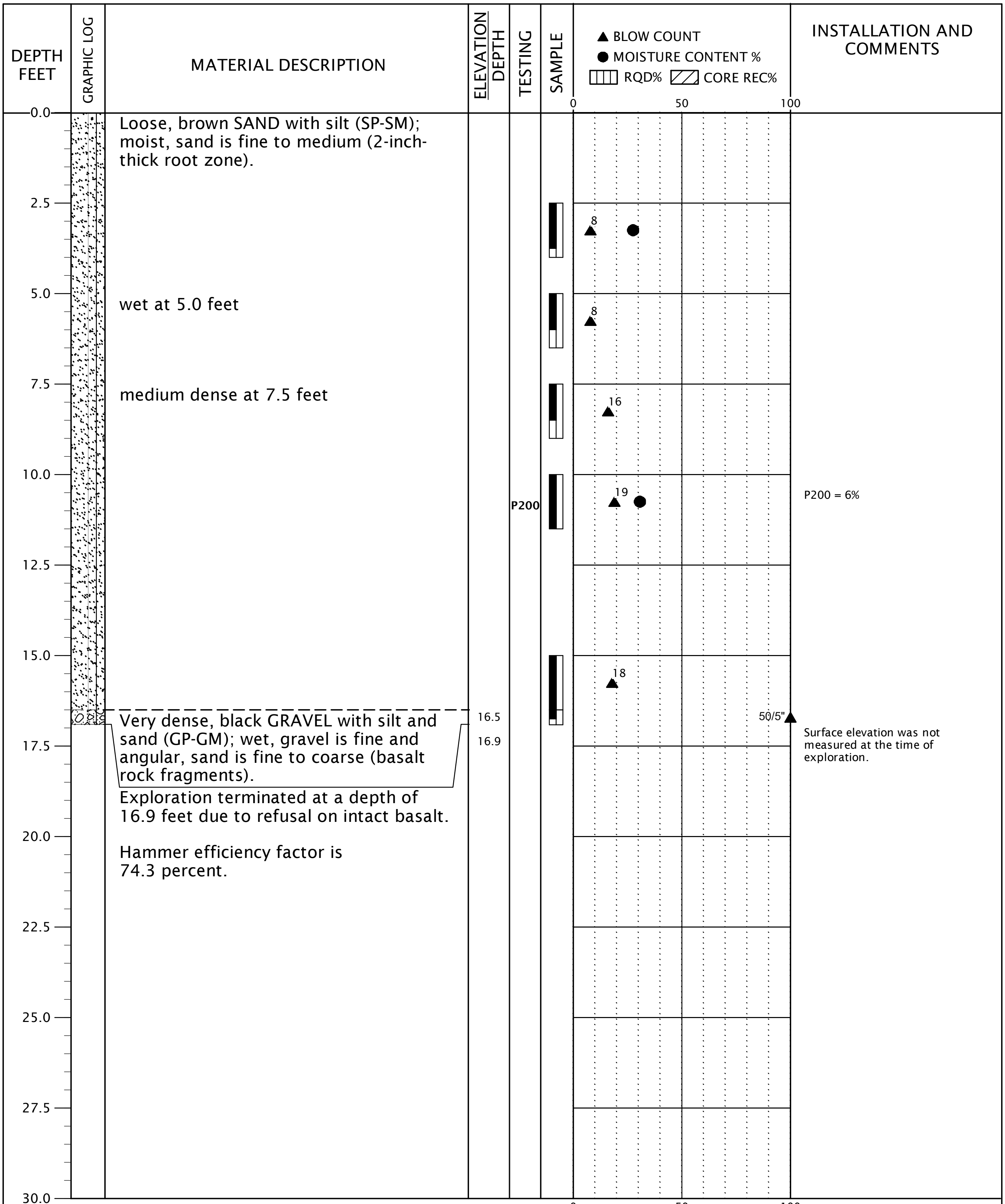
SEASIDESD-3-01

BORING B-8

APRIL 2023

BROADWAY FIELD
SEASIDE, OR

FIGURE A-8



Loose, brown SAND with silt (SP-SM); moist, sand is fine to medium (2-inch-thick root zone).

wet at 5.0 feet

medium dense at 7.5 feet

Very dense, black GRAVEL with silt and sand (GP-GM); wet, gravel is fine and angular, sand is fine to coarse (basalt rock fragments).

Exploration terminated at a depth of 16.9 feet due to refusal on intact basalt.

Hammer efficiency factor is 74.3 percent.

P200 = 6%

Surface elevation was not measured at the time of exploration.

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: S. Freeman
COMPLETED: 03/03/23

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

NV5

SEASIDESD-3-01

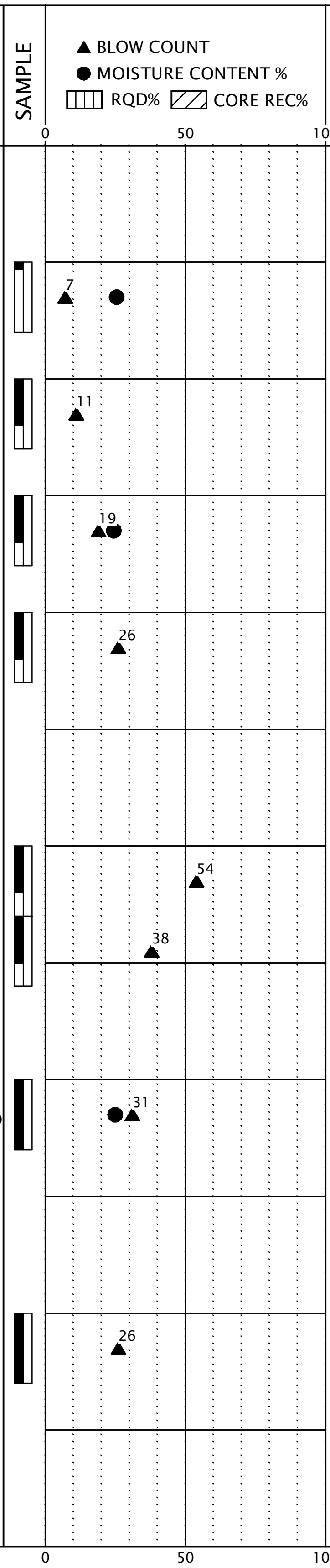
APRIL 2023

BORING B-9

BROADWAY FIELD
SEASIDE, OR

FIGURE A-9

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Loose, brown SAND with silt (SP-SM); moist, sand is fine to medium.			
2.5				
5.0	medium dense at 5.0 feet			
7.5				
10.0	with cobbles; cobbles are approximately 10% at 9.0 feet minor gravel; gravel is fine to coarse and rounded to subangular at 10.0 feet			
12.5	without cobbles at 12.5 feet			
15.0	very dense; gravel is rounded to subangular (fragments) at 15.0 feet dense; sand is fine to coarse at 16.5 feet			
17.5				
20.0			P200	P200 = 8%
22.5				
25.0	medium dense at 25.0 feet			
27.5	Exploration completed at a depth of 26.5 feet. Hammer efficiency factor is 74.3 percent.	26.5		Surface elevation was not measured at the time of exploration.
30.0				



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: S. Freeman

COMPLETED: 03/03/23

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 3 7/8 inches

SEASIDESD-3-01

APRIL 2023

BORING B-10

BROADWAY FIELD
SEASIDE, OR

FIGURE A-10

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	3.0		5				4			
B-2	2.5		26				5			
B-2	5.0		27							
B-4	3.0		27				4			
B-5	2.5		23							
B-6	2.5		12							
B-6	5.0		25							
B-7	5.0		33				5			
B-8	2.5		16				11			
B-8	5.0		22							
B-8	7.5		27							
B-9	2.5		28							
B-9	10.0		31				6			
B-10	2.5		25							
B-10	7.5		24							
B-10	20.0		25				8			

NV5

SEASIDESD-3-01

APRIL 2023

SUMMARY OF LABORATORY DATA

BROADWAY FIELD
SEASIDE, OR

FIGURE A-11

LAB SUMMARY - GDI-NV5 SEASIDESD-3-01-B1_10.GPJ GDI_NV5.GDT PRINT DATE: 4/12/23:KT

Summary of SPT Test Results

Project: WSSC-8-06, Test Date: 12/23/2021

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Blows Applied /6"	**N Value**	**N60 Value**	Average FMX kips	Average VMX ft/s	Average BPM bpm	Average EFV ft-lb	Average ETR %
60.00	**1-2-4**	**6**	**7**	51	13.7	50.5	225	64.3
60.00	**2-4-8**	**12**	**14**	50	14.6	51.0	228	65.0
60.00	**4-4-5**	**9**	**11**	53	16.1	50.8	260	74.2
60.00	**3-6-10**	**16**	**19**	49	14.6	51.1	274	78.4
60.00	**3-3-5**	**8**	**9**	52	16.5	51.3	270	77.0
60.00	**9-9-10**	**19**	**23**	50	14.8	51.2	276	78.8
			Overall Average Values:	50	15.0	51.0	260	74.3
			Standard Deviation:	3	1.2	0.5	21	6.0
			Overall Maximum Value:	55	17.6	52.3	285	81.5
			Overall Minimum Value:	39	13.0	48.2	219	62.7

NV5

Delivering Solutions
Improving Lives