July 7, 2023

Willamette University
900 State Street
Salem, OR 97301

Attention: Mark Mazurier

Report of Geotechnical Engineering Services
Willamette University Softball Field Improvements
501 14th Street SE
Salem, Oregon
Project: WillUniv-16-01

INTRODUCTION

NV5 is pleased to submit this report of geotechnical engineering services for the proposed Willamette University softball field improvements located at 501 14th Street SE in Salem, Oregon. We understand the proposed improvements will include installation of synthetic turf and on-site stormwater disposal systems for the softball field. In addition, new safety netting will be installed in a portion of the outfield. We assume that netting poles will be supported by intermediate foundations.

Our services for this project were conducted in accordance with our proposal dated September 30, 2022. Figure 1 shows the site vicinity relative to the surrounding features. Figure 2 shows the softball field area and our approximate exploration locations.

PURPOSE AND SCOPE

The purpose of our scope was to explore subsurface conditions at the site and provide geotechnical recommendations for the proposed field improvements. Our specific scope of services is summarized as follows:

- Reviewed geotechnical and geologic information provided for the site and information from our in-house project files for projects in the site vicinity.
- Coordinated our field explorations, including utility locates and scheduling subcontractors and NV5 field staff.

- Explored subsurface soil and groundwater conditions for the proposed improvements by conducting the following explorations and testing:
  - Drilled five borings depths between 5.5 and 9 feet below ground surface (BGS) using a trailer-mounted drill rig.
  - Performed four infiltration tests at depths between 1 foot and 4 feet BGS.
- Classified the material encountered in the explorations, maintained a detailed log of each exploration, and collected samples at representative intervals.
- Conducted the following laboratory testing program:
  - Thirteen moisture content determinations in general accordance with American Society for Testing and Materials (ASTM) D2216
  - Four particle-size analyses in general accordance with ASTM D1140 or ASTM C117
- Provided recommendations for site preparation, grading and drainage, stripping depths, fill type for imported material, compaction criteria, trench excavation and backfill, use of on-site soil, and wet/dry weather earthwork.
- Provided geotechnical design parameters for safety netting pole foundations.
- Provided recommendations for managing identified groundwater conditions that may affect the performance of structures or pavement.
- Provided recommendations for American Society of Civil Engineers (ASCE) 7-16 seismic coefficients and evaluated the risk of liquefaction and lateral spreading at the site.
- Prepared this geotechnical engineering report summarizing the results of our geotechnical evaluation.

SITE CONDITIONS

GEOLOGIC SETTING

The site is located along the southern margin of the Northern Willamette Valley physiographic province. The Willamette Valley is bound by the Coast Range to the west and the Cascade Range to the east. The geologic profile in the vicinity of the site consists of approximately 65 feet of fluvial gravel deposits underlain by older fluvial terrace deposits that extend to a depth of approximately 160 to 170 feet BGS.

The near-surface geologic unit is mapped as late-Pleistocene Age (36,000 to 10,000 years before present) Linn gravel (Qlg) that consists of fine to coarse fluvial gravel (Bela, 1981; Yeats et al., 1991). The Linn gravel is underlain by the middle Pleistocene Age (36,000 to 30,000 years before present) high terrace deposits (Qth) that consist of semi-consolidated deposits of sand, silt, and clay forming a broad, flat terrace along the Willamette River. The upper portion (10 to 30 feet) of the terrace is comprised of silt deposits. The middle Miocene Age (16 million to 6 million years before present) Columbia River Basalt Group (Tcr) underlies the terrace deposits and forms the basement geologic unit at this site (Bela, 1981; Yeats et al., 1991; Gannett and Caldwell, 1998).

SURFACE CONDITIONS

The softball field is located on the Willamette University campus. The field is surrounded by university buildings and an asphalt concrete (AC)-paved parking lot to the north, a gravel parking lot to the south, the Salem Amtrak Station to the west, and 14th Street SE to the east. The ground surface of the softball field is relatively flat, with surface elevations ranging from 167 to 168 mean sea level based on topography available on Google Earth. The infield is bare, with exposed soil, and the outfield is covered with grass.

SUBSURFACE CONDITIONS

General

Subsurface conditions at the field were explored by drilling five borings (B-1 through B-5) to depths between 5.5 and 9 feet BGS. The approximate exploration locations are shown on Figure 2. The exploration logs and laboratory testing results are presented in the Attachment. Subsurface conditions generally consist of undocumented sand and gravel fill overlying native gravel to the maximum depth explored. A 3-inch-thick root zone was encountered at the ground surface. The following sections present descriptions of the soil units encountered.

Undocumented Fill

Undocumented fill was encountered below the root zone to depths between 1 foot and 6 feet BGS in all of the borings at the site. The undocumented fill consists of sand with varying amounts of silt and silty gravel with sand. The sand fill generally overlies the gravel fill. Sand particles are fine to medium grained. Standard penetration test (SPT) results indicate the sand is very loose to medium dense and the gravel is loose. Laboratory testing of the fill soil indicates moisture contents of 7 to 20 percent and fines contents between 15 and 17 percent at the time of our explorations.

Silt

A single layer of sandy silt was encountered in boring B-5 at a depth of 5 feet BGS. The silt layer is 6 inches thick and SPT results indicate the silt is soft.

Gravel

Native gravel with silt and sand was encountered below the undocumented fill in all of the borings at the site. The gravel is generally rounded to subrounded. Although not encountered in our explorations, it is likely that cobbles exist in the gravel unit. SPT results indicate that the gravel is medium dense to very dense. Laboratory testing indicates moisture contents between 8 and 14 percent at the time of our explorations. Fines content analysis of one native gravel sample indicates a fines content of 11 percent.

Groundwater

Groundwater was not encountered in the borings performed within the softball field. Regional groundwater is mapped at depths between 7 and 15 feet BGS. The depth to groundwater may fluctuate in response to seasonal changes, changes in surface topography, and other factors not observed in this study.

INFILTRATION TESTING

Infiltration testing was conducted to evaluate the feasibility of on-site stormwater disposal in borings B-2 through B-5 at depths between 1 foot and 4 feet BGS. Testing was conducted using the encased falling head method in the borings using either a PVC pipe or an open hole method. A representative soil sample was collected below the infiltration test depths for particle-size analysis after infiltration testing.

Table 1 summarizes the infiltration testing results and fines content determinations. The exploration logs and results of particle-size analysis are presented in the Attachment.

Table 1. Infiltration Testing Results

Exploration	Depth (feet BGS)	Soil Description	Fines Content[1] (percent)	Observed Infiltration Rate[2] (in/hr)
B-2	4	GRAVEL with silt and sand (GP-GM)	11	9.1
B-3	2	Silty SAND (SM) – Fill	17	4.6
B-4	1.5	Silty SAND (SM) – Fill	15	7.2
B-5	1	Silty SAND (SM) – Fill	15	8.8

1. Fines content – material passing the U.S. Standard No. 200 sieve
2. In-situ infiltration rate observed in the field

in/hr: inches per hour

CONCLUSIONS AND RECOMMENDATIONS

SEISMIC DESIGN

Seismic Design Parameters

Based on our knowledge of local geology, the native gravel layer encountered at depths between 1 foot and 6 feet BGS likely extends to bedrock or to a depth of at least 100 feet BGS. Based on this assumption, the soil profile is consistent with Site Class C in accordance with ASCE 7-16. The values presented in Table 2 can be used to compute design levels of ground shaking.

Table 2. Seismic Design Parameters

Parameter	Short Period (T_s)	1 Second Period (T_1)
MCE Spectral Acceleration, S	S_s = 0.822 g	S_1 = 0.413 g
Site Class	C	
Site Coefficient, F	F_a = 1.2	F_v = 1.5
Adjusted Spectral Acceleration, S_M	S_{MS} = 0.986 g	S_{M1} = 0.619 g
Design Spectral Response Acceleration Parameters, S_D	S_{DS} = 0.658 g	S_{D1} = 0.413 g

g: gravitational acceleration (32.2 feet/second^2)

MCE: maximum considered earthquake

Liquefaction

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the sudden loss of shear strength in a soil. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Low plasticity, sandy silt may be moderately susceptible to liquefaction under relatively higher levels of ground shaking.

Based on the depth to groundwater and the medium dense to very dense gravel present at the site, liquefaction is not considered a hazard.

Lateral Spreading

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.

Based on the soil conditions, site topography, and distance from an open face, lateral spreading is not considered a hazard at the site.

INFILTRATION SYSTEMS

The infiltration testing results presented in Table 1 can be used to design stormwater disposal facilities. The infiltration rates shown in Table 1 are short-term field rates and factors of safety have not been applied. Appropriate correction factors should be applied by the project civil engineer to determine long-term infiltration parameters. From a geotechnical perspective, we recommend a minimum factor of safety of 2 be applied to the field infiltration rates presented in Table 1 to account for soil variability. The infiltration system design engineer should determine the appropriate remaining correction factor values to account for maintenance, vegetation, siltation, etc.

It is important to establish on-site stormwater disposal systems near the locations and depths where the testing was performed in order to rely on the tested field rates. The actual infiltration rates of installed systems can vary from the values presented in Table 1. We recommend the design infiltration values for the stormwater disposal system be confirmed by field testing completed during installation. The results of the field testing might necessitate that the system be enlarged to achieve the design infiltration rate.

The infiltration flow rate of a disposal system will diminish over time as suspended solids and precipitates in the stormwater slowly clog the void spaces between the soil particles. Eventually, the infiltration system may fail and will need to be replaced. We recommend the infiltration system include an overflow that is connected to a suitable discharge point. Finally, infiltration systems will cause localized high groundwater levels; therefore, the infiltration system should not be located near basement walls, retaining walls, or other embedded structures, unless these are specifically designed to account for the resulting hydrostatic pressure. The infiltration system should not be located on sloping ground, unless it is approved by a geotechnical engineer.

TURF FIELD

We anticipate that the playing field will consist 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 "Site Preparation" section. The greatest demand on the subgrade will be during construction when earthwork equipment performs grading work. Subgrade protection will be critical to the long-term performance of the field, especially during the wet season. Cement amendment of the subgrade could be considered for subgrade protection during construction, especially during wet weather construction. Not only will cement-amended subgrade provide protection during original construction, it will also provide subgrade protection if the turf needs to be replaced in the future.

DRAINAGE

The turf base can consist of aggregate base or drain rock as discussed in the "Structural Fill" section. Subsurface drainpipes should be installed within the aggregate base course to convey water to the stormwater disposal system. 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. However, the thickness of the aggregate base course for the turf field will likely be controlled by subgrade support during construction. It may be necessary to increase the aggregate base layer to 12 inches during the wet season to support repeated construction traffic. Alternatively, the subgrade below the drainage layer can be cement amended instead of increasing aggregate thickness, as discussed in the "Subgrade Protection" section. We note that cement-amended soil has very low permeability and will preclude direct stormwater infiltration.

The turf drainage system should be capable of handling flow from high groundwater that could occur during periods of extreme precipitation. If a drainage shock pad is used (such as the Brock PowerBase), then a reduced thickness of drainage aggregate can be used in conjunction with a stabilized subgrade.

FOUNDATION SUPPORT

We recommend that the safety netting supports be established on intermediate foundations to resist overturning moments. We recommend that drilled concrete piers be used for deep foundation support. Recommendations for drilled piers are presented below.

Drilled Pier Foundations

A drilled pier foundation system will likely consist of concrete piers drilled open-hole into the native gravel. 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 300 pounds per square foot. 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.

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 3. If a passive resistance value is used for design of deep foundations, we recommend using a value of 300 pounds per cubic foot (pcf) in the sand/gravel fill material and 375 pcf in the native gravel material, provided that up to 1 inch of lateral displacement is acceptable at the top of the foundation.

Table 3. LPILE Input Parameters

Depth (feet BGS)	LPILE Soil Type	Unit Weight (pcf)	Friction Angle, ϕ	Static Soil Modulus, k (pci)
0 to 4	Sand (Reese)	115	30	30
Greater than 4	Sand (Reese)	130	40	250

pci: pounds per cubic inch

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.

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.

SITE PREPARATION

Stripping

The existing root zone should be stripped and removed from all improvement areas. Based on our explorations, the root zone thickness is approximately 3 inches, although greater stripping depths may be required to remove localized zones of loose or organic soil. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or as required by the project specifications. Given the moisture-sensitive subgrade, special construction procedures will likely be required to protect the subgrade as discussed below.

Subgrade Protection

Earthwork planning, regardless of the time of year, should include considerations for minimizing subgrade disturbance. Therefore, we recommend the following considerations to assist in protecting the subgrade from damage and the associated risk of repair or reconstruction of the subgrade and drainage layer.

- To the extent possible, construction traffic should be track-mounted vehicles.
- Heavy construction traffic should only be allowed on the field after subgrade protection measures are in place. During the dry season, subgrade protection can likely be accomplished with a minimum of 6 inches of aggregate base. During the wet season, this thickness may need to be increased to 12 inches or the subgrade below the aggregate base should be cement amended in accordance with the "Cement Amendment" section.
- Grading should be completed with track-mounted equipment, with finished grading kept to a minimum with lightweight graders.
- Compaction should be completed using maximum 3-foot rollers and carefully monitored such that vibration does not damage the subgrade soil.

We note that these procedures will also be required if the turf is replaced at later date.

Subgrade Evaluation

A member of our geotechnical staff should observe exposed structural subgrade after stripping and site cutting have been completed to determine if there are areas of unsuitable or unstable soil. Our representative should observe a proof roll of structural fill, pavement, and field subgrade with a fully loaded dump truck or similar heavy, rubber tire construction equipment to identify soft, loose, or unsuitable areas. In areas not accessible to proof rolling equipment, the subgrade should be evaluated by probing. Areas identified as soft, unstable, or otherwise unsuitable should be over-excavated and replaced with compacted material recommended for structural fill. Areas that appear too wet or soft to support proof rolling or compaction equipment should be evaluated by probing and prepared in accordance with the "Subgrade Protection" section.

EXCAVATION

General

Conventional earthmoving equipment in proper working conditions should generally be capable of making necessary excavations for site cuts and utilities in the on-site soil. Excavation difficulty will increase in the native gravel. Vertical excavation sidewalls will likely experience caving in the sand and gravel soil. Open excavation techniques may be used, provided the walls of the excavation are cut at a slope of 1.5 horizontal to 1 vertical and groundwater seepage is not present. In lieu of large and open cuts, approved temporary shoring may be used for excavation support. A variety of shoring systems are available; consequently, we recommend that the contractor be responsible for selecting the appropriate system.

If box shoring is used, it should be understood that box shoring is a safety feature used to protect workers and does not prevent caving. 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.

Excavations should be made in accordance with applicable Occupational Safety and Health Administration and state regulations. While this report describes certain approaches to excavation, the contractor should be responsible for selecting excavation methods, dewatering, monitoring the excavations for safety, and providing shoring as required to protect personnel and adjacent utilities and structures.

Dewatering

Groundwater was not observed within the borings. Groundwater could rise during periods of persistent wet weather. For shallow excavations less than approximately 10 feet BGS, it should be possible to remove groundwater encountered by pumping from a sump. Removed water should be routed to a suitable discharge point. While we have described certain approaches to excavation dewatering, it is the contractor's responsibility to select the dewatering methods.

TEMPORARY DRAINAGE

In addition to the erosion control measures (see “Erosion Control” section) during mass grading 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 prepared subgrade free of water.

EROSION CONTROL

The site contains predominantly coarse-grained material, but in some areas, the fine-grained soil at this site is eroded easily by wind and water; therefore, erosion control measures should be carefully planned and in place before construction begins, if necessary. Measures that can be employed to reduce erosion include the use of silt fences, hay bales, buffer zones of natural growth, sedimentation ponds, and granular haul roads. All erosion control methods should be in accordance with local jurisdiction standards. During earthwork at the site, the contractor should be responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface.

MATERIALS

Structural Fill

General

Fill should be placed on subgrade that has been prepared in conformance with the "Site Preparation" section. A variety of material may be used as structural fill at the site. However, all material used as structural fill should be free of organic material or other unsuitable material and should meet the specifications provided in 2021 Oregon Standard Specifications for Construction (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. Fill should be compacted as described in the "Fill Placement and Compaction" section.

On-Site Soil

The on-site material should generally be suitable for use as general structural fill, provided it is properly moisture conditioned; free of debris, organic material, and particles over 8 inches in diameter; and meets the specifications provided in OSSC 00330.12 (Borrow Material). The undocumented sand fill is generally suitable for use as structural fill during periods of dry weather and possibly during light precipitation. The sand will be difficult to compact during the wet season when it becomes saturated. The on-site gravel (native and fill) is rounded to subrounded. When rounded particles are touching, they will tend to roll during compaction, making it difficult to achieve compaction standards. The on-site gravel should be crushed to have at least two fractured faces or adequately nested in a sand/gravel matrix to minimize contact between rounded sides.

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 6 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have at least two fractured faces.

Aggregate Base Rock

Imported granular material used as base rock should consist of ¾- or 1½-inch-minus material (depending on the application) and meet the requirements in OSSC 00641 (Aggregate Subbase, Base, and Shoulders). The aggregate should have at least two mechanically fractured faces. In addition, the aggregate should have less than 6 percent by dry weight passing the U.S. Standard No. 200 sieve.

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 1½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.13 (Pipe Zone Material). 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

10 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.14 (Trench Backfill; Class B, C, or D).

Outside of structural improvement areas (e.g., roadway alignments or building pads), trench backfill placed above the pipe zone may consist of general fill material that is free of organic material and material over 6 inches in diameter and meets the specifications provided in OSSC 00405.14 (Trench Backfill; Class A, B, C, or D).

Stabilization Material

Stabilization material at the field will help provide a stable base for construction of the turf during dry weather. If soft areas are identified during construction, subgrade stabilization can be achieved using 12 to 24 inches of stabilization material or 12 inches of cement-amended subgrade. The stabilization material should consist of 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.15 (Selected Stone Backfill). The material should have a maximum particle size of 6 inches, should have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and should have at least two mechanically fractured faces. The material should be free of organic material and other deleterious material. Stabilization material should be placed in lifts between 12 and 24 inches thick and compacted to a firm condition.

Drain Rock

Drain rock should consist of angular, granular material with a maximum particle size of 2 inches and should meet the specifications provided in OSSC 00430.11 (Granular Drain Backfill Material). The material should be free of roots, organic material, and other unsuitable material; should have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis); and should have at least two mechanically fractured faces. Drain rock should be compacted to a well-keyed, firm condition.

Geotextile Fabric

Subgrade Geotextile

Subgrade geotextile should conform to OSSC Table 02320-4 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

Drainage Geotextile

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

Cement Amendment

General

In order to stabilize the subgrade if earthwork occurs during the wet season, an experienced contractor may be able to amend the on-site soil with portland cement to obtain suitable support properties. Successful use of soil amendment depends on the use of correct mixing techniques, soil moisture content, and amendment quantities.

Subgrade Stabilization

We recommend a target strength for cement-amended subgrade subbase (below aggregate base) soil of 150 pounds per square inch. Successful use of soil amendment depends on use of correct techniques and equipment, soil moisture content, and the amount of cement added to the soil. The recommended percentage of cement is based on soil moisture contents at the time of placing the structural fill. Based on our experience, 6 percent cement by weight of dry soil is generally satisfactory when the soil moisture content does not exceed approximately 25 percent. If the soil moisture content is in the range of 25 to 35 percent, 7 to 9 percent by weight of dry soil is recommended. It is difficult to accurately predict field performance due to the variability in soil response to cement amendment. The amount of cement added to the soil may need to be adjusted based on field observations and performance. Moreover, depending on the time of year and moisture content levels during amendment, water may need to be applied during tilling to appropriately condition the soil moisture content. The amount of cement used during amendment should be based on an assumed soil dry unit weight of 110 pcf. For preliminary design purposes, we recommend a minimum of 6 percent cement. It may be possible to reduce this to 5 percent if work occurs during the dry season. It may be necessary to inject water into the sand fill during the tilling process if the material has a low moisture content. It is not possible to amend soil during heavy or continuous rainfall. Work should be completed during suitable conditions.

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

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

Cement amendment should not be attempted when the air temperature is below 40 degrees Fahrenheit or during moderate to heavy precipitation. Cement should not be placed when the ground surface is saturated or standing water exists.

Other Considerations

Portland cement-amended soil is hard and has low permeability. This soil does not drain well and it is not suitable for planting. Future planted areas should not be cement amended, if practical, or accommodations should be made for drainage and planting. The field drainage system must be constructed over the cement-amended soil since cement-amended soil will be essentially impermeable.

Specification Recommendations

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

- In general, cement amendment is not recommended during the cold weather (temperatures less than 40 degrees Fahrenheit) or during steady rainfall.
- Mixing Equipment
  - Use a pulverizer/mixer capable of uniformly mixing the cement into the soil to the design depth. Blade mixing will not be allowed.
  - Pulverize the soil-cement mixture such that 100 percent by dry weight passes a 1-inch sieve and a minimum of 70 percent passes a No. 4 sieve, exclusive of gravel or stone retained on these sieves. If water is required, the pulverizer should be equipped to inject water to a tolerance of ¼ gallon per square foot of surface area.
  - Use machinery that will not disturb the subgrade, such as using low-pressure "balloon" tires on the pulverizer/mixer vehicle. If subgrade is disturbed, the tilling/amendment depth shall extend the full depth of the disturbance.
  - Multiple "passes" of the tiller will likely be required to adequately blend the cement and soil mixture.
- Spreading Equipment
  - Use a spreader capable of distributing the cement uniformly on the ground to within 5 percent variance of the specified application rate.
  - Use machinery that will not disturb the subgrade, such as using low-pressure "balloon" tires on the spreader vehicle. If subgrade is disturbed, the tilling/amendment depth shall extend the full depth of the disturbance.
- Compaction Equipment
  - Use a static, sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds for initial compaction of fine-grained soil (silt and clay) or an alternate approved by the geotechnical engineer.

FILL PLACEMENT AND COMPACTION

Fill soil should be compacted at a moisture content that is within 3 percent of optimum. The maximum allowable moisture content varies with the soil gradation and should be evaluated during construction. Fill and backfill material should be placed in uniform, horizontal lifts and compacted with the appropriate equipment. The maximum lift thickness will vary depending on the material and compaction equipment used, but should generally not exceed the loose thicknesses provided in Table 4. Fill material should be compacted in accordance with the compaction criteria provided in Table 5.

Table 4. Recommended Uncompacted Lift Thickness

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

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

Table 5. Compaction Criteria

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

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

OBSERVATION OF CONSTRUCTION

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

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

LIMITATIONS

We have prepared this report for use by Willamette University and members of the design and construction teams for the proposed development. The data and report can be used for estimating purposes, but our report, conclusions, and interpretations should not be construed as a warranty of the subsurface conditions and are not applicable to other sites.

Soil explorations 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 not finalized at the time this report was prepared. When the design has been finalized and if there are changes in the site grades, location, or configuration; design loads; or type of construction, the conclusions and recommendations presented may not be applicable. If design changes are made, we should be retained to review our conclusions and recommendations and to provide a written evaluation or modification.

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 the generally accepted practices in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

◆ ◆ ◆

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

Sincerely,

NV5

Staci Butler

Staci R. Butler, G.I.T.
Technical Specialist

Zane M. Rogers, P.E.
Project Engineer

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

REGISTERED PROFESSIONAL ENGINEER
63094
OREGON
JULY 11, 2000
SCOTT P. McDEVITT
EXPIRES: 12/31/24

cc: Matt Koehler, Cameron McCarthy

SRB:ZMR:SPM:kt
Attachments
One copy submitted
Document ID: WillUniv-16-01-070723-geolr.docx

REFERENCES

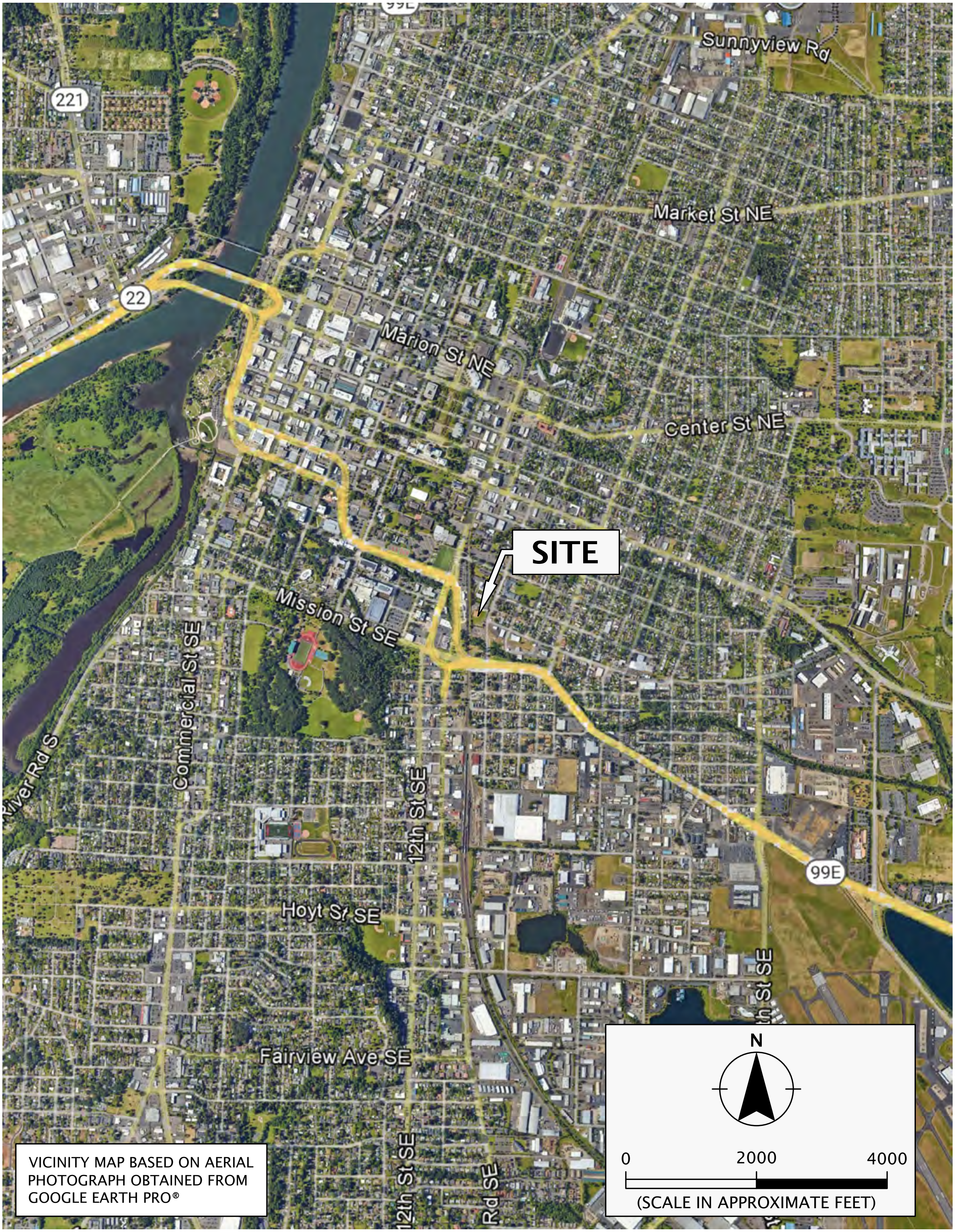
ASCE, 2016. *Minimum Design Loads and Associated Criteria for Buildings and Other Structures.* ASCE Standard ASCE/SEI 7-16.

Bela, James L., 1981, Geology of the Rickreall, Salem West, Monmouth, and Sidney 7 ½' Quadrangles, Marion, Polk, and Linn Counties, Oregon, State of Oregon Department of Geology and Mineral Industries, GMS-18, scale 1:24,000.

Gannett, Marshall W., and Caldwell, Rodney R., 1998, Geologic Framework of the Willamette Lowland Aquifer System, Oregon and Washington: U.S. Geological Survey Professional Paper 1424-A, 32p, 8 plates.

Yeats, Robert S., Graven, Erik P., Werner, Kenneth S., Goldfinger, Chris, and Popowski, Thomas A., 1991, Tectonics of the Willamette Valley, Oregon; in Assessing Earthquake Hazards and Reducing Risk in the Pacific Northwest: U. S. Geological Survey Professional Paper 1560, Vol. 1, p.183-222.

FIGURES



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

NV5

WILLUNIV-16-01

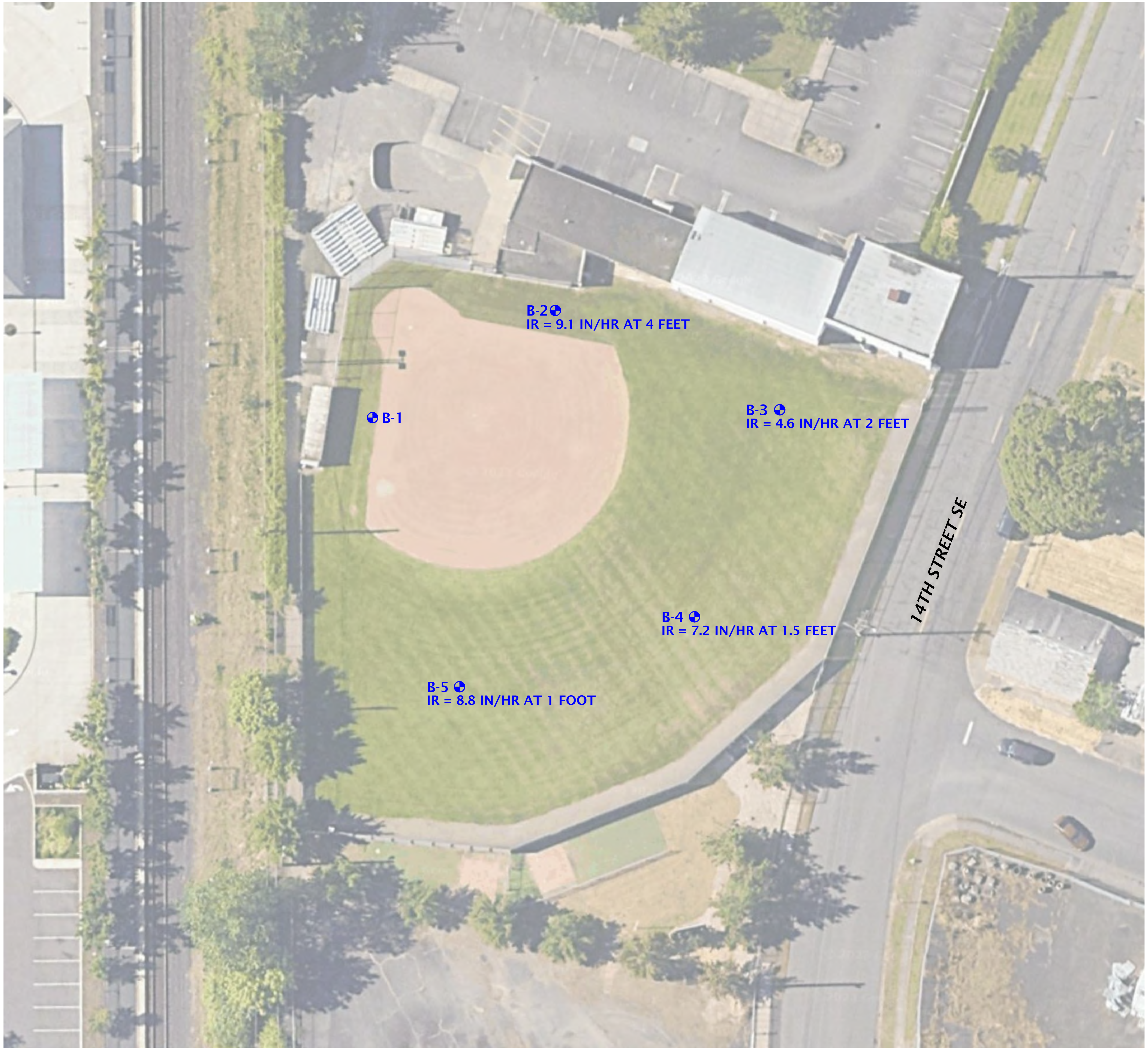
VICINITY MAP

JULY 2023

WILLAMETTE UNIVERSITY SOFTBALL FIELD
SALEM, OR

FIGURE 1

Printed By: aday | Print Date: 7/5/2023 3:51:07 PM
File Name: J:\S-Z\WillUniv\WillUniv-16\WillUniv-16-01\Figures\CAD\WillUniv-16-01-VM01.dwg | Layout: FIGURE 1



LEGEND:

B-1 ●	BORING
IR = 9.1 IN/HR AT 4 FEET	UNFACTORED INFILTRATION RATE

SITE PLAN BASED ON AERIAL PHOTOGRAPH DATED JUNE 25, 2022, OBTAINED FROM GOOGLE EARTH PRO.

FIGURE 2

SITE PLAN

WILLAMETTE UNIVERSITY SOFTBALL FIELD
SALEM, OR

WILLUNIV-16-01

JULY 2023

NV5

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ATTACHMENT

ATTACHMENT

FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at site by drilling five borings (B-1 through B-5) to depths between 5.5 and 9 feet BGS. Drilling services were provided by Dan J. Fisher Excavating, Inc. on May 31, 2023, using a trailer-mounted drill rig and solid-stem auger drilling techniques. The exploration logs are presented in this attachment.

The approximate exploration locations 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

Disturbed soil samples were collected from the drilled borings using 1½- and 3-inch-inside-diameter, split-spoon SPT samplers in general accordance with ASTM D1586. Each sampler was driven into the soil with a 140-pound hammer free falling 30 inches. Each sampler was driven a total distance of 18 inches. The number of blows required to drive the sampler 12 inches is recorded on the exploration logs, unless otherwise noted. Representative disturbed samples of soil were collected from the drill cuttings. Sampling methods and intervals are shown on the exploration logs.

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

SOIL CLASSIFICATION

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

LABORATORY TESTING

CLASSIFICATION

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

MOISTURE CONTENT

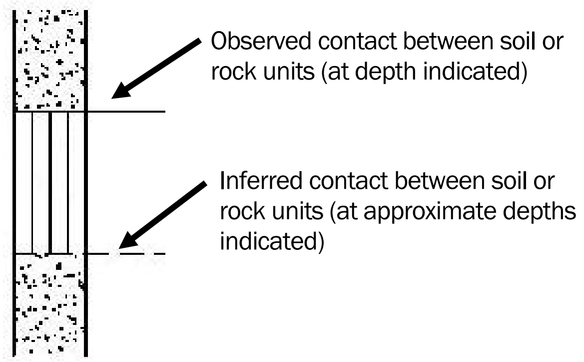
The natural moisture content of select soil samples was determined in general accordance with ASTM D2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this attachment.

PARTICLE-SIZE ANALYSIS

Particle-size analysis was performed on select soil samples 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 attachment.

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

Graphic Log of Soil and Rock Types



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

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

GEOTECHNICAL TESTING EXPLANATIONS

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

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

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 %

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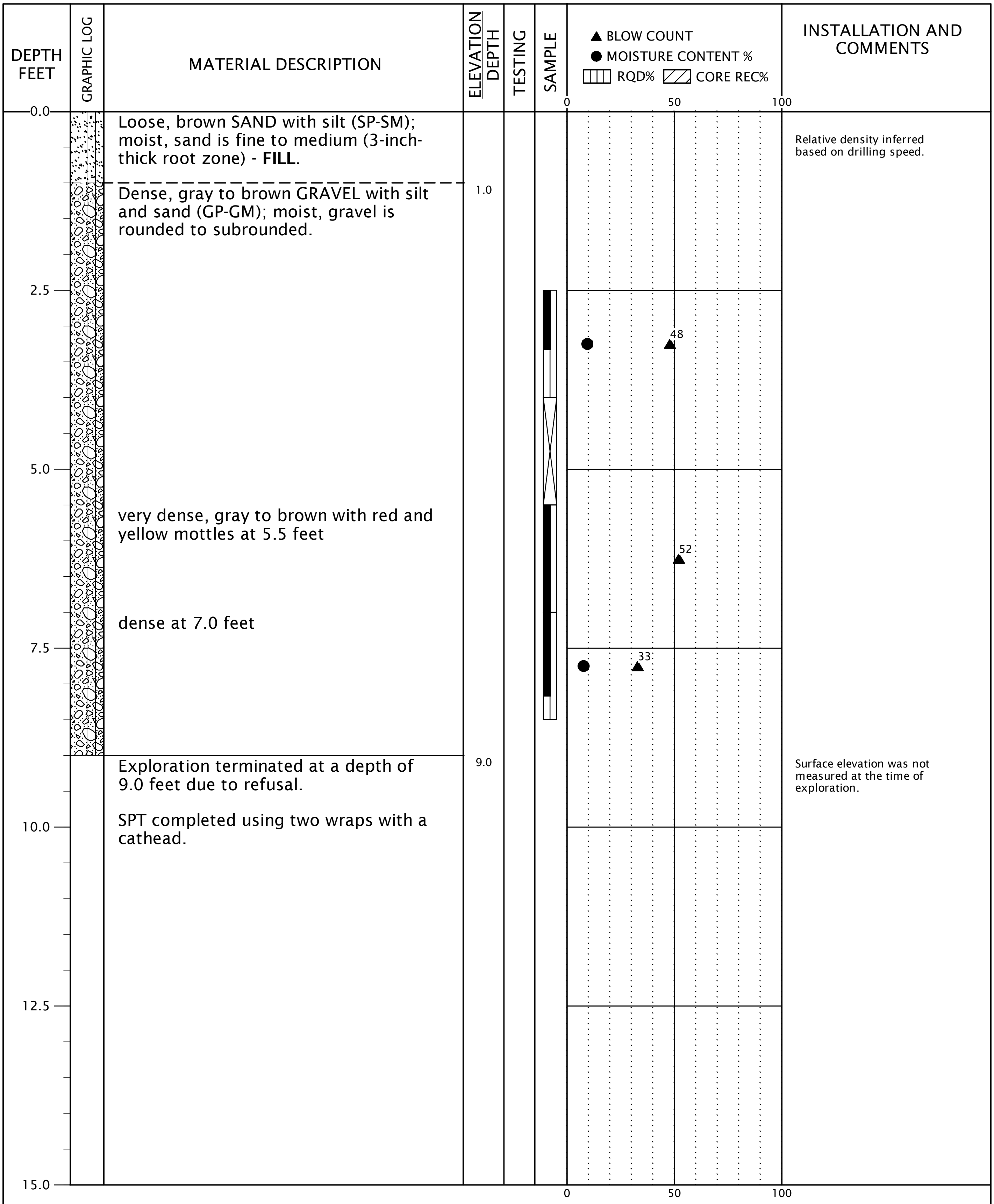
SOIL CLASSIFICATION SYSTEM

TABLE A-2



MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
Loose, brown SAND with silt (SP-SM); moist, sand is fine to medium (3-inch-thick root zone) - **FILL**.			Relative density inferred based on drilling speed.
Dense, gray to brown GRAVEL with silt and sand (GP-GM); moist, gravel is rounded to subrounded.	1.0		
very dense, gray to brown with red and yellow mottles at 5.5 feet			
dense at 7.0 feet			
Exploration terminated at a depth of 9.0 feet due to refusal. SPT completed using two wraps with a cathead.	9.0		Surface elevation was not measured at the time of exploration.

DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: S. Butler
COMPLETED: 05/31/23

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

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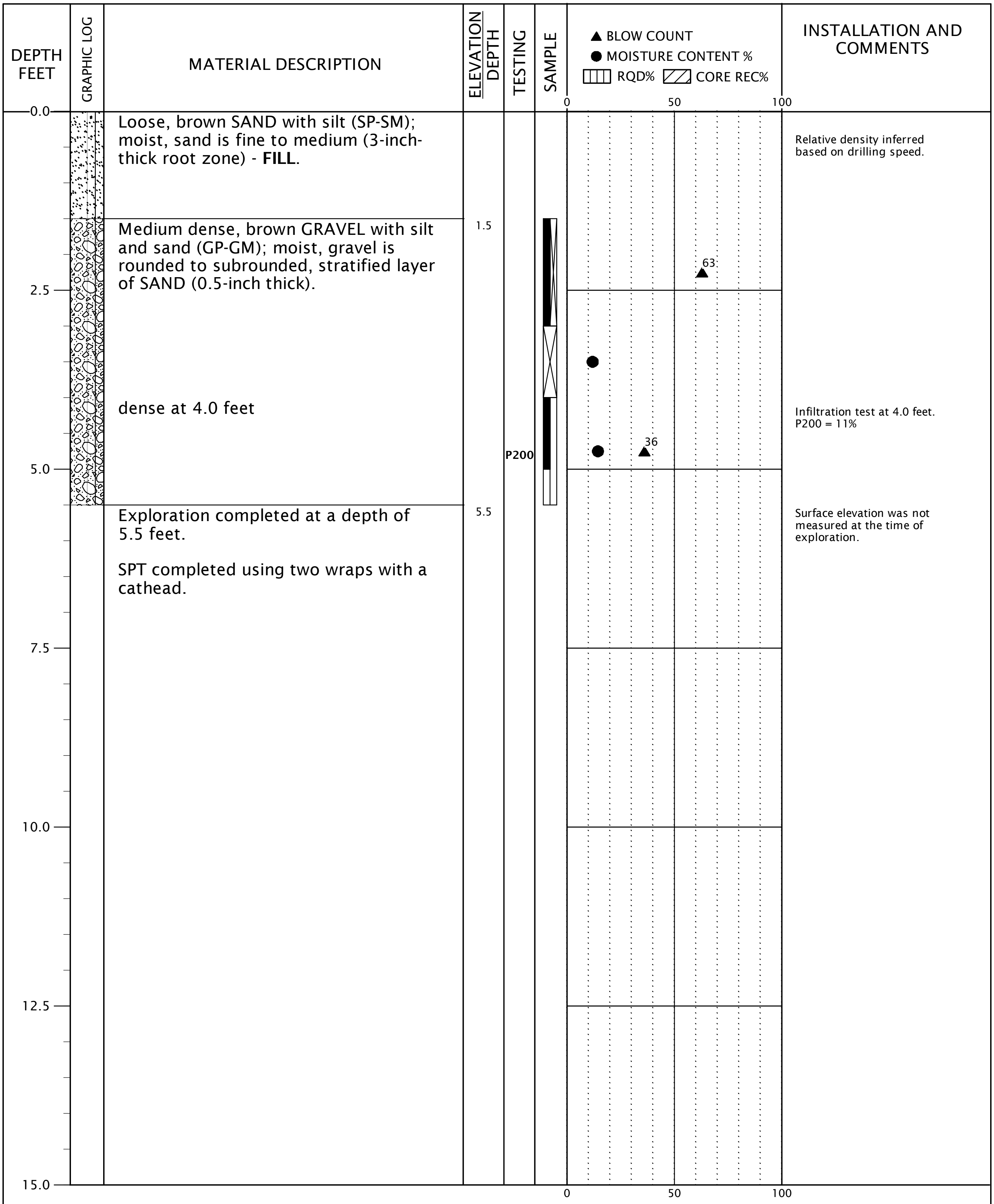
JULY 2023

BORING B-1

WILLAMETTE UNIVERSITY SOFTBALL FIELD
SALEM, OR

FIGURE A-1



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 (3-inch-thick root zone) - **FILL**.			Relative density inferred based on drilling speed.
	Medium dense, brown GRAVEL with silt and sand (GP-GM); moist, gravel is rounded to subrounded, stratified layer of SAND (0.5-inch thick).	1.5		
	dense at 4.0 feet		P200	Infiltration test at 4.0 feet. P200 = 11%
	Exploration completed at a depth of 5.5 feet. SPT completed using two wraps with a cathead.	5.5		Surface elevation was not measured at the time of exploration.

DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: S. Butler

COMPLETED: 05/31/23

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

BORING BIT DIAMETER: 4 inches

NV5

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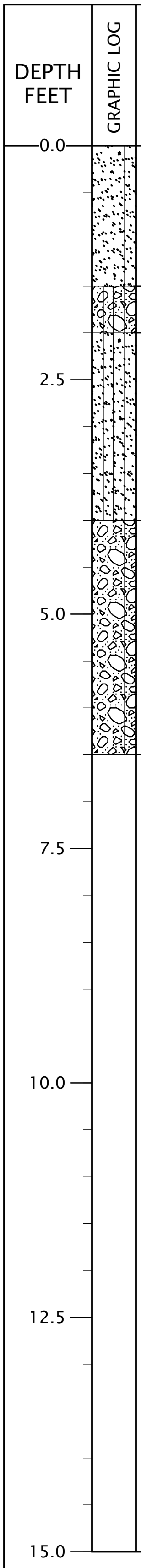
JULY 2023

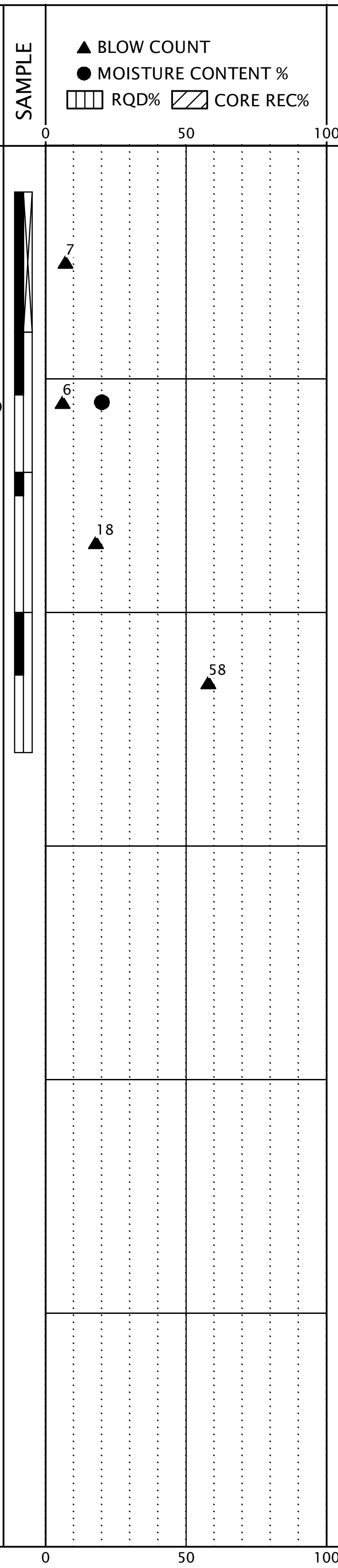
BORING B-2

WILLAMETTE UNIVERSITY SOFTBALL FIELD
SALEM, OR

FIGURE A-2



MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
Very loose, brown SAND with silt (SP-SM), trace gravel; moist, sand is fine to medium (3-inch-thick root zone) - **FILL**.			
Loose, brown, silty GRAVEL with sand (GM); moist - **FILL**.	1.5		
Loose, brown, silty SAND (SM), trace gravel; moist - **FILL**. medium dense at 3.5 feet	2.0	P200	Infiltration test at 2.0 feet. P200 = 17%
Medium dense, brown to gray with yellow mottled GRAVEL with silt and sand (GP-GM); moist, gravel is rounded to subrounded. very dense at 5.0 feet	4.0		
Exploration completed at a depth of 6.5 feet. SPT completed using two wraps with a cathead.	6.5		Surface elevation was not measured at the time of exploration.



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: S. Butler
COMPLETED: 05/31/23

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

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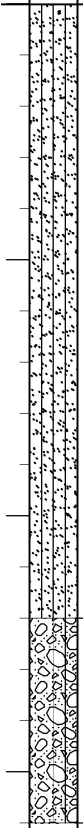
JULY 2023

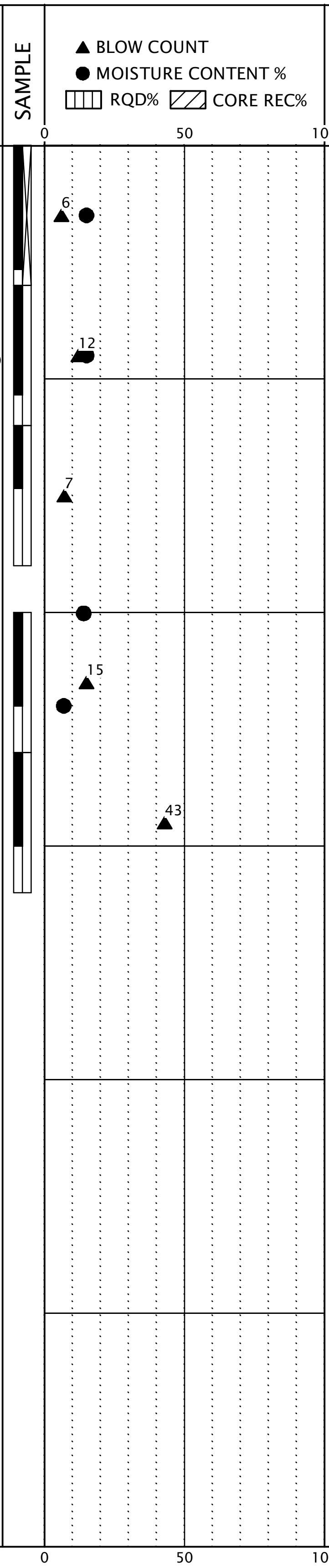
BORING B-3

WILLAMETTE UNIVERSITY SOFTBALL FIELD
SALEM, OR

FIGURE A-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Very loose, brown, silty SAND (SM); moist (3-inch-thick root zone) - **FILL**.			
	medium dense at 1.5 feet			Infiltration test at 1.5 feet. P200 = 15%
	trace gravel; stratified bed of black SILT (0.5 inch thick) at 2.0 feet		P200	
2.5	loose at 3.0 feet			
5.0	medium dense at 5.0 feet			
	Dense, gray to brown with yellow mottled GRAVEL with silt and sand (GP-GM); moist.	6.0		
7.5	Exploration completed at a depth of 8.0 feet. SPT completed using two wraps with a cathead.	8.0		Surface elevation was not measured at the time of exploration.
10.0				
12.5				
15.0				



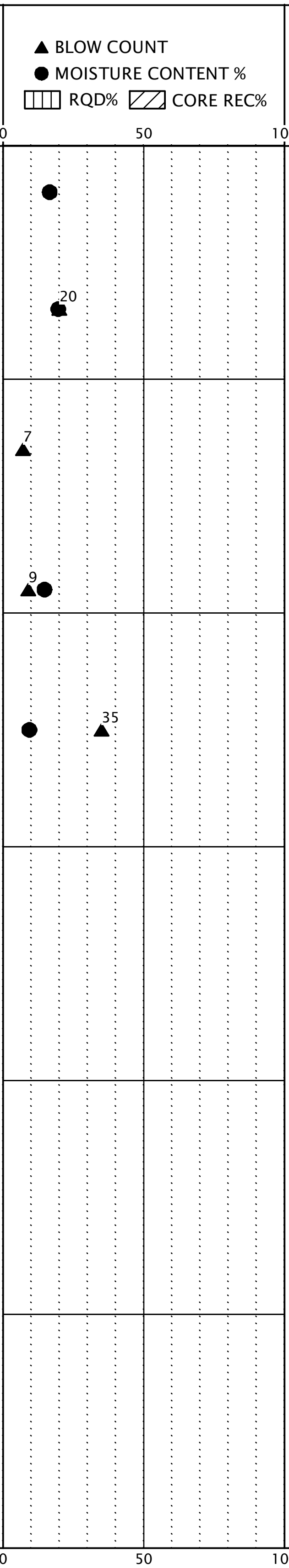


DRILLED BY: Dan J. Fischer Excavating, Inc.   LOGGED BY: S. Butler   COMPLETED: 05/31/23

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

NV5	WILLUNIV-16-01	BORING B-4	
	JULY 2023	WILLAMETTE UNIVERSITY SOFTBALL FIELD SALEM, OR	FIGURE A-4

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	INSTALLATION AND COMMENTS
0.0		Medium dense, brown, silty SAND (SM); moist (3-inch-thick root zone) - **FILL**.		P200		Infiltration test at 1.0 foot. P200 = 15%
2.5		Loose, dark brown, silty GRAVEL with sand (GM); moist - **FILL**.	2.2			
5.0		Soft, brown, sandy SILT (ML); moist.	5.0			
		Dense, gray-brown GRAVEL with silt and sand (GP-GM); moist, gravel is rounded to subrounded.	5.5			
7.5		Exploration completed at a depth of 7.0 feet. SPT completed using two wraps with a cathead.	7.0			Surface elevation was not measured at the time of exploration.
10.0						
12.5						
15.0						



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: S. Butler
COMPLETED: 05/31/23

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

NV5

WILLUNIV-16-01

JULY 2023

BORING B-5

WILLAMETTE UNIVERSITY SOFTBALL FIELD
SALEM, OR

FIGURE A-5

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		9							
B-1	7.0		8							
B-2	3.0		12							
B-2	4.0		14				11			
B-3	2.0		20				17			
B-4	0.0		15							
B-4	1.5		15				15			
B-4	5.0		14							
B-4	6.0		7							
B-5	0.0		17							
B-5	1.0		20				15			
B-5	4.5		15							
B-5	5.5		9							

WILLUNIV-16-01

JULY 2023

SUMMARY OF LABORATORY DATA

WILLAMETTE UNIVERSITY SOFTBALL FIELD
SALEM, OR

FIGURE A-6