REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Central City Apartments
420 Center Street NE
Salem, Oregon

For
Deacon Development, LLC
July 19, 2021

Project: DDG-20-01

July 19, 2021

Deacon Development, LLC
901 NE Glisan Street, Suite 100
Portland, OR 97232

Attention: Steve Deacon

Report of Geotechnical Engineering Services
Central City Apartments
420 Center Street NE
Salem, Oregon
Project: DDG-20-01

NV5 is pleased to present this report of geotechnical engineering services for the proposed Central City Apartments development located at 420 Center Street NE in Salem, Oregon. Our services were provided in general conformance with our proposal dated June 3, 2021.

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

Sincerely,

NV5

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

cc: Stephanie Gittings, Deacon Development, LLC (via email only)

NAK:SMD:kt
Attachments
One copy submitted (via email only)
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EXECUTIVE SUMMARY

- The proposed building can be supported on conventional spread footings bearing on firm native soil or structural fill underlain by firm native soil.
- While not directly observed in explorations at the site, undocumented fill from existing site development could be present at the site. Undocumented fill will not be suitable to support the proposed structure, and foundations or structural fill below foundations will need to extend through any fill and bear on firm native soil.
- The planned development will require demolition of structures. Demolition should include complete removal of the floor slabs and buried foundation elements to allow for evaluating subgrades. After evaluation, the excavations should be backfilled with compacted structural fill.
- Oversize materials will require removal from the on-site soil for use as structural fill. The moisture content of the on-site soil generally varied from 3 to 13 percent at the time of our explorations (June 2021). Moisture conditioning (drying or wetting) will also be required to use the material as structural fill, which given the site constraints may not be feasible during wet weather conditions.
- The site soils have a high fines content and may be disturbed when the moisture content is above optimum. The subgrade should be protected from construction traffic. A granular working blanket consisting of imported granular material may be required to support construction activities. The contractor should select the thickness of the working blanket, as they are in control of the type and frequency of construction traffic.
- Cobbles and potentially boulders are present at the site. Excavations may be difficult, if not impossible, with conventional equipment. Excavations may require removal of cobbles and potentially boulders. The gravel and sand with cobbles and potentially boulders are prone to raveling and caving. If difficult excavations are encountered, trenches may be wider than anticipated, increasing the amount of backfill material needed.
- Infiltration rates were measured at 35 in/hr. Infiltration testing results and recommendations are discussed in the “Infiltration System” section.

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ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ACP	asphalt concrete pavement
ASTM	American Society for Testing and Materials
BGS	below ground surface
ESAL	equivalent single-axle load
g	gravitational acceleration (32.2 feet/second^2)
GPR	ground penetrating radar
H:V	horizontal to vertical
in/hr	inches per hour
MCE	maximum considered earthquake
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Standard Specifications for Construction (2021)
PCC	portland cement concrete
pcf	pounds per cubic foot
PG	performance grade
psf	pounds per square foot
psi	pounds per square inch
SPT	standard penetration test

1.0 INTRODUCTION

NV5 is pleased to submit this report of geotechnical engineering services for the proposed Central City Apartments development located at 420 Center Street NE in Salem, Oregon. The site location is shown relative to surrounding features on Figure 1. Existing conditions and the proposed site layout (overlay) are shown on Figure 2. Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

2.0 PROJECT UNDERSTANDING

The site includes an approximately 0.7-acre parcel located on the northern portion of Block 23. The site is developed with a retail building (which occupies the entire parcel), which will be demolished for the proposed development.

The proposed project includes constructing one apartment building. The building will include four levels of multi-family apartment units and a parking lot. We understand the building will be a wood-framed structure with slab-on-grade floor. Plans do not include a basement or off-site improvements. We understand stormwater infiltration systems will be required for on-site disposal of stormwater. Foundation loads were not available at the time this report was prepared; however, we have assumed the maximum column loads will be less than 350 kips and maximum wall loads less than 5 kips per lineal foot. We estimate the distributed slab live load will be less than 100 psf. The site is flat and we anticipate mass cuts and fills will be less than a few feet each.

3.0 PURPOSE AND SCOPE

The purpose of our services was to provide geotechnical engineering recommendations for use in design and construction of the proposed development. We completed the following specific scope of services:

- Reviewed readily available, published geologic data and our in-house files for existing information on subsurface conditions in the site vicinity.
- Conducted a site reconnaissance to evaluate access into the existing building and observe proposed boring locations.
- Coordinated and managed the field explorations, including private and public utility locates and scheduling subcontractors and NV5 staff.
- Provided a geophysical site survey of the boring locations using GPR.
- Drilled five borings to depths between 4.5 and 12.5 feet BGS using hollow-stem auger or mud rotary drilling methods.
- Collected geotechnical soil samples from the explorations for laboratory testing and maintained a log of encountered soil and groundwater conditions in the explorations.
- Performed two infiltration tests in general accordance with City of Salem requirements.
- Conducted a laboratory testing program, including the following tests:
    - Thirteen moisture content determinations in general accordance with ASTM D2216
    - Six particle-size analyses in general accordance with ASTM C117

- Prepared this geotechnical engineering report that presents our findings, conclusions, and recommendations, including the following:
  - Soil and groundwater conditions at the site
  - Recommendations for site preparation and grading, including temporary and permanent slopes, fill placement criteria, suitability of on-site soil for fill, and subgrade preparation
  - Recommendations for wet weather construction
  - Discussion of seismic hazards at the site
  - Recommendations for foundation support for the proposed building
  - Recommendations for use in design of conventional retaining and basement walls, including backfill and drainage requirements and lateral earth pressures
  - Recommendations for AC pavement design sections and pavement subgrade preparation
  - General recommendations for dewatering during construction and subsurface drainage
  - Results of field infiltration testing

4.0 SITE CONDITIONS

4.1 SURFACE CONDITIONS

The site includes an approximately 0.7-acre parcel located on the northern portion of Block 23. The site is developed with a retail building (which occupies the entire parcel), which will be demolished for the proposed development. The site is bordered by Center Street NE to the north, Liberty Street NE to the west, and neighboring building to the south and east. The site is flat.

4.2 SUBSURFACE CONDITIONS

Subsurface conditions were explored by drilling five borings (B-1 through B-5) to depths between 4.5 and 12.5 feet BGS. The locations of the explorations are shown on Figure 2. The exploration logs and laboratory test results are presented in the Appendix.

Subsurface conditions encountered in our explorations consist of gravel with varying amounts of silt, sand, and cobbles (and potentially boulders) to the depths explored. Borings B-3 through B-5 encountered refusal to drilling on very dense gravel. The following sections provide a detailed description of the geologic unit encountered.

4.2.1 Slab/Pavement Section

Our explorations encountered 2.9 to 4.4 inches of PCC. Aggregate base was not observed.

4.2.2 Gravel

Our borings encountered loose to very dense (generally medium dense to very dense) gravel with varying amounts of silt, sand, and cobbles below the pavement. It is possible that boulders are present in this geologic unit. Borings B-3 through B-5 were terminated due to refusal on very dense gravel at depths of between 9 and 12.5 feet BGS.

Laboratory testing indicates the moisture content of the gravel at the time of our explorations ranged between 3 and 13 percent. Fines content analysis of select gravel samples indicates a fines content ranging between 5 and 21 percent.

4.2.3 Groundwater

Groundwater was not observed in the explorations completed at the site to the maximum depth explored of 12.5 feet BGS. Reportedly, groundwater was encountered (by others) in the site vicinity at depths between 12 and 18 feet BGS. Water well logs in the site vicinity indicate groundwater is present at a depth of approximately 15 feet BGS. We anticipate that perched groundwater may be present above the permanent groundwater during certain times of the year. The depth to groundwater may fluctuate in response to seasonal changes, changes in surface topography, water level fluctuation in nearby Willamette River, and other factors not observed in this study.

4.3 INFILTRATION TESTING

We understand stormwater infiltration systems are proposed for the development. The locations and configurations were conceptual at the time of this report. We conducted one infiltration test in borings B-1 and B-2 at depths of 3 and 2.5 feet BGS, respectively. The infiltration testing procedures are described in the Appendix, and the results of the infiltration testing are described in the “Infiltration System” section.

4.4 GEOLOGIC HAZARDS

4.4.1 Liquefaction

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.

Based on the subsurface conditions encountered in the explorations, it is our opinion that liquefaction is not a hazard at the site.

4.4.2 Lateral Spreading

Lateral spreading is a liquefaction-related seismic hazard. Areas subject to lateral spreading are typically gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face, such as a riverbank. Due to the fact that there are no open faces in the site vicinity and liquefaction is not a hazard at the site, lateral spreading is not considered a site hazard.

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on our understanding of the proposed development and the results of our explorations, laboratory testing, and analyses, it is our opinion the proposed development can be constructed at the site. The primary geotechnical considerations for the project are summarized in the “Executive Summary” section. Our specific recommendations are provided in the following sections.

6.0 DESIGN

6.1 GENERAL

The following sections provide our design recommendations for the project. All site preparation and structural fill should be prepared as recommended in the "Construction" section.

6.2 SHALLOW FOUNDATIONS

6.2.1 General

Based on the estimated foundation loads as previously stated, and assuming the site is prepared as recommended in the "Construction" section, it is our opinion the proposed building can be supported on conventional spread footings founded on undisturbed native soil or on structural fill overlying undisturbed native soil. Foundation elements should not be supported on topsoil or undocumented fill material. If present, these materials should be removed and replaced with structural fill.

6.2.2 Bearing Capacity

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 native dense gravel subgrade prepared as recommended above should be sized based on an allowable bearing pressure of 4,000 psf. This is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and may be increased by one-third for short-term loads such as those resulting from wind or seismic forces.

6.2.3 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 native soil and structural fill is 350 pcf modeled as an equivalent fluid pressure. Adjacent floor slabs, pavement, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. In addition, in order to rely on passive resistance, a minimum of 10 feet of horizontal clearance must exist between the face of the footings and any adjacent down slopes. For footings that bear on dense gravel, a coefficient of friction equal to 0.4 may be used when calculating resistance to sliding.

6.2.4 Settlement

Total foundation settlement should be less than 1 inch with differential settlement between similarly loaded foundations of less than 0.5 inch under dead and long-term live loads.

6.2.5 Subgrade Observation

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

6.2.6 Construction Considerations

If footing excavations are conducted during wet weather conditions, we recommend that a minimum of 3 inches of granular material be placed and compacted until well keyed at the base of the excavations. The granular material reduces water softening of subgrade soil, reduces subgrade disturbance during placement of forms and reinforcement, and provides clean conditions for the reinforcing steel. If subgrade surfaces are irregular and disturbed due to the removal of oversize cobbles and boulders, we also recommend placing and compacting a leveling course of imported granular material at the surface.

6.3 SEISMIC DESIGN CONSIDERATIONS

6.3.1 Seismic Design Parameters

Based on the results of our subsurface explorations and review of water well logs in the site vicinity, the seismic design parameters for Site Class C presented in Table 1 can be used for design.

Table 1. Seismic Design Parameters

Seismic Design Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
MCE Spectral Acceleration	S_s = 0.827 g	S_1 = 0.415 g
Site Class	C	
Site Coefficient	F_a = 1.200	F_v = 1.500
Adjusted Spectral Acceleration	S_{MS} = 0.992 g	S_{M1} = 0.623 g
Design Spectral Response Acceleration Parameters	S_{DS} = 0.662 g	S_{D1} = 0.415 g

6.4 FLOOR SLABS

Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Load-bearing concrete slabs may be designed assuming a modulus of subgrade reaction, k, of 250 psi per inch for design of slabs on native gravel. To aid as a capillary break, we recommend a 6-inch-thick layer of floor slab base rock be placed and compacted over the prepared subgrade. The floor slab base rock should meet the requirements in the "Structural Fill" section and be compacted to at least 95 percent of ASTM D1557.

The near-surface native soil has a high fines content and will tend to maintain a high moisture content. In areas where moisture-sensitive floor slab and flooring will be installed, installation of a vapor barrier is warranted in order to reduce the potential for moisture transmission through and efflorescence growth on the slab and flooring. In addition, flooring manufacturers often require vapor barriers to protect flooring and flooring adhesives and will warrant their product only if a vapor barrier is installed according to their recommendations. Selection and design of an appropriate vapor barrier should be a collaborative effort with members of the design team.

6.5 RETAINING STRUCTURES

6.5.1 Assumptions

Our retaining wall design recommendations are based on the following assumptions: (1) the walls consist of conventional, cantilevered retaining walls, (2) the walls are less than 8 feet in height, (3) the backfill is drained and consists of imported granular material, and (4) the backfill has a slope flatter than 4H:1V. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

6.5.2 Lateral Earth Pressures

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

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

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

6.5.3 Wall Drainage and Backfill

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

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

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

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

6.6 DRAINAGE

6.6.1 Temporary

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 pads and subgrade free of ponding water.

6.6.2 Surface

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

6.6.3 Subsurface

Assuming the site grades around the building will be sloped as discussed previously, it is our opinion that perimeter footing drains will not be required around the proposed building. However, the use of these drains should be considered in areas where landscaping planters are placed proximate to the foundations or where surface grades cannot be completed as outlined above.

If installed, the foundation drains should be constructed at a minimum slope of approximately ½ percent and pumped or drained by gravity to a suitable discharge. The perforated drainpipe should not be tied to a stormwater drainage system without backflow provisions. The foundation drains should consist of 4-inch-diameter, perforated drainpipe embedded in a minimum 2-foot-wide zone of crushed drain rock that extends to the ground surface. The invert elevation of the drainpipe should be installed at least 18 inches below the elevation of the floor slab.

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

6.7 INFILTRATION SYSTEM

We understand stormwater infiltration systems will be required for the proposed development. The locations and configurations have not been established at the time of this report. We understand these systems will be located in the parking lot at the southeastern portion of the site. The infiltration tests were performed to evaluate the infiltration potential for the proposed infiltration systems. The results of our field infiltration testing are presented in Table 2.

Table 2. Infiltration Testing Summary

Location	Depth (feet BGS)	Observed Infiltration Rate[1] (in/hr)	Fines Content[2] (percent)	Soil Type at Test Depth
B-1	3	35	6	Gravel with silt, sand, and cobbles
B-2	2.5	35	5	Gravel with silt, sand, and cobbles

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

We recommend all infiltration systems be at least 2.5 feet deep and installed in clean gravel with low fines content. Based on groundwater information in the site vicinity, we recommend a maximum depth of 7 feet. For infiltration systems in the gravel, we recommend an unfactored field infiltration rate of 35 in/hr based on the soil classification and the fines contents of 6 and 5 percent in borings B-1 and B-2, respectively, which indicate similar soil conditions.

The recommended infiltration rates are measured rates and are unfactored. Correction factors should be applied to the recommended infiltration rates by the civil engineer during design to account for the degree of long-term maintenance and influent/pre-treatment control, as well as the potential for long-term clogging due to siltation and buildup of organic material, depending on the proposed length, location, and type of infiltration facility. We recommend a minimum factor of safety of at least 2 be applied to the recommended unfactored rates.

The actual depths and estimated infiltration rates can vary significantly from the values presented above. We recommend that the design infiltration values for the stormwater systems be confirmed by field testing completed during installation of the systems. The results of this field testing might necessitate that the stormwater system be enlarged to achieve the design infiltration rate.

We recommend NV5 be contacted to review the configuration of the infiltration systems to evaluate that the design will not significantly affect bearing capacity of neighboring footings (if any).

6.8 PAVEMENT

New AC pavement may be constructed for drive aisles and car parking at the proposed project. The pavement subgrades should be prepared in accordance with the “Site Preparation” section.

The design parameters provided below assume the pavement is underlain by native soil or structural fill underlain by native soil.

Specific traffic information was not provided to us at the time of this report. Our pavement recommendations are based on the following assumptions:

- A resilient modulus of 20,000 psi was estimated for the aggregate base.
- Initial and terminal serviceability indices of 4.2 and 2.0, 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.
- No growth.
- A resilient modulus of 4,500 psi for site subgrades assuming the surface 12 inches of subgrade is scarified and compacted to at least 92 percent of the maximum dry density, as defined by ASTM D1557 (modified).
- Design ESALs of 45,000 and 10,000 were used for the access roadways and parking areas, respectively.
- The pavement section of access roadways associated with the fire access road will support an imposed fire apparatus load of 75,000 pounds on an infrequent basis.

If any of these assumptions are incorrect, our office should be contacted with the appropriate information so that the pavement designs can be revised.

Based on the traffic assumptions, we recommend the AC pavement sections presented in Table 3.

Table 3. Recommended Standard Pavement Sections

Pavement Use	ESALs	AC Thickness (inches)	Aggregate Base Thickness (inches)
Drive Aisles	45,000	3	10
Automobile Parking Stalls	10,000	2.5	8

All thicknesses are intended to be the minimum acceptable. Design of the recommended pavement sections 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 aggregate is not designed to support construction traffic. Increased aggregate thicknesses will likely be required to support construction traffic as discussed in the “Construction Considerations” section. The AC and aggregate base should meet the requirements outlined in the “Materials” section.

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

7.0 CONSTRUCTION

7.1 SITE PREPARATION

7.1.1 Demolition

Demolition includes complete removal of the existing buildings, retaining walls, pavement, concrete curbs, abandoned utilities, and any subsurface elements within 5 feet of areas to receive new pavement, buildings, retaining walls, or engineered fills. Demolished material should be transported off site for disposal. In general, this material will not be suitable for re-use as engineered fill. However, concrete material may be recycled in accordance with the requirements set forth by the project jurisdiction and the recommendations provided in the "Structural Fill" section.

Excavations remaining from removing basements, foundations, utilities, and other subsurface elements should be backfilled with structural fill where these are below planned site grades. The base of the excavations should be excavated to expose firm subgrade before filling. The sides of the excavations should be cut into firm material and sloped a minimum of 1½H:1V. Utility lines abandoned under new structural components should be completely removed and backfilled with structural fill or grouted full if left in place. Soft or disturbed soil encountered during demolition should be removed and replaced with structural fill.

Considerable subgrade damage can occur during demolition activities, and we recommend that the subgrade protection measures discussed in the "Construction Considerations" section be implemented.

7.1.2 Subgrade Evaluation

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

7.2 CONSTRUCTION CONSIDERATIONS

Trafficability on exposed silty, sandy gravel subgrade may be difficult during or after extended wet periods or when the moisture content of the surface soil is more than a few percentage points above optimum. Soil that has been disturbed during site preparation activities or soft or loose zones should be removed and replaced with compacted structural fill.

If construction occurs during or extends into the wet season, or if the moisture content of the surficial soil is more than a few percentage points above optimum, site demolition and cutting commonly need to be accomplished using track-mounted equipment to prevent damage to the subgrade. Likewise, the use of granular haul roads and staging areas will be necessary for support of construction traffic during the rainy season or when the moisture content of the surficial soil is more than a few percentage points above optimum. The base rock thickness for pavement areas is intended to support post-construction design traffic loads, not for support of construction traffic. If construction is planned for periods when the subgrade soil is wet, staging and haul roads with increased thicknesses of base rock will be required. The amount of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor's sequencing of a project and type/frequency of construction equipment. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul roads areas. Stabilization material may be used as a substitute, provided the top 4 inches of material consists of imported granular material. The actual thickness will depend on the contractor's means and methods and, accordingly, should be the contractor's responsibility. In addition, a geotextile fabric should be considered to assist as a barrier between the subgrade and imported granular material in areas of repeated construction traffic. The imported granular material, stabilization material, and geotextile fabric should meet the specifications in the "Materials" section.

7.3 PERMANENT SLOPES

Permanent cut and fill slopes should not exceed 2H:1V for a maximum height of 10 feet. Taller slopes or steeper slope gradients should be evaluated on a case-by-case basis. Access roads 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.

7.4 EXCAVATION

7.4.1 Excavation and Shoring

As mentioned previously, cobbles (and potentially boulders) might be encountered in the site soil. The dense gravel with cobbles (and potentially boulders) encountered below the pavement in our explorations will result in difficult excavation with conventional equipment. We note that borings B-3 through B-5 were terminated in very dense gravel due to refusal to drilling. Utility trenches may result in slowed excavation and larger backfill volumes due to the presence of cobbles (and potentially boulders) and related caving.

If buried construction debris from prior on-site structures is encountered beneath the ground surface, these materials will result in difficult trench excavations and may require additional effort or special equipment. If difficult excavations are encountered, trenches may also be wider than anticipated, increasing the amount of backfill material required.

Temporary excavation sidewalls should stand vertical to a depth of approximately 4 feet, provided groundwater seepage is not observed in the sidewalls and surcharge loads will not be present within H feet, where H is the depth of the trench. Open excavation techniques may be used to excavate trenches with depths between 4 and 8 feet, provided the walls of the excavation are cut at a slope of 1½H:1V and groundwater seepage and surcharge loads are not present. At this inclination, slopes with loose gravel and sand may ravel and require some ongoing repair. Excavations should be flattened to 2H:1V if excessive sloughing or raveling occurs. In lieu of large and open cuts, approved temporary shoring may be used for excavation support. Use of approved temporary shoring is recommended where the slopes cannot be cut back, within the influence area of structural elements, and for cuts below the water table. The influence area can be defined as a 1H:1V slope extending down from a 5-foot setback from the edge of a foundation element. A wide variety of shoring and dewatering systems are available. Consequently, we recommend the contractor be responsible for selecting the appropriate shoring and dewatering systems.

If box shoring is used, it should be understood that box shoring is a safety feature used to protect workers and does not prevent caving. If excavations are left open for extended periods of time, 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 the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation.

7.4.2 Trench Dewatering

Groundwater was encountered (by others) in the site vicinity at depths between 12 and 18 feet BGS. We anticipate the groundwater level will rise during extended periods of wet weather. Dewatering will be required for excavations where groundwater is encountered. Dewatering systems are best designed by the contractor.

Sloughing, caving, and “running’ conditions are likely for excavations in gravel containing sand and silt below the groundwater table. Accordingly, positive control of groundwater may be necessary to maintain stable trench sides and base for excavations below groundwater levels. Dewatering within trenches may be sufficient for excavations that encounter perched groundwater or only extending a shallow depth below groundwater. The proposed dewatering plan should be capable of maintaining groundwater levels at least 2 feet below the base of the trench excavation (including the depth required for trench bedding and stabilization material). In addition to safety considerations, running soil, caving, or other loss of ground will increase backfill volumes and can result in damage to adjacent structures or utilities.

Flow rates for dewatering are likely to vary depending on location, soil type, and the season in which the excavation occurs. The dewatering systems should be capable of adapting to variable flows. Because of the tendency of saturated silt and sand-rich soil to “run,” we recommend that dewatering wells or well points be considered if trench excavations extend below groundwater levels. Tight-joint driven sheets in conjunction with a scaled-down dewatering program can also be an effective way to control groundwater seepage, provided the sheets are driven deep enough to control heaving conditions at the base of the excavation.

If groundwater is present at the base of utility excavations, we recommend placing 1 to 1.5 feet of stabilization material at the base of the excavation. The use of a subgrade geotextile fabric may reduce the amount of stabilization material required. The actual thickness should be based on field observations during construction.

Trench stabilization material and the subgrade geotextile fabric should meet the requirements described in the "Materials" section. Trench stabilization material should be placed in one lift and compacted until well keyed.

While we have described certain approaches to the excavation dewatering, it is the contractor's responsibility to select the dewatering methods.

7.4.3 Safety

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

7.5 MATERIALS

7.5.1 Structural Fill

7.5.1.1 General

Fill should be placed on subgrade that has been prepared in conformance with the "Site Preparation" section. A variety of 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. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill are provided below.

7.5.1.2 On-Site Soil

The on-site gravelly soil is suitable for use as structural fill, provided it is free of organic material or other unsuitable material and has particles less than 4 inches in diameter. Occasional cobbles up to 8 inches in diameter may be acceptable if they can be properly mixed into the fill matrix. Fine grading of gravelly soil may result in segregating cobbles or coarse gravel from the soil matrix, resulting in unsatisfactory (poorly graded or "bony") fill. The material may also include a silt or clay matrix that will render the material moisture sensitive and require moisture conditioning. In order to adequately compact the soil, it will be necessary to moisture condition (i.e., drying or wetting) the soil to within a few percentage points of the optimum moisture content.

Fill material should be maintained as well graded with gravel, sand, and silt material for proper compaction during fill placement and mass grading. A qualified geotechnical engineer should observe fill material prior to placement.

When used as structural fill, the on-site gravelly soil 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.

7.5.1.3 Imported Granular Material

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

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

7.5.1.4 Stabilization Material

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

7.5.1.5 Trench Backfill

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

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

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

7.5.1.6 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; should have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis); and should have at least two mechanically fractured faces. Drain rock should be compacted to a well-keyed, firm condition.

7.5.1.7 Aggregate Base Rock

Imported granular material used as base rock for building floor slabs and pavement should consist of ¾- or 1½-inch-minus material (depending on the application). In addition, the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve and have at least two mechanically fractured faces. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

7.5.1.8 Retaining Wall Select Backfill

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

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

7.5.1.9 Retaining Wall Leveling Pad

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

7.5.1.10 Existing Concrete

Concrete from the existing pavement areas and improvements can be used in general structural fill, provided particles greater than 3 inches are not present, it is thoroughly mixed and well graded so that there are no voids between the fragments, and the resulting mix is moisture conditioned for compaction. This material can be used as trench backfill if it meets the requirements for imported granular material, which would require a smaller maximum particle size. The material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

7.5.2 Geotextile Fabric

7.5.2.1 Subgrade Geotextile

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

7.5.2.2 Drainage Geotextile

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

7.5.3 AC

7.5.3.1 ACP

The AC should be Level 2, ½-inch, dense ACP according to OSSC 00744 (Asphalt Concrete Pavement) and compacted to 91 percent of the theoretical maximum density of the mix, as determined by AASHTO T 209. The minimum and maximum lift thicknesses are 2 and 3 inches, respectively, for ½-inch ACP. Asphalt binder should be performance graded and conform to PG 64-22 or better. The binder grade should be adjusted depending on the aggregate gradation and amount of recycled asphalt pavement and/or recycled asphalt shingles in the contractor's mix design submittal.

7.5.3.2 Cold Weather Paving Considerations

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

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

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

7.6 EROSION CONTROL

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

8.0 OBSERVATION OF CONSTRUCTION

Satisfactory foundation and earthwork performance depends to a large degree on quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during the subsurface 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 NV5 be retained to observe earthwork activities, including stripping, proof rolling of the subgrade and repair of soft areas, footing subgrade and granular pad preparation, infiltration system installation, final proof rolling of the pavement subgrade and base rock, and AC placement and compaction, and performing laboratory compaction and field moisture-density tests.

9.0 LIMITATIONS

We have prepared this report for use by Deacon Development, LLC and members of the design and construction team 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 conceptual 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 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 continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

NV5

Najib A. Kalas, P.E.
Senior Associate Engineer

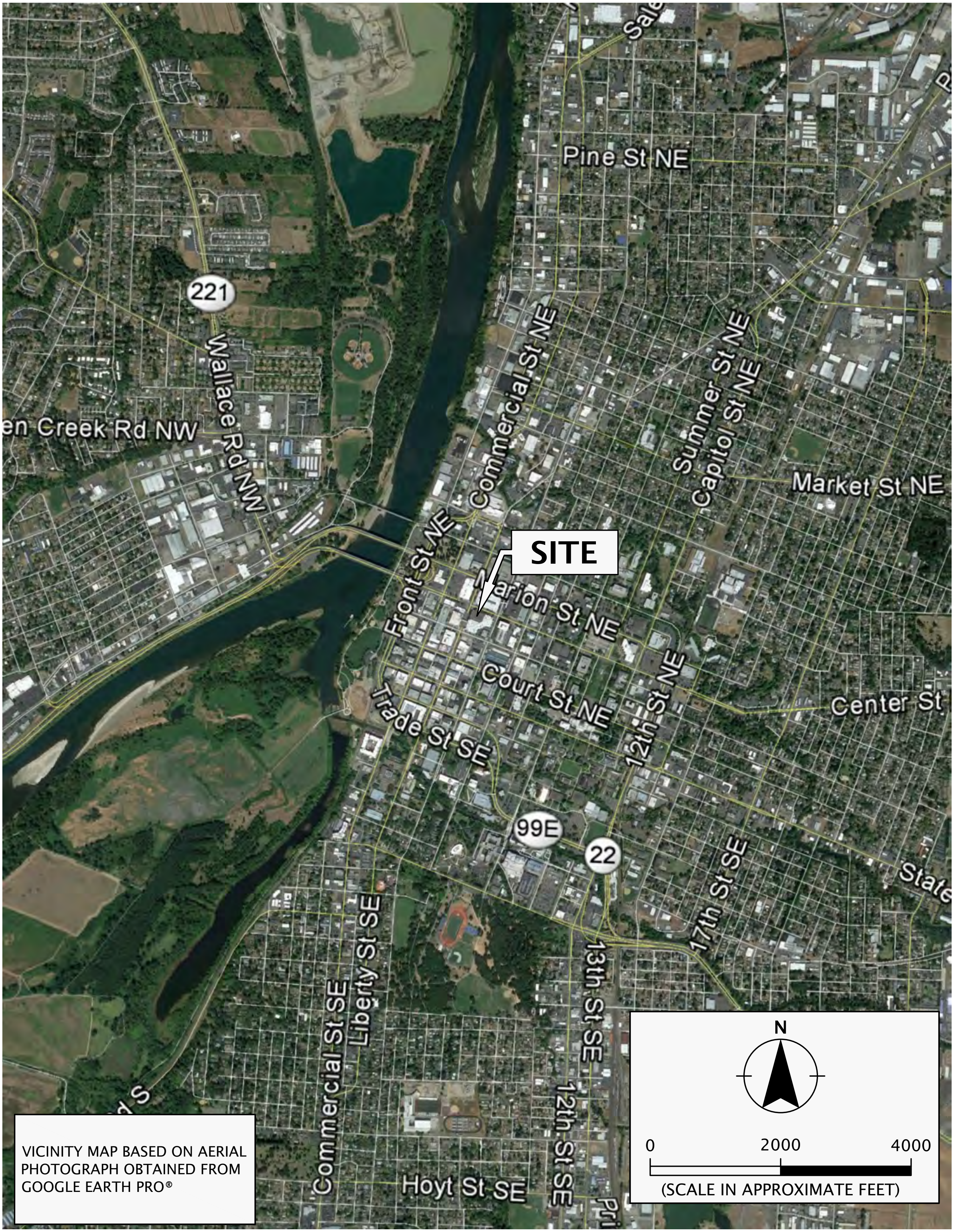
Shawn M. Dimke, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL
ENGINEER
84829PE
OREGON
November 10, 2015
NAJIB A. KALAS
EXPIRES: 06/30/23

REFERENCES

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

FIGURES



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

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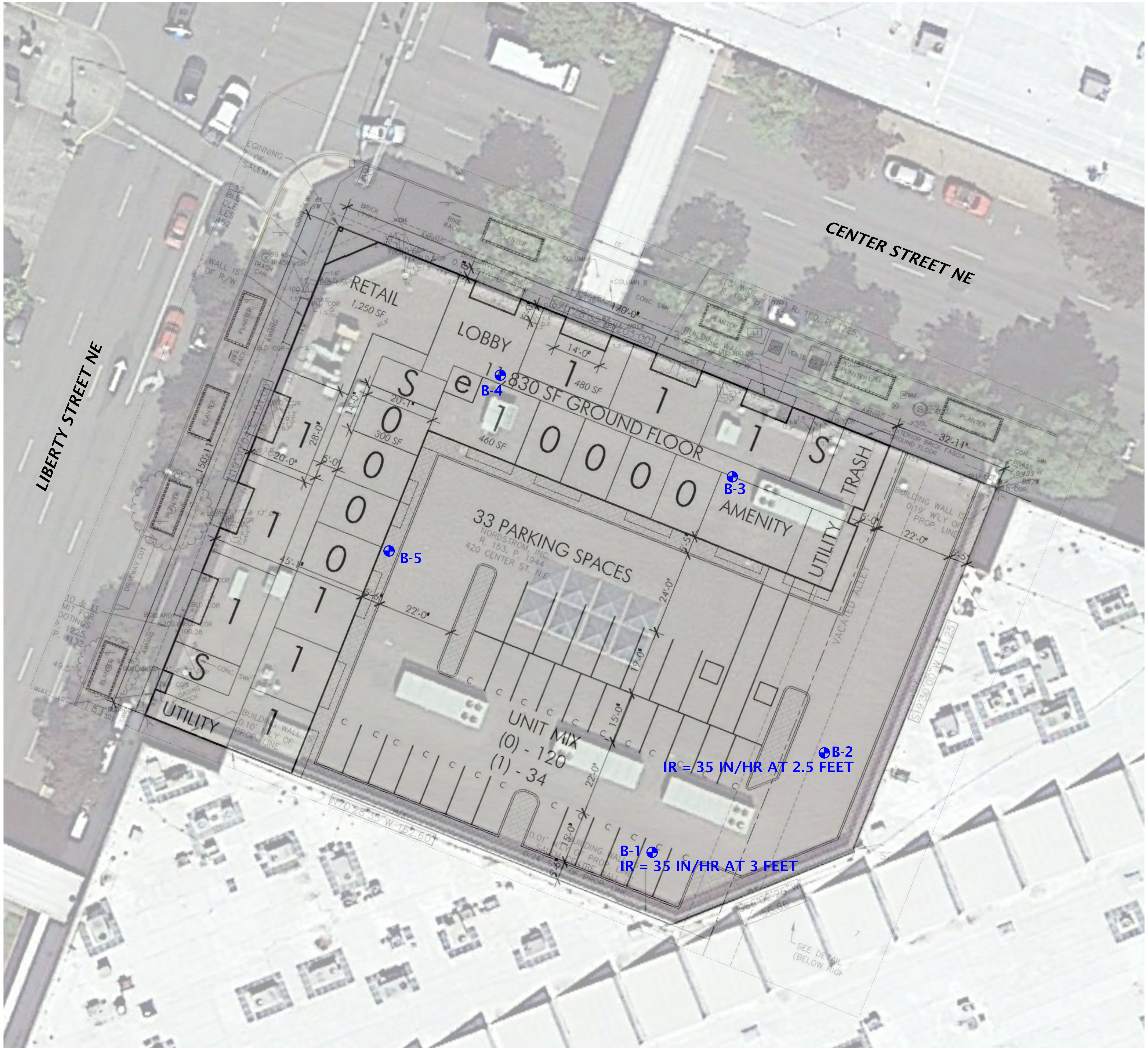
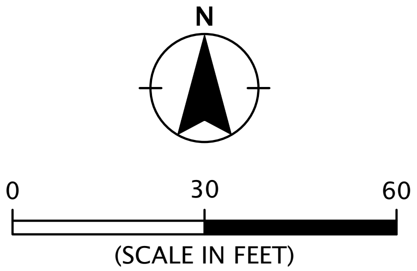
File Name: J:\A-D\DDG\DDG-20\DDG-20-01\Figures\CAD\DDG-20-01-VM01.dwg | Layout: FIGURE 1

NV5	DDG-20-01	**VICINITY MAP**	
	JULY 2021	CENTRAL CITY APARTMENTS SALEM, OR	**FIGURE 1**



LEGEND:

B-1 ⊕	BORING
IR = 35 IN/HR AT 3 FEET	UNFACTORED INFILTRATION RATE



(SCALE IN FEET)

NOTES:

1. SITE PLAN BASED ON IMAGE PROVIDED BY DEACON DEVELOPMENT, LLC ON MAY 22, 2021.
2. AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO ON JULY 2, 2021.

FIGURE 2

SITE PLAN

CENTRAL CITY APARTMENTS
SALEM, OR

DDG-20-01

JULY 2021

NV5

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APPENDIX

APPENDIX

FIELD EXPLORATIONS

GENERAL

Subsurface conditions were explored by drilling five borings (B-1 through B-5) to depths between 4.5 and 12.5 feet BGS. The borings were drilled by Western States Soil Conservation, Inc. of Hubbard, Oregon, using hollow-stem auger or mud rotary drilling techniques under the supervision of NV5. The exploration logs are presented in this appendix. The approximate exploration locations are shown on Figure 2.

The exploration locations were determined by pacing from existing site features and should be considered accurate to the degree implied by the methods used. A member of our geology staff observed the explorations.

SOIL SAMPLING

We collected representative samples of the various soils encountered during drilling for geotechnical laboratory testing. Samples were collected from the borings using 3-inch and/or 1½-inch-inside-diameter, split-spoon SPT samplers in general accordance with ASTM D1586. The samplers were driven into the soil with a 140-pound automatic trip hammer free-falling 30 inches. The sampler was driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the exploration logs, unless otherwise noted. Disturbed grab samples were collected from the soil cuttings. Sampling methods and intervals are shown on the exploration logs.

The average efficiency of the automatic SPT hammer used by Western States Soil Conservation, Inc. was 82.2 percent. The calibration testing results are presented at the end of 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.

INFILTRATION TESTING

Infiltration testing was conducted borings B-1 and B-2 at depths of 3 and 2.5 feet BGS, respectively. The infiltration rates were estimated by filling the auger with water, allowing the area to saturate, and then measuring the drop in water with time. The tests were conducted under a hydrostatic head of generally less than approximately 18 inches. Representative soil samples were collected from at or below the infiltration test locations for grain-size distribution analyses, as described in this appendix.

LABORATORY TESTING

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

MOISTURE CONTENT

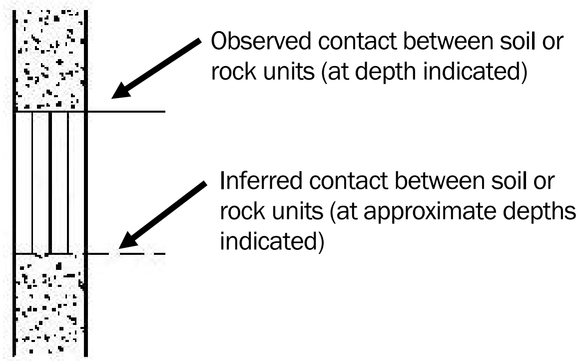
We tested the natural moisture content of select soil samples in general accordance with ASTM D2216. The test results are presented in this appendix.

PARTICLE-SIZE ANALYSIS

We completed particle-size testing on select soil samples in order to determine the distribution of soil particle sizes. The testing consisted of fines content determination completed in general accordance with ASTM C117. The test results are presented in this appendix.

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

Graphic Log of Soil and Rock Types


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

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

GEOTECHNICAL TESTING EXPLANATIONS

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

ENVIRONMENTAL TESTING EXPLANATIONS

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

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.

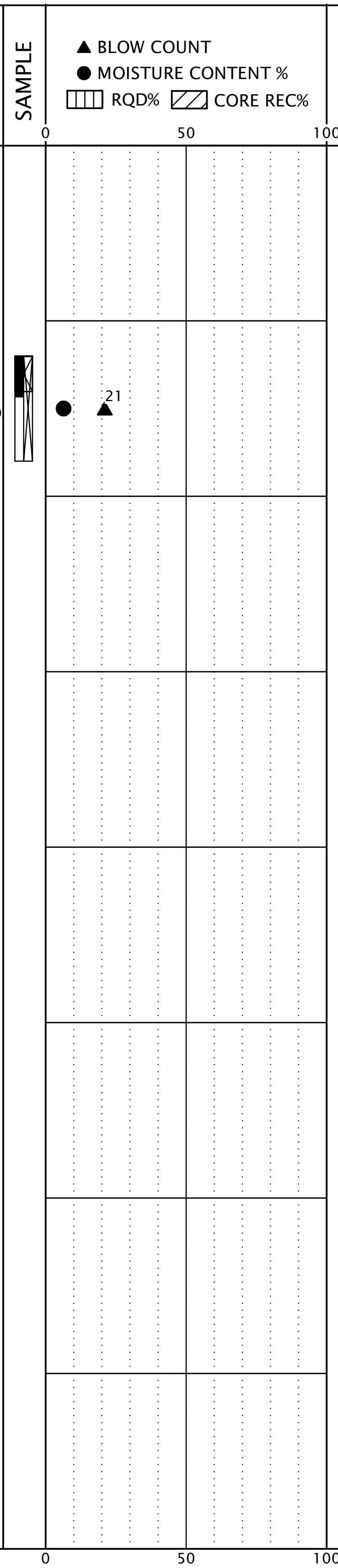
Percent	Silt and Clay In:		Percent	Sand and Gravel In:	
	Fine-Grained Soil	Coarse-Grained Soil		Fine-Grained Soil	Coarse-Grained Soil
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

NV5

SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	CONCRETE (4.25 inches).			
	Loose, brown GRAVEL with silt, sand, and cobbles (GP-GM); moist, gravel is fine to coarse and rounded to subrounded, sand is fine to coarse, cobbles are approximately 40%.	0.4		Cored through concrete then switched to hollow-stem auger.
2.5			P200	Infiltration test at 3.0 feet. P200 = 6%
	Exploration completed at a depth of 4.5 feet. Hammer efficiency factor is 82.2 percent.	4.5		Surface elevation was not measured at the time of exploration.
5.0				
7.5				
10.0				
12.5				
15.0				
17.5				
20.0				



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: L. Gose

COMPLETED: 06/11/21

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

BORING BIT DIAMETER: 10 inches and 4 1/4 inches

NV5

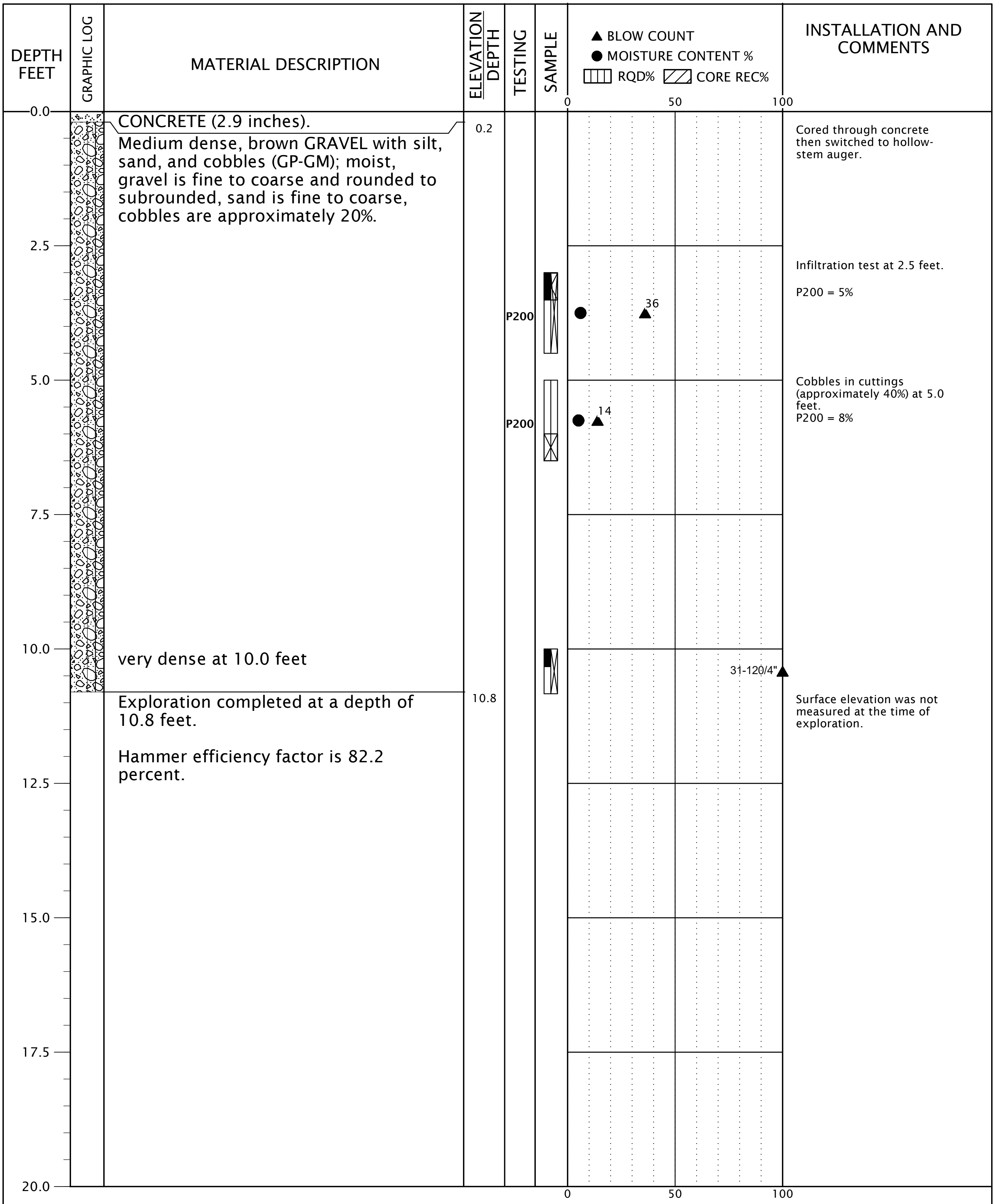
DDG-20-01

JULY 2021

BORING B-1

CENTRAL CITY APARTMENTS
SALEM, OR

FIGURE A-1



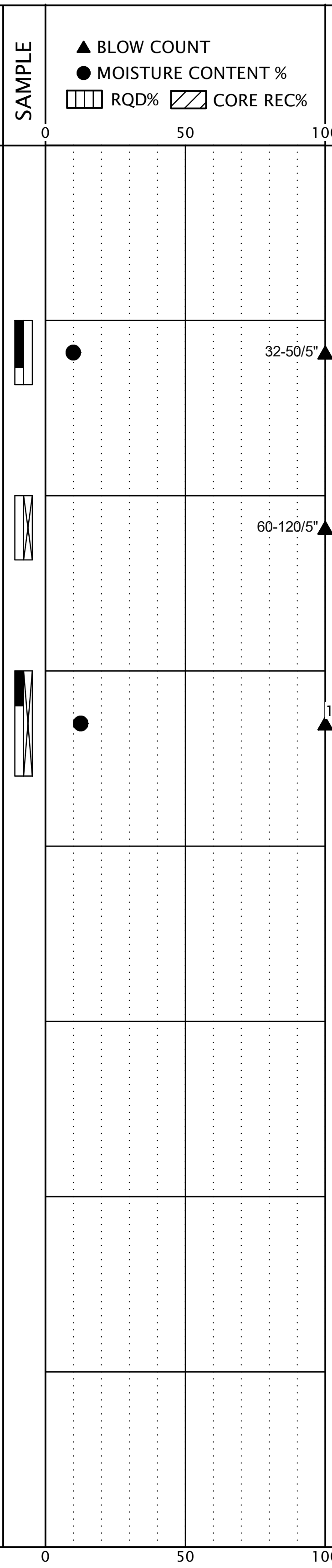
DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	CONCRETE (2.9 inches).	0.2			Cored through concrete then switched to hollow-stem auger.
	Medium dense, brown GRAVEL with silt, sand, and cobbles (GP-GM); moist, gravel is fine to coarse and rounded to subrounded, sand is fine to coarse, cobbles are approximately 20%.				
2.5					Infiltration test at 2.5 feet.
			P200	36	P200 = 5%
5.0					Cobbles in cuttings (approximately 40%) at 5.0 feet. P200 = 8%
			P200	14	
7.5					
10.0	very dense at 10.0 feet			31-120/4"	
	Exploration completed at a depth of 10.8 feet. Hammer efficiency factor is 82.2 percent.	10.8			Surface elevation was not measured at the time of exploration.
12.5					
15.0					
17.5					
20.0					

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: L. Gose COMPLETED: 06/11/21

BORING METHOD: core drill and hollow-stem auger (see document text) BORING BIT DIAMETER: 10 inches and 4 1/4 inches

NV5	DDG-20-01	**BORING B-2**
	JULY 2021	CENTRAL CITY APARTMENTS SALEM, OR
		FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
0.0	CONCRETE (4.4 inches).		Cored through concrete and 1-inch PVC; appears abandoned. No cables observed; runs approximately north to south. Switched to mud rotary at 0.4 foot.
	Very dense, brown GRAVEL with silt, sand, and cobbles (GP-GM); moist, gravel is fine to coarse and rounded to subrounded, sand is fine to coarse, cobbles are approximately 15%.	0.4	
2.5			Refill mud at 3.0 feet.
			Driller Comment: large cobbles likely > 50% from 0.0 to 8.5 feet.
5.0			Refill mud at 5.0 feet.
			Refill mud at 6.0 feet.
7.5			
	Exploration terminated at a depth of 9.0 feet due to refusal.	9.0	Surface elevation was not measured at the time of exploration.
10.0	Hammer efficiency factor is 82.2 percent.		
12.5			
15.0			
17.5			
20.0			



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: L. Gose

COMPLETED: 06/14/21

BORING METHOD: core drill and mud rotary (see document text)

BORING BIT DIAMETER: 6 inches and 4 7/8 inches

NV5

DDG-20-01

JULY 2021

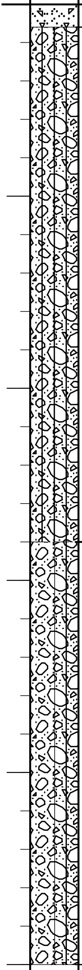
BORING B-3

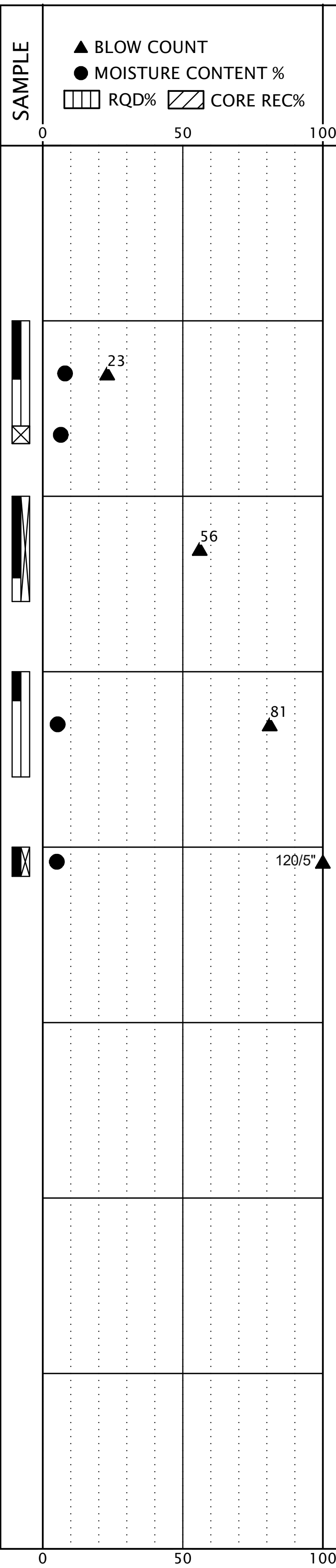
CENTRAL CITY APARTMENTS
SALEM, OR

FIGURE A-3

BORING LOG - NV5 - 1 PER PAGE DDG-20-01-B1_5.GPJ GDI_NV5.GDT PRINT DATE: 7/9/21:SN:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	CONCRETE (3.1 inches).			Cored through concrete then switched to hollow-stem auger.
	Medium dense, brown, silty GRAVEL with sand and cobbles (GM); moist, gravel is fine to coarse and rounded to subrounded, sand is fine to coarse, cobbles are approximately 10%.	0.3		
2.5			P200	P200 = 21%
	without cobbles; gravel is fine at 4.0 feet		P200	P200 = 19%
5.0	with cobbles; gravel is fine to coarse, cobbles are approximately 30% at 5.0 feet			Chatter at 5.0 feet.
	Very dense, brown GRAVEL with silt, sand, and cobbles (GP-GM); moist, gravel is fine to coarse and rounded to subrounded, sand is fine to coarse, cobbles are approximately 30%.	7.0		
7.5				
10.0				
12.5	Exploration terminated at a depth of 12.5 feet due to refusal. Hammer efficiency factor is 82.2 percent.	12.5		Surface elevation was not measured at the time of exploration.
15.0				
17.5				
20.0				





DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: L. Gose

COMPLETED: 06/11/21

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

BORING BIT DIAMETER: 10 inches and 4 1/4 inches

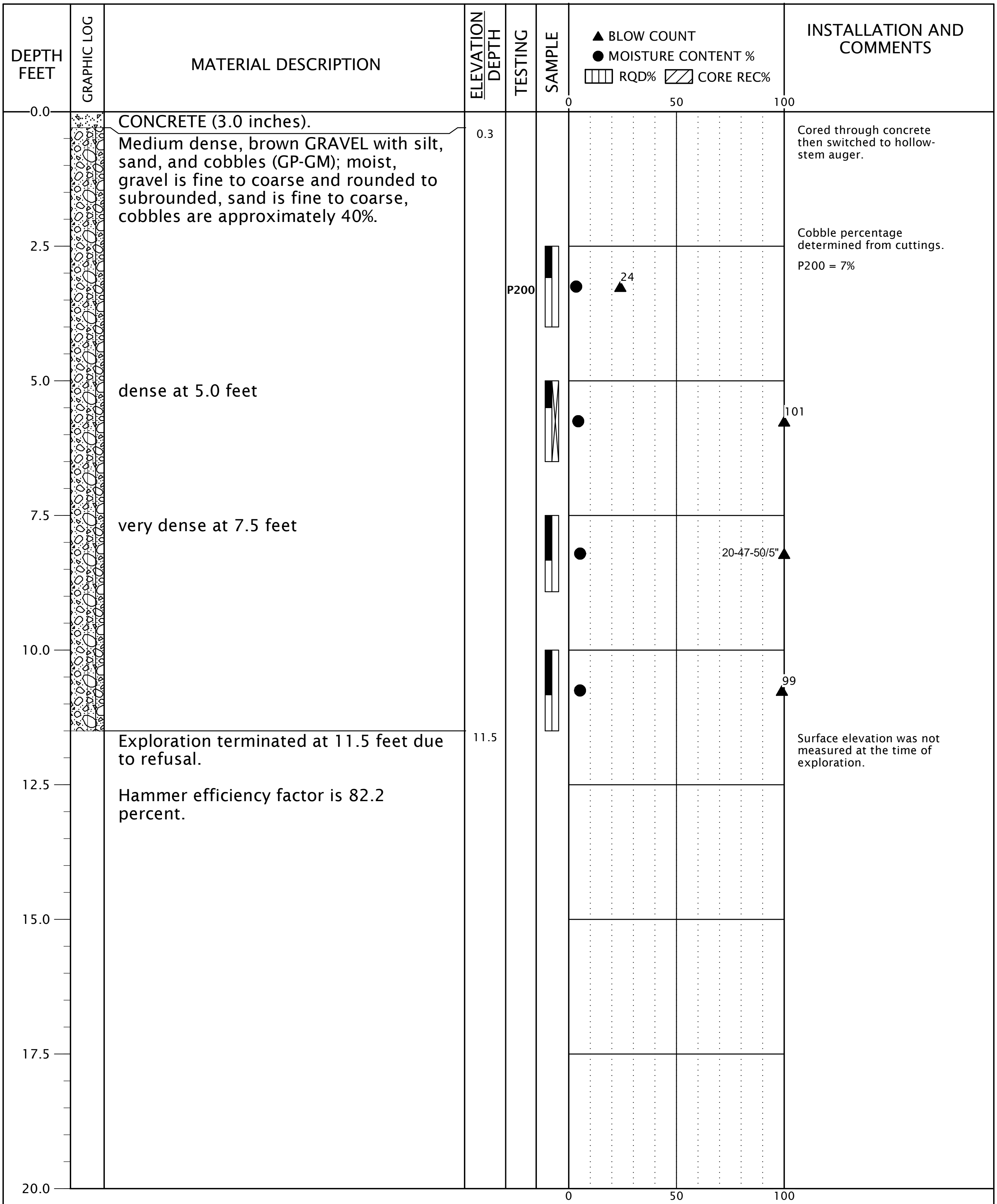
DDG-20-01

JULY 2021

BORING B-4

CENTRAL CITY APARTMENTS
SALEM, OR

FIGURE A-4



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	CONCRETE (3.0 inches).				
	Medium dense, brown GRAVEL with silt, sand, and cobbles (GP-GM); moist, gravel is fine to coarse and rounded to subrounded, sand is fine to coarse, cobbles are approximately 40%.	0.3			Cored through concrete then switched to hollow-stem auger.
2.5			P200	24	Cobble percentage determined from cuttings. P200 = 7%
5.0	dense at 5.0 feet			101	
7.5	very dense at 7.5 feet			20-47-50/5"	
10.0				99	
	Exploration terminated at 11.5 feet due to refusal. Hammer efficiency factor is 82.2 percent.	11.5			Surface elevation was not measured at the time of exploration.

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: L. Gose COMPLETED: 06/14/21

BORING METHOD: core drill and hollow-stem auger (see document text) BORING BIT DIAMETER: 10 inches and 4 1/4 inches

NV5

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JULY 2021

BORING B-5

CENTRAL CITY APARTMENTS
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	3.0		6				6			
B-2	3.0		6				5			
B-2	5.0		5				8			
B-3	2.5		10							
B-3	7.5		13							
B-4	2.5		8				21			
B-4	4.0		6				19			
B-4	7.5		5							
B-4	10.0		5							
B-5	2.5		3				7			
B-5	5.0		4							
B-5	7.5		5							
B-5	10.0		5							

NV5

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SUMMARY OF LABORATORY DATA

CENTRAL CITY APARTMENTS
SALEM, OR

FIGURE A-6

Summary of SPT Test Results

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

EMX: Maximum Energy

ETR: Energy Transfer Ratio - Rated

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

NV5

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