GEODESIGN INC
AN NV5 COMPANY

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Salem Public Works Operations Building
City Shops Complex
Salem, Oregon

For
Howard S. Wright
May 20, 2021

Project: Salem-59-01

GEODESIGN INC
AN NV5 COMPANY

May 20, 2021

Howard S. Wright
1455 NW Irving Street, Suite 400
Portland, OR 97209

Attention: Evan Grimm

Report of Geotechnical Engineering Services
Salem Public Works Operations Building
City Shops Complex
Salem, Oregon
Project: Salem-59-01

GeoDesign, Inc., DBA NV5 (GeoDesign) is pleased to submit this report of geotechnical engineering services for the proposed Salem Public Works Operations Building located in the City Shops Complex in Salem, Oregon. Our services for this project were conducted in general accordance with the contract between Howard S. Wright and GeoDesign dated March 15, 2021. We appreciate the opportunity to be of service to you. Please call if you have questions regarding this report.

Sincerely,

GeoDesign, Inc., DBA NV5

Brett A. Shipton, P.E., G.E.
Principal Engineer

RTL:BAS:kt
Attachments
One copy submitted (via email only)
Document ID: Salem-59-01-052021-geor.docx

EXECUTIVE SUMMARY

This report presents the results of our geotechnical engineering evaluation for the proposed Salem Public Works Operations Building located in the City Shops Complex in Salem, Oregon. This project includes the construction of a new two-story, approximately 45,000-square-foot office building, along with new pavement, parking lots, sitework, and other associated improvements. The new building will occupy an area that is currently being used as a gravel storage yard. The site is located along the eastern edge of the City Shops Complex.

Based on our review of the available information and the results of our explorations, it is our opinion that the site can be developed as proposed. Our specific recommendations for site development and design are provided later in this report. The following items will have an impact on design and construction of the proposed project:

- The proposed building can be supported on shallow foundations that bear on firm native soil, structural fill, or granular pads. We understand that structural fill will be placed beneath the proposed building to raise grades by approximately 2.5 feet. Higher bearing pressures can be used if the existing clay and silt soils are removed from beneath building foundations and foundations bear on dense native gravel or granular pads. Lower bearing pressures should be used if the existing clay and silt are left in place beneath foundations.
- We recommend that floor slabs be supported on at least 6 inches of imported granular material to aid as a capillary break and to provide uniform support. If soft soil is present at the slab subgrade elevation, it may be necessary to scarify and re-compact the soil or to remove and replace it with imported structural fill. We understand that a vapor barrier may be required beneath the building footprint to mitigate environmental concerns from nearby contamination. Future environmental investigations will be performed to determine if a vapor barrier is necessary.
- Our explorations indicate that the near-surface soil consists of clay, silt, clayey gravel, or silty gravel. We measured relatively low infiltration rates in these soil types. If on-site stormwater infiltration is used, we recommend that infiltration testing be performed during construction to verify the design infiltration rates are being achieved.
- State of Oregon hazard mapping shows that the site is within the 100-year flood zone. As a result, it is our opinion that there is a risk of flooding at the site that should be considered by the design team.
- Our seismic analysis indicates that the site should be classified as seismic Site Class C. The on-site clay and silt will be difficult to use as structural fill because this material will be very difficult to adequately moisture condition and compact. The on-site gravel will be suitable for use as structural fill, provided it is properly moisture conditioned and oversized material is removed.
- A small amount of demolition may be required to construct the proposed project. This includes the two covered carport structures, as well as abandoned features that may be uncovered during construction. We understand that multiple abandoned sewer pipes and laterals may be present beneath the proposed building footprint and paved areas. We recommend that these abandoned pipes be completely removed or filled with grout during construction.

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ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ACP	asphalt concrete pavement
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BGS	below ground surface
CRBG	Columbia River Basalt Group
CSZ	Cascadia subduction zone
DOGAMI	Oregon Department of Geology and Mineral Industries
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
km	kilometers
MCE	maximum considered earthquake
MCE_G	maximum considered earthquake geometric mean
mps	meters per second
NA	not applicable
OSHA	Occupational Safety and Health Administration
OSHPD	Office of Statewide Health Planning and Development
OSSC	Oregon Standard Specifications for Construction (2021)
pcf	pounds per cubic foot
PG	performance grade
PGA	peak ground acceleration
PGA_M	maximum considered earthquake geometric mean peak ground acceleration adjusted for site affects
psf	pounds per square foot
psi	pounds per square inch
SEAOC	Structural Engineers Association of California
SOSSC	State of Oregon Structural Specialty Code
SPT	standard penetration test
USGS	U.S. Geological Survey
Vs_{30}	shear wave velocity for the upper 100 feet (30 meters)

1.0 INTRODUCTION

This report presents the results of our geotechnical engineering evaluation for the proposed Salem Public Works Operations Building located in the City Shops Complex in Salem, Oregon. This project includes the construction of a new two-story, approximately 45,000-square-foot office building, along with new pavement, parking lots, sitework, and other associated improvements. The new building will occupy an area that is currently being used as a gravel storage yard. The site is located along the eastern edge of the City Shops Complex. The site location relative to surrounding physical features is shown on Figure 1.

The project structural engineer, KPFF Consulting Engineers, estimates that maximum column, wall, and floor slab loads will be 100 kips, 2.5 kips per foot, and 100 psf, respectively. The project civil engineer, Westech Engineering, Inc., estimates that grades will be raised approximately 2.5 feet at the proposed building location. We anticipate that cuts will be negligible. We should be contacted to update our recommendations if the actual structural loads, cuts, or fills will exceed these estimates.

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

2.0 PURPOSE AND SCOPE

The purpose of this evaluation was to provide geotechnical engineering recommendations for use in design and construction of the proposed development. Specifically, we completed the following scope of services:

- Reviewed readily available, published geologic data and in-house files for existing information on subsurface conditions within the site vicinity.
- Coordinated and managed the field exploration, including utility locates, coordination with existing tenants, and scheduling subcontractors.
- Conducted a subsurface exploration program that consisted of drilling six borings to depths between 8.4 and 21.4 feet BGS at the proposed building and pavement locations.
- Performed infiltration tests in three of the borings.
- Maintained continuous logs of the explorations and collected samples at representative intervals.
- Conducted a laboratory testing program that consisted of the following tests:
  - Seven moisture content determinations in general accordance with ASTM D2216
  - Five particle-size analyses in general accordance with ASTM D1140
  - Two Atterberg limits tests in general accordance with ASTM D4318
- Provided recommendations for site preparation and grading, including temporary and permanent slopes, fill placement criteria, suitability of on-site soil for fill, subgrade preparation, and wet weather earthwork.
- Provided foundation support recommendations for the proposed building, including preferred foundation type, allowable bearing pressure, lateral resistance parameters, and settlement estimates.

- Provided recommendations for use in design of conventional retaining walls, including backfill and drainage requirements and lateral earth pressures.
- Evaluated groundwater conditions at the site, and provided general recommendations for dewatering during construction and subsurface drainage (if required).
- Provided a site-specific seismic hazard evaluation in accordance with the procedures outlined in the 2019 SOSSC.
- Prepared this geotechnical engineering report that presents our findings, conclusions, and recommendations.

3.0 SITE CONDITIONS

3.1 REGIONAL AND SITE GEOLOGY

The site is located in the Willamette Valley physiographic province, which extends from approximately Cottage Grove, Oregon, in the south to the Columbia River in the north (Orr and Orr, 1999). The Willamette Valley province is a part of the larger Puget Sound-Willamette Valley lowland, a tectonically active forearc basin located along the convergent Cascadia margin. The lowland is generally an elongated alluvial/fluvial plain bordered on the west by the Coast Ranges and on the east by the Cascade Mountains.

The site is located in a low-lying area east of the Willamette River and north of the Salem Hills. The near-surface geologic unit is mapped as the Linn Gravel; a Quaternary to Upper Pleistocene alluvium composed of stratified fine to coarse fluvial gravels deposited as an alluvial fan during an early stage of the Santiam River (Bela, 1981). The Linn Gravel generally ranges in thickness from 30 to 40 feet to 300 feet. The Grande Ronde Member of the Columbia River Basalt Group underlies the Linn Gravel in this region.

3.2 SURFACE CONDITIONS

The site is an undeveloped parcel within the City of Salem City Shops Complex. The site is bound on the north and east by retail stores, parking lots, and a motel. The site is bound on the west and south by the City Shops Complex. The site measures approximately 350 feet by 280 feet. Most of the site is covered with gravel that was spread on the ground surface. The northern approximately one-third of the site is covered by grass. The site is currently being used by the City of Salem as a storage yard for equipment, trailers, aggregate stockpiles, and other miscellaneous items. There are two covered carport structures in the southwestern corner of the site that will be removed. There are several large trees scattered across the northern portion of the site in the grass area. The site is relatively level, with elevations ranging from approximately 178 to 180 feet. A topography map that we reviewed indicates there is a 10-foot-wide sewer easement that crosses the northern half of the site in the east-west direction that will be beneath a future parking lot.

3.3 SUBSURFACE CONDITIONS

We explored subsurface conditions at the site by drilling six borings (B-1 through B-6) to depths between 8.4 and 21.4 feet BGS. The approximate locations of the explorations are shown on Figure 2. Descriptions of the field exploration and laboratory testing programs, the exploration logs, and results of our laboratory testing are presented in Appendix A.

Based on the information obtained from our explorations, the soil profile generally consists of crushed rock fill or grass at the ground surface that is underlain by clay, silt, and gravel. The following sections provide a detailed description of subsurface conditions encountered at the site.

3.3.1 Crushed Rock Fill

We encountered a layer of crushed rock fill at the ground surface in most of our borings. This layer is approximately 1 foot thick and occasionally includes fragments of AC. We did not observe this layer of crushed rock fill in our northern-most borings drilled in a grass-covered area (boring B-4 and B-6). Where the crushed rock fill was not present, we observed a 3.0 to 3.5-inch-thick root zone in grass-covered areas.

3.3.2 Clay and Silt

In borings B-1, B-2, and B-4 we observed a layer of clay and silt that was beneath the crushed rock fill (where present). The clay and silt extend to a depth of approximately 4.5 feet BGS. The clay and silt are generally medium stiff to stiff, gray to brown with orange mottling, moist, exhibit high plasticity, contain varying amounts of fine- to coarse-grained sand, and contain trace organics. Laboratory testing indicates that moisture contents in the clay and silt layer ranged from 22 to 38 percent at the time of our exploration. Soil of this type and consistency generally exhibits relatively low to moderate strength and moderate compressibility.

3.3.3 Gravel

Beneath the clay and silt (where present) we observed a layer of gravel that extends to a depth of at least 21.4 feet BGS, the maximum depth we explored. The gravel is generally medium dense to very dense, brown, moist to wet, fine to coarse, subrounded to subangular, contains varying amounts of fine- to coarse-grained sand, and contains varying amounts of fines. Laboratory testing indicates that moisture contents in the gravel layer ranged from 8 to 16 percent at the time of our exploration. Soil of this type and consistency generally exhibits relatively high strength and low compressibility.

3.3.4 Groundwater

We observed groundwater at depths of 10 to 13 feet BGS in our explorations. We also observed a small amount of perched water in boring B-1 at a depth of approximately 1.6 feet BGS. We note that the depth to groundwater will fluctuate in response to seasonal changes, changes in surface topography, and other factors.

3.4 INFILTRATION TESTING

We performed three falling head infiltration tests to evaluate infiltration rates for potential stormwater facilities. We performed all three tests at a depth of 3.5 feet BGS, as requested by the project civil engineer. We performed the tests in general accordance with the City of Salem infiltration testing requirements inside hollow-stem augers using the encased falling head test method. The tests were conducted using a water head of approximately 6 inches. Representative soil samples were collected at the infiltration test depth for grain-size analysis. Table 1 summarizes the infiltration test results and fines content determinations. The exploration logs and laboratory test results are presented in Appendix A. Plots of the infiltration data we collected are presented in Appendix B.

Table 1. Measured Infiltration Rates

Location	Depth (feet BGS)	Material	Infiltration Rate[1] (inches per hour)	Fines Content[2] (percent)
B-4	3.5	Sandy SILT	0.0	66
B-5	3.5	Clayey GRAVEL with sand	3.2	18
B-6	3.5	Silty GRAVEL with sand	0.3	16

1. Infiltration rates are not factored.
2. Fines content: material passing the U.S. Standard No. 200 sieve

The infiltration rates provided in Table 1 are measured rates and are unfactored. Factors of safety should be applied to the measured infiltration rates by the civil engineer during design to account for soil variations, the potential for long-term clogging due to siltation and buildup of organic material, maintenance, influent/pre-treatment control, and consequences of failure. We recommend that a factor of safety of at least 2.0 be applied to the field-measured infiltration rates.

Based on the infiltration rates we measured, it appears that infiltration rates at the site are relatively low. If on-site stormwater infiltration is used, we recommend that infiltration testing be performed during construction to verify that the design infiltration rates are being achieved.

4.0 GEOLOGIC HAZARDS

We evaluated the presence of geologic hazards within the site vicinity based on a review of published literature and our experience with nearby projects. Individual geologic hazards are summarized in the following sections.

4.1 LANDSLIDE HAZARDS

The topography of the site and surrounding properties are relatively level. State of Oregon hazard mapping does not show any landslides at the site or on adjacent sites (DOGAMI, 2018). As a result, it is our opinion that the risk of landslides at the site is low.

4.2 SEISMIC HAZARDS

4.2.1 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 silty sand and silt may be moderately susceptible to liquefaction under relatively higher levels of ground shaking. Liquefaction can densify subsurface soil, which can result in settlement at the ground surface.

Soil below the groundwater level consists of dense gravel that is not susceptible to liquefaction. As a result, it is our opinion that liquefaction is not a hazard at this site.

4.2.2 Lateral Spreading

Lateral spreading is a liquefaction-related seismic hazard and occurs on gently sloping sites or flat sites adjacent to an open face, such as a riverbank, that are underlain by liquefiable soil. Liquefied soil adjacent to an open face can flow toward the open face, resulting in lateral ground displacement. Since the soil at the site is not susceptible to liquefaction, it is our opinion that the site is also not susceptible to lateral spreading.

4.2.3 Fault Surface Rupture

Based on USGS mapping, the are no active faults mapped within approximately 3 miles of the site (USGS, 2021). In our opinion, the risk of fault surface rupture at the site is low.

4.2.4 Seismically Induced Landslides

Earthquake-induced landslides generally occur in steeper slopes comprised of relatively weak soil deposits. Since the topography of the site and surrounding properties are relatively flat, it is our opinion that seismic-induced landslides are not a hazard at the site.

4.2.5 Ground Motion Amplification

Soil capable of significantly amplifying ground motions beyond the levels determined by our site-specific seismic hazard analysis was not encountered during our subsurface explorations. We conclude the level of amplification determined by our seismic hazard analysis is appropriate and the proposed building can be designed using the levels of ground shaking prescribed by ASCE 7-16.

4.2.6 Dry Seismic Settlement

Dry seismic settlement due to earthquakes is most prevalent in relatively deep deposits of dry, clean sand, which are not present at the site. We do not anticipate that significant settlement will occur during design levels of ground shaking.

4.2.7 Subsidence/Uplift

Subduction zone earthquakes can cause vertical tectonic movements. The movements reflect coseismic strain release accumulation associated with interplate coupling in the CSZ. Based on our review of the literature, the locked zone of the CSZ is located in excess of 60 miles from the site. Consequently, we do not anticipate that subsidence or uplift is a significant design concern.

4.2.8 Lurching

Lurching is a phenomenon generally associated with very high levels of ground shaking, which cause localized failures and distortion of the soil. The anticipated ground accelerations from our site response analysis are below the threshold required to induce lurching of the site soil.

4.2.9 Tsunami and Seiche

The site is not in a mapped tsunami inundation zone (DOGAMI, 2018) and is away from large, enclosed bodies of water that may develop seiches. In our opinion, tsunamis and seiches are not hazards at the site.

4.3 FLOOD HAZARDS

State of Oregon hazard mapping shows that the site is within the 100-year flood zone (DOGAMI, 2018). As a result, it is our opinion that there is a risk of flooding at the site that should be considered by the design team.

4.4 VOLCANIC HAZARDS

State of Oregon hazard mapping indicates there are no mapped volcanic hazards near the site (DOGAMI, 2018).

5.0 DESIGN RECOMMENDATIONS

5.1 FOUNDATION SUPPORT

5.1.1 General

Based on the results of our explorations and analysis, we recommend that the proposed building be supported on shallow foundations that bear on firm native soil, structural fill, or granular pads. We understand that structural fill will be placed beneath the proposed building to raise grades by approximately 2.5 feet. Higher bearing pressures can be used if the existing clay and silt soils are removed from beneath building foundations and foundations bear on dense native gravel or granular pads. Lower bearing pressures should be used if the existing clay and silt are left in place beneath foundations.

Granular pads should be used to replace clay and silt that are over-excavated. The granular pads should extend down to dense native gravel. The granular pads should extend at least 6 inches beyond the margins of the footings for every foot of depth. The material should consist of durable, well-graded, crushed ¾- or 1½-inch-minus rock containing no organic or other deleterious materials; should have a maximum particle size of 1½ inches; should have at least two mechanically fractured faces; and should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. 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.

We recommend that isolated column and continuous wall footings have minimum widths of 24 and 18 inches, respectively. The bottom of exterior footings should be founded at least 18 inches below the lowest adjacent grade. Interior footings should be founded at least 12 inches below the bottom of the floor slab.

All foundation subgrade should be evaluated by the project geotechnical engineer or their representative to evaluate bearing conditions. Observations should determine whether all loose or soft material, organic material, unsuitable fill, prior topsoil zones, and softened subgrades (if present) have been removed. Localized deepening of foundation excavations may be required to penetrate unsuitable material.

5.1.2 Bearing Capacity

Foundations that are supported on dense native gravel, or that are supported on granular pads that extend to dense native gravel, can be sized using an allowable bearing pressure of

5,000 psf. This value may be increased by one-third for short-term loads such as wind or seismic forces. These bearing pressure values can only be used if the existing clay and silt are removed from beneath foundations.

Alternatively, foundations that are supported on native clay or silt or new structural fill placed to raise grades should be designed using an allowable bearing pressure of 2,500 psf. This value may be increased by one-third for short-term loads such as wind or seismic forces. At locations where the layer of existing crushed rock fill will be left in place, we recommend compacting it before additional fill is placed to raise grades.

5.1.3 Settlement

We anticipate that footings supporting the estimated design loads and constructed as recommended will experience less than 1 inch of total post-construction settlement and ½ inch of differential settlement between similarly loaded adjacent footings.

5.1.4 Lateral Resistance

Lateral loads on spread footings can be resisted by passive earth pressure on the sides of the footings and by friction along the base of the footings. Our analysis indicates that the available passive earth pressure is 350 pcf modeled as an equivalent fluid pressure. The upper 12 inches of adjacent, unpaved areas should not be considered when calculating passive resistance. A coefficient of friction value equal to 0.30 may be used when calculating resistance to sliding for foundations in direct contact with native clay or silt. Foundations in contact with crushed rock or native gravel should be designed using a coefficient of friction value of 0.50.

5.2 SLABS ON GRADE

We recommend floor slabs be supported on at least 6 inches of imported granular material to aid as a capillary break and to provide uniform support. If soft soil is present at the slab subgrade elevation, it may be necessary to scarify and re-compact the soil or to remove and replace it with imported structural fill. The 6-inch-thick layer of imported granular material should have a maximum particle size of 1½ inches, should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have at least two mechanically fractured faces. The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

Vapor barriers beneath floor slabs are typically required by flooring manufactures to maintain the warranty on their products. In our experience, adequate performance of floor adhesives can be achieved by using a clean base rock (less than 5 percent fines) beneath the floor slab with no vapor barrier. In fact, vapor barriers can frequently cause moisture problems by trapping water beneath the floor slab that is introduced during construction. If a vapor barrier is used, water should not be applied to the base rock prior to pouring the slab and the work should be completed during extended dry weather so that rainfall is not trapped on top of the vapor barrier. Selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team. If requested, we can provide additional information to assist you with your decision.

We understand that a vapor barrier may be required beneath the building footprint to mitigate environmental concerns from nearby contamination. Future environmental investigations will be performed to determine if a vapor barrier is necessary.

5.3 SEISMIC DESIGN PARAMETERS

We performed a site-specific seismic hazard evaluation for this project, which is presented in Appendix C.

5.4 RETAINING WALLS

5.4.1 Assumptions

These retaining wall recommendations apply to permanent above-grade retaining walls. 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 10 feet in height, (3) drains are provided to prevent hydrostatic pressure from developing, and (4) the retained soil is level. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

5.4.2 Retaining Wall Design Parameters

For unrestrained retaining walls, an active equivalent fluid pressure of 35 pcf should be used for design. Where retaining walls are restrained from rotation prior to being backfilled, an at-rest equivalent fluid pressure of 55 pcf should be used for design. A superimposed seismic lateral force should be calculated based on a dynamic force of $7H^2$ pounds per linear foot of wall (where H is the height of the wall in feet). The load should be applied as a distributed load with the centroid located at a distance of 0.6H above the base of the wall.

If surcharges (e.g., retained slopes, building foundations, vehicles, terraced walls, etc.) are located within a horizontal distance from the back of a wall equal to the height of the wall, additional pressures will need to be accounted for in the wall design. Figure 3 presents additional pressures resulting from some common loading scenarios. Our office should be contacted for additional pressures resulting from alternate loading scenarios. We recommend a vertical live load of 250 psf be applied at the surface of the retained soil where the wall retains roadways.

The base of the wall footing excavations should extend a minimum of 18 inches below the lowest adjacent grade. The wall footings should be designed in accordance with the guidelines in the “Foundation Support” section. At locations where there is a slope in front of the retaining wall, we recommend a minimum 5-foot-wide, horizontal bench be placed between the wall and the top of the slope.

5.4.3 Retaining Wall Drainage and Backfill

The above design parameters have been provided assuming that drains will be installed behind the walls to prevent buildup of hydrostatic pressures. 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 meeting the requirements described in the “Structural Fill” section. Alternatively, the native soil can be used as backfill material, provided a minimum 2-foot-wide column of angular drain rock wrapped in a drainage geotextile is placed against the wall and the native soil can be adequately moisture conditioned for compaction. The rock column should extend from the perforated drainpipe or foundation drains to within approximately 1 foot of the ground surface. The angular drain rock should have a maximum particle size of 2 inches, should have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve, should have at least two mechanically fractured faces, and should be free of organic material and other unsuitable materials.

Perforated collector pipes should be placed at the base of the granular backfill behind the walls. The pipe should be embedded in a minimum 2-foot-wide zone of angular drain rock wrapped in a drainage geotextile fabric. The collector pipes should discharge at an appropriate location away from the base of the wall. Unless measures are taken to prevent backflow into the drainage system of the wall, the discharge pipe should not be tied directly into stormwater drain systems.

Backfill should be placed and compacted as recommended for structural fill, with the exception of backfill placed immediately adjacent to walls. Backfill adjacent to walls should be compacted to a lesser standard to reduce the potential for compaction-induced earth pressures on the walls. Backfill located within a horizontal distance of 3 feet from the retaining walls should 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 (such as slabs, sidewalk, or pavement) will be placed adjacent to the wall, we recommend that the upper 2 feet of fill be compacted to 95 percent of the maximum dry density, as determined by ASTM D1557. 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 construction, unless survey data indicates that settlement is complete prior to that time.

5.5 PERMANENT SLOPES

Permanent cut or fill slopes should not exceed a gradient of 2H:1V, unless specifically evaluated for stability. Upslope buildings, access roads, and hardscapes should be set back a minimum of 5 feet from the crest of such slopes. 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.

5.6 DRAINAGE

5.6.1 Surface

The finished ground surface around the building should be sloped away from foundations at a minimum 2 percent gradient for a distance of at least 5 feet. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. Runoff water should not be directed to the top of slopes.

5.6.2 Subsurface

We recommend foundation drains be installed around the perimeter of the building at locations where the finish floor elevation will be lower than adjacent grades. Foundation drains are not necessary at locations where the finish floor elevation will be above adjacent grades. We recommend foundation drains and roof downspouts or scuppers discharge to a solid pipe that carries the collected water to an appropriate stormwater system that is designed to prevent backflow. If drywells are used, we recommend that the top of the perforated drywell sections be at least 10 feet below adjacent on-site or off-site enclosed spaces such as basements, elevator pits, etc.

5.6.3 Temporary

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 grading. The contractor should keep all footing excavations and building pads free of water during rough and finished grading of the building site.

5.7 PAVEMENT

5.7.1 Design Assumptions and Parameters

The proposed project includes the construction of new AC parking lots and drive aisles. The parking lots will be used by passenger vehicles and we anticipate that most of the pavement traffic will consist of automobiles and pickup trucks. We anticipate that some delivery truck, garbage truck, or fire truck traffic will also occur in the drive aisles. We anticipate this pavement will not be used regularly by large trucks or construction-type equipment. Our pavement recommendations are based on the assumptions listed below. If any of these assumptions are incorrect, our office should be contacted with the appropriate information so that the pavement designs can be revised. Our pavement design recommendations assume the subgrade has been prepared in accordance with the “Site Preparation” and “Structural Fill” sections.

- A resilient modulus value of 3,500 psi for native subgrade based on the soil type and SPTs.
- A pavement design life of 20 years.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability of 75 percent and standard deviation of 0.5.
- No growth.
- Traffic will consist of 400 cars per day; ten 2-axle delivery trucks per day; and one 3-axle delivery truck, garbage truck, or similarly heavy vehicles per day.
- Construction traffic will not be allowed on new pavement. If construction traffic is to be allowed on the newly constructed pavement, our design pavement sections will need to be revised.

5.7.2 Recommended AC Pavement Design Sections (Post Construction)

Our pavement design recommendations for the assumptions and loads provided above are presented in Table 2.

Table 2. Recommended Standard Pavement Sections

Pavement Use	AC Thickness[1] (inches)	Aggregate Base Thickness[1,2] (inches)
Drive Aisles – Automobile and Occasional Heavy Vehicle	3.0	12.0
Automobile Parking Only	3.0	8.0

1. All thicknesses are intended to be the minimum acceptable values. Additional thickness will be necessary if construction traffic is allowed on the pavement.
2. A subgrade geotextile fabric should be used beneath the aggregate base rock at locations where the subgrade soil consists of clay or silt.

The subgrade should be unyielding or compacted to 95 percent of the maximum dry density, as determined by ASTM D1557. Areas that exhibit yielding or pumping should be repaired, as described in this report. A subgrade geotextile fabric should be used to provide additional support and extend the life of the pavement if the subgrade consists of fine-grained soil. Fine-grained subgrade soil can more easily be pumped into aggregate base when the subgrade soil is wet, traffic volumes are high, and traffic consists of heavy vehicles. The use of a subgrade geotextile fabric will help protect the aggregate base from being weakened by fine-grained subgrade soil gradually migrating into the aggregate base.

5.7.3 Pavement Materials

A submittal should be made for each pavement material prior to the start of paving operations. Each submittal should include the test information necessary to evaluate the degree to which the material's properties comply with the properties that were recommended or specified. The geotechnical engineer and other appropriate members of the design team should review each submittal.

5.7.3.1 Aggregate Base

Imported granular material used as aggregate base for pavement should consist of ¾-, 1-, or 1½-inch-minus material (depending on the application) and meet the requirements in OSSC 00641 (Aggregate Subbase, Base, and Shoulders). In addition, the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

5.7.3.2 AC

The AC should be Level 2, ½-inch, dense ACP according to OSSC 00744 (Asphalt Concrete Pavement) and compacted to 92 percent of the theoretical maximum density of the mix, as determined by AASHTO T 209. The minimum and maximum lift thicknesses are 2.0 and 3.0 inches, respectively, for ½-inch ACP. Asphalt binder should be performance graded and conform to PG 64-22. AC paving should only occur when ground temperatures are 40 degrees Fahrenheit or warmer.

5.7.3.3 Subgrade Geotextile Fabric

A subgrade geotextile fabric should be placed as a barrier between fine-grained subgrade (if present) and granular material. The geotextile fabric should meet the specifications provided in OSSC 02320 (Geosynthetics) for separation geotextiles (Table 02320-4) and be installed in accordance with OSSC 00350 (Geosynthetic Installation).

6.0 CONSTRUCTION RECOMMENDATIONS

6.1 SITE PREPARATION

6.1.1 Stripping and Grubbing

Stripping and grubbing will be required to remove any tree roots, shrubs, and topsoil that remain in landscape areas after cuts are performed. Root material should be removed from all building, pavement, and structural fill areas. We anticipate that a stripping depth of 3 to 3.5 inches will generally be adequate for removing the topsoil in the grass areas. The actual stripping and grubbing depth should be based on field observations at the time of construction. Stripping and grubbing should extend at least 5 feet beyond the limits of proposed building and pavement areas. Excavated roots should be transported off site for disposal or used as fill in landscaped areas.

6.1.2 Demolition

Demolition may be required to remove the two existing covered carport structures from the site. If abandoned foundations, walls, slabs, utilities, or other buried elements are encountered during construction beneath the new building, they should be completely removed. Any monitoring wells or underground storage tanks that are encountered should be abandoned in accordance with state and local regulations prior to site redevelopment. Excavations resulting from the demolition of existing improvements should be backfilled with compacted structural fill as recommended in this report. The bottom of the excavations should expose firm subgrade. The sides of the temporary excavations should be cut into firm material and sloped no steeper than 1½H:1V.

We understand that multiple abandoned sewer pipes and laterals may be present beneath the proposed building footprint and paved areas. We recommend that these abandoned pipes be completely removed or filled with grout during construction.

6.1.3 Subgrade Evaluation

A member of our geotechnical or geology staff should observe all footing, floor slab, and hardscape subgrades after stripping and grubbing, excavation, scarifying and re-compaction (if applicable), and placement of structural fill have been completed to confirm that there are no areas of unsuitable or unstable soil. The subgrade should be evaluated using moisture-density testing, a hand probe, or proof rolling with a fully loaded dump truck (or similar heavy, rubber tire construction equipment). Soft, loose, or unsuitable soil found at the subgrade level should be over-excavated and replaced with structural fill.

6.2 EXCAVATION

6.2.1 General

Excavations will be required to construct foundations, utilities, stormwater infiltration facilities, and other improvements. Conventional earthmoving equipment in proper working condition should be capable of making the necessary excavations. We anticipate that temporary excavation sidewalls will generally stand vertical to a depth of approximately 4 feet, provided water seepage does not occur. Excavation slopes may need to be flattened if raveling gravel is encountered in the excavation.

Excavations deeper than 4 feet will require shoring or should be sloped. Sloped excavations may be used to vertical depths of 10 feet BGS and should have side slopes no steeper than 1½H:1V, provided groundwater seepage does not occur. We recommend a minimum horizontal distance of 5 feet from the edge of existing improvements to the top of any temporary slope. All cut slopes should be protected from erosion by covering them during wet weather. If seepage, sloughing, or instability is observed, the slope should be flattened or shored. Shoring will be required where slopes are not possible. The contractor should be responsible for selecting the appropriate shoring system.

Excavations should not be allowed to undermine adjacent improvements. If existing roads or structures are located near a proposed excavation, unsupported excavations can be maintained outside of a 1H:1V downward projection that starts 5 feet from the base of the existing elements. Excavations that must be inside of this zone should be supported by temporary or permanent shoring designed for moment resistance for the full height of the excavation, including kick-out for the full buried depth of the retaining system.

While we have described certain approaches to performing excavations, it is the contractor's responsibility to select the excavation and dewatering methods, monitor the excavations for safety, and provide any shoring required to protect personnel and adjacent improvements. All excavations should be in accordance with applicable OSHA and state regulations.

6.2.2 Excavation Dewatering

Excavations will be above the groundwater level. However, some perched water could still seep into the site excavations, especially after periods of heavy rain. We anticipate that dewatering methods consisting of pumping water from excavation sumps will generally be adequate. If possible, we recommend that construction be scheduled for the dry season. Water generated during dewatering operations should be treated, if necessary, and pumped to a suitable disposal point.

Where groundwater seepage occurs in excavations, we recommend placing at least 1 foot of stabilization material at the base of the excavations. The stabilization material 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 materials.

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

6.3 STRUCTURAL FILL

Structural fill includes fill beneath foundations, slabs, hardscapes, and other structures. Structural fill should generally consist of particles no larger than 3 inches in diameter and should be free of organic material and other deleterious materials. Recommendations for suitable fill material are provided in the following sections.

6.3.1 On-Site Soil

We recommend that the on-site clay and silt not be used as structural fill because this material will be very difficult to adequately moisture condition and compact. The on-site gravel will be suitable for use as structural fill. On-site gravel 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. It may be necessary to remove oversized material. It may also be necessary to moisture condition the gravel before it can be used as structural fill. We recommend using imported granular material for structural fill if the moisture content of the on-site gravel cannot be reduced or if there is too much oversized material to remove.

6.3.2 Imported Granular Material

Imported granular material should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand that is fairly well graded between coarse and fine and has less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. All granular material must be durable such that there is no degradation of the material during and after installation as structural fill. 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. During the wet season or when wet subgrade conditions exist, the initial lift should have a maximum thickness of 18 inches and should be compacted by rolling with a smooth-drum, non-vibratory roller.

6.3.3 Recycled Concrete

Recycled concrete can be used for structural fill, provided the concrete is broken to a maximum particle size of 3 inches. This material must be durable such that there is no degradation of the material during and after installation as structural fill. Recycled concrete can be used as trench backfill if it meets the size requirements for that application and the requirements for imported granular material. 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.

6.3.4 Trench Backfill Material

City of Salem trench backfill requirements should be followed for any public utilities that are installed.

6.3.5 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 materials. Stabilization material should be placed in lifts between 12 and 24 inches thick and compacted to a well-keyed, firm condition.

6.4 EROSION CONTROL

The on-site soil is susceptible to erosion. Consequently, we recommend that slopes be covered with an appropriate erosion control product if construction occurs during periods of wet weather. We recommend that all slope surfaces be planted as soon as practical to minimize erosion. 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.

6.5 WET WEATHER CONSTRUCTION

Trafficability of soil at the ground surface may be difficult during extended wet periods or when the moisture content of the surface soil is more than a few percentage points above optimum. If not carefully executed, earthwork activities can create extensive soft areas, resulting in significant repair costs.

When the subgrade is wet of optimum, site preparation may need to be accomplished using track-mounted equipment loading into trucks supported on granular haul roads or working blankets. Based on our experience, at least 12 inches of granular material are typically required for light staging areas and at least 18 inches of granular material for haul roads subject to repeated equipment traffic. We typically recommend that imported granular material for haul roads and working blankets consist of durable crushed rock that is well graded and has less than 8 percent by dry weight passing the U.S. Standard No. 200 sieve. Where silt or clay is exposed at the ground surface, the performance of haul roads can typically be improved by placing a geotextile on the subgrade before placing the granular material. The granular material should be placed in a single lift and the surface compacted until well keyed. Although we have presented typical recommendations for haul road and working blankets, the actual thickness and material should be determined by the contractor based on their sequencing of the project and the type and frequency of construction equipment. The base rock thickness for building and pavement areas is intended to support post-construction design loads and will not support construction traffic when the subgrade soil is wet. If construction is planned for periods when the subgrade soil is wet, an increased thickness of base rock will be required.

7.0 OBSERVATION OF CONSTRUCTION

Satisfactory foundation and earthwork performance depends to a large degree on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications.

Subsurface conditions observed during construction should be compared with those encountered during the subsurface 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 GeoDesign be retained to observe earthwork activities. We anticipate this will consist of evaluating foundation subgrade, observing the placement of structural fill and repair of soft subgrade areas, observing AC pavement installation, and performing laboratory compaction and field moisture-density tests.

8.0 LIMITATIONS

We have prepared this report for use by Howard S. Wright and their design and construction teams for the proposed project. The data and report can be used for bidding or estimating purposes, but our report, conclusions, and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other 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 preliminary at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, configuration, design loads, or type of construction, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written verification 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 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,

GeoDesign, Inc., DBA NV5

Ryan Lawrence

Ryan T. Lawrence, P.E.
Associate Engineer

Brett A. Shipton, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL
ENGINEER
56207
OREGON
July 11, 2000
BRETT A. SHIPTON
EXPIRES: 6/30/22

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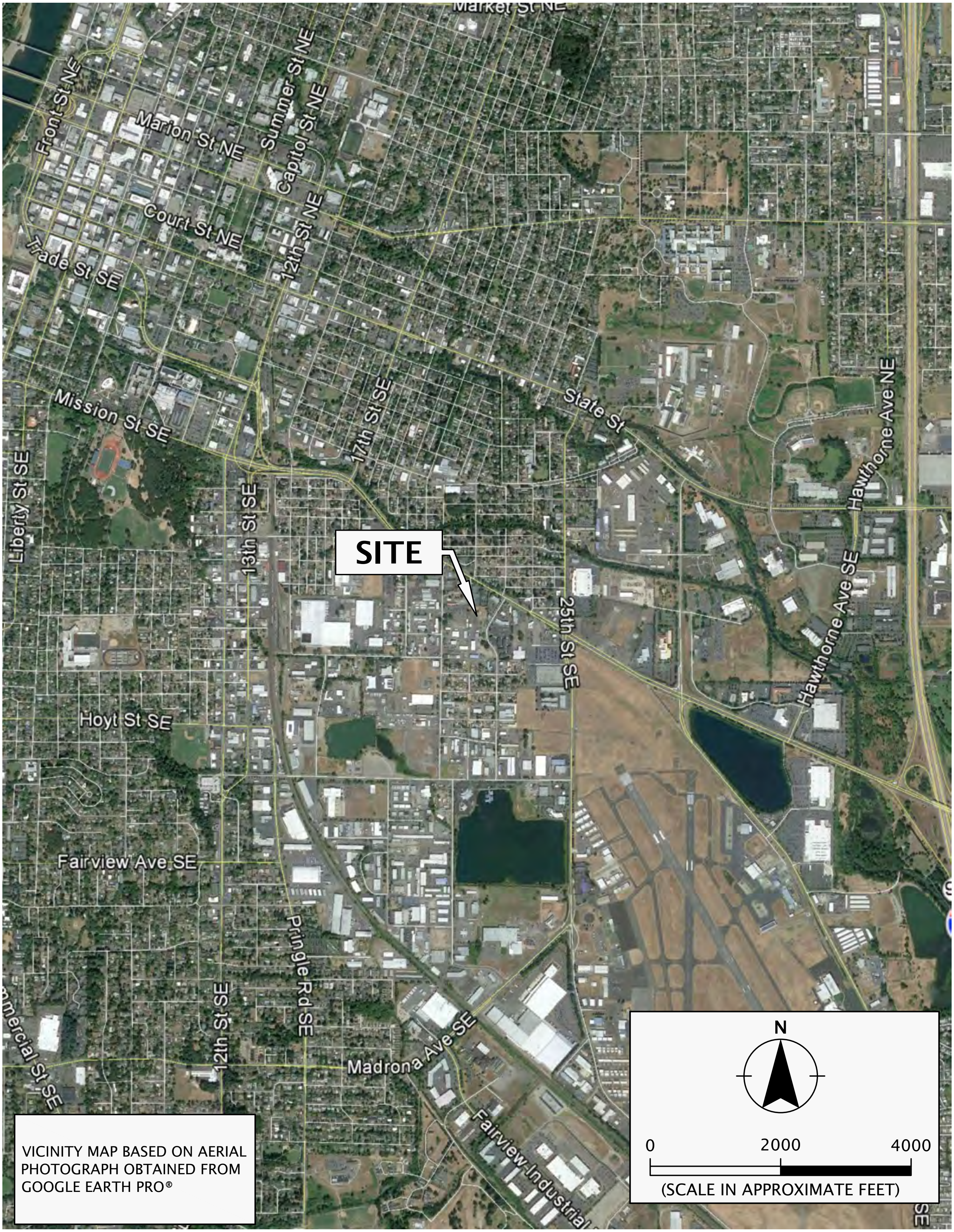
FIGURES



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

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SALEM-59-01

VICINITY MAP

MAY 2021

SALEM PUBLIC WORKS OPERATIONS BUILDING
SALEM, OR

FIGURE 1

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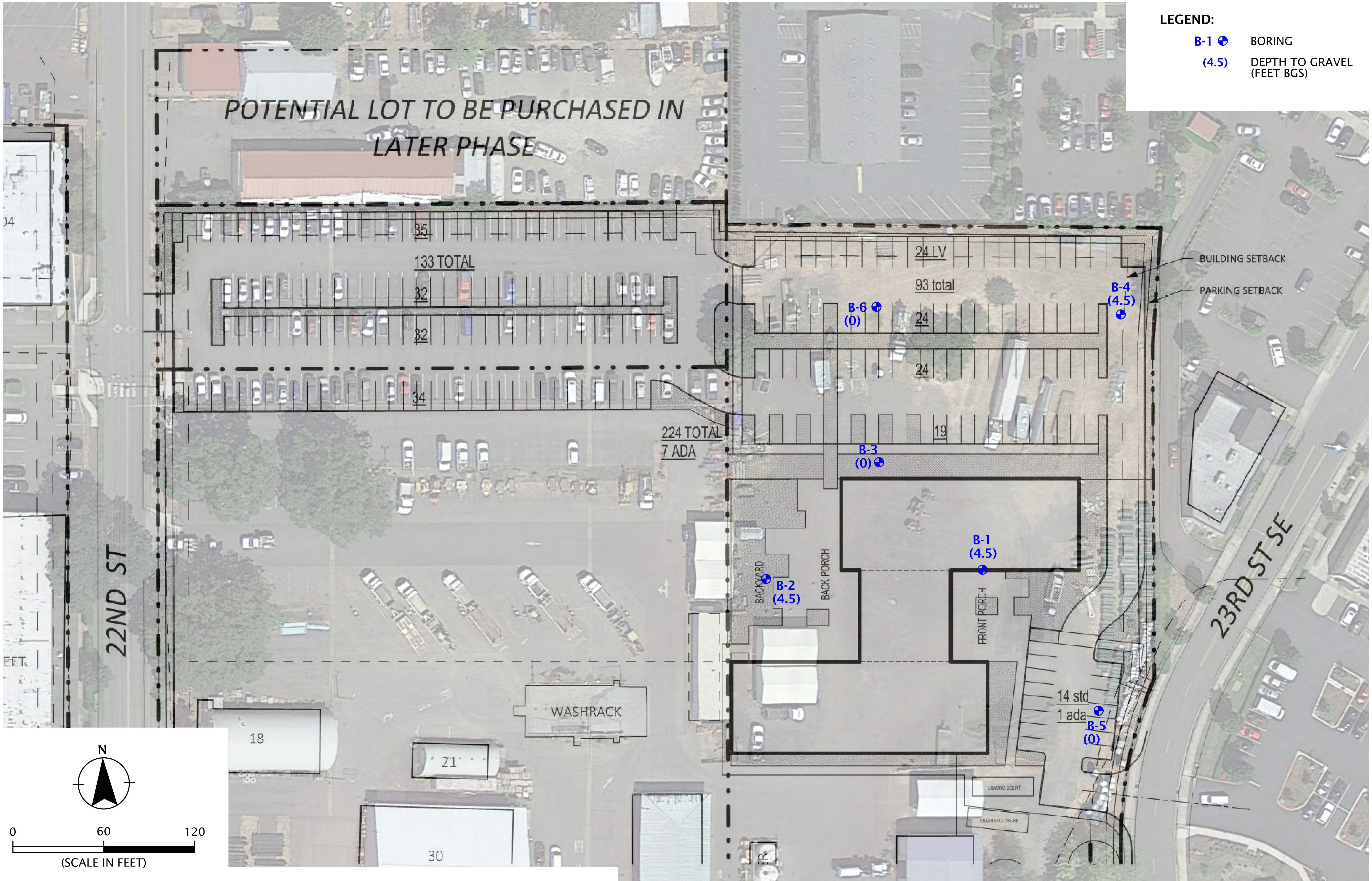
File Name: J:\S-Z\Salem\Salem-59\Salem-59-01\Figures\CAD\Salem-59-01-VM01.dwg | Layout: FIGURE 1



NOTES:

1. SITE PLAN BASED ON IMAGE OF SITE PLAN PHASE 01 PREPARED AND PROVIDED BY HACKER APRIL 19, 2021.
2. AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO APRIL 1, 2021.

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MAY 2021

SITE PLAN

SALEM PUBLIC WORKS OPERATIONS BUILDING
SALEM, OR

FIGURE 2

Printed By: mmiller | Print Date: 5/19/2021 12:00:54 PM
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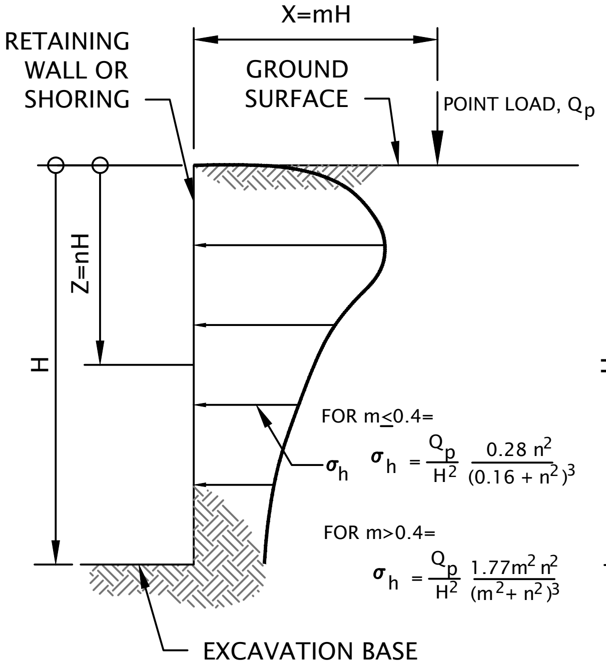
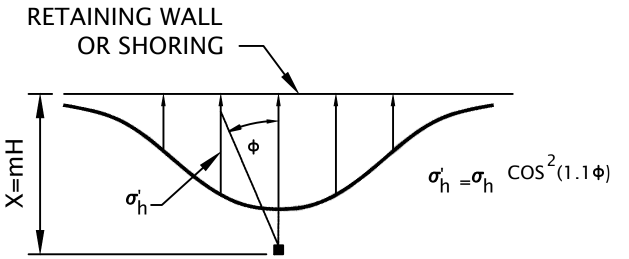


FOR m≤0.4=

$$\sigma_h = \frac{Q_p}{H^2}\,\frac{0.28\,n^2}{(0.16 + n^2)^3}$$

FOR m>0.4=

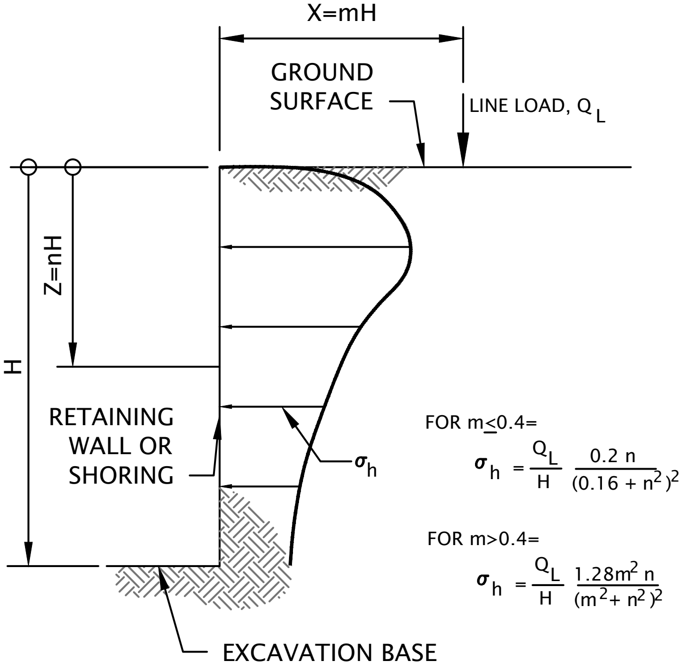
$$\sigma_h = \frac{Q_p}{H^2}\,\frac{1.77m^2\,n^2}{(m^2 + n^2)^3}$$



$$\sigma'_h = \sigma_h \; \cos^2(1.1\phi)$$

DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD



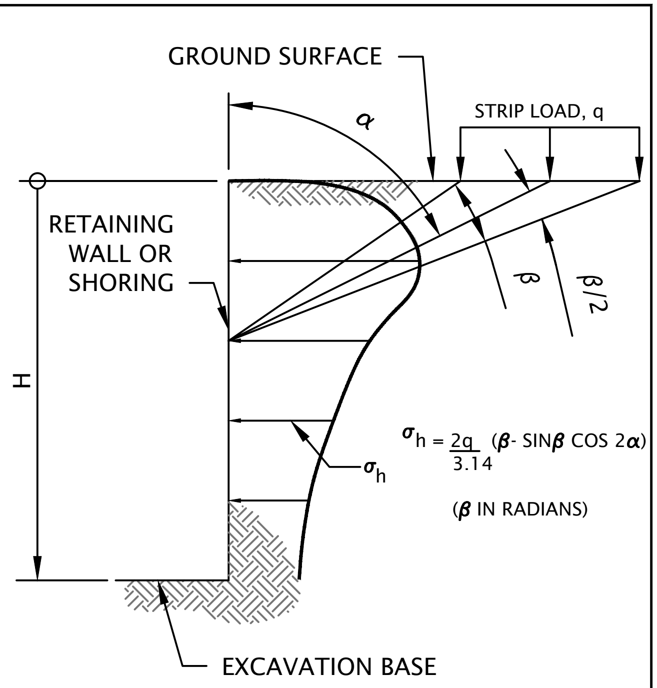
FOR m≤0.4=

$$\sigma_h = \frac{Q_L}{H}\,\frac{0.2\,n}{(0.16 + n^2)^2}$$

FOR m>0.4=

$$\sigma_h = \frac{Q_L}{H}\,\frac{1.28m^2\,n}{(m^2 + n^2)^2}$$

LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{3.14}\;(\beta - \sin\beta\,\cos 2\alpha)$$

(β IN RADIANS)

STRIP LOAD PARALLEL TO WALL

NOT TO SCALE

NOTES:

1. FIGURE SHOULD BE USED IN CONJUNCTION WITH REPORT TEXT.
2. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.
3. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

GEODESIGN INC
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SALEM-59-01

MAY 2021

SURCHARGE-INDUCED LATERAL EARTH PRESSURES

SALEM PUBLIC WORKS OPERATIONS BUILDING
SALEM, OR

FIGURE 3

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We conducted a subsurface exploration program that consisted of drilling six borings (B-1 through B-6) at the approximate locations shown on Figure 2. The borings were drilled to depths between 8.4 and 21.4 feet BGS. The borings were drilled using hollow-stem auger drilling methods. Drilling services were provided by Western States Soil Conservation, Inc. of Hubbard, Oregon, on March 17 and 23, 2021. The explorations were observed and logged by a member of our geology staff. We collected representative samples of the various soil encountered in the explorations for visual classification and laboratory testing. The exploration logs are presented in this appendix.

The exploration locations were marked in the field using visual references. The exploration locations should be considered accurate only to the degree implied by the methods used. We estimated the exploration elevations using a topographic map of the site that was provided to us by the project civil engineer.

SOIL SAMPLING

We collected soil samples from the borings using SPTs performed in general conformance with ASTM D1586. The sampler was driven with a 140-pound automatic trip hammer free-falling 30 inches. The number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soil is shown adjacent to the sample symbols on the exploration logs. Disturbed samples were collected from the split barrel for subsequent classification and index testing. Sampling methods and intervals are shown on the exploration logs.

The average efficiency of the automatic SPT hammers used by Western States Soil Conservation, Inc. was 78.4 or 69.2 percent, depending on the drill rig used. The calibration testing results are presented at the end of this appendix.

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 appendix. The exploration logs indicate the depths at which the soils or their characteristics change, although the change could be gradual. A horizontal line between soil types indicates an observed (visual or digging action) change. If the change occurred between sample locations and was not observed or obvious, the depth was interpreted and the change is indicated using a dashed line. Classifications are shown on the exploration logs.

LABORATORY TESTING

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

MOISTURE CONTENT

We determined the natural moisture content of select soil samples in general accordance with ASTM D2216. The natural moisture content is the 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 appendix.

PARTICLE-SIZE ANALYSIS

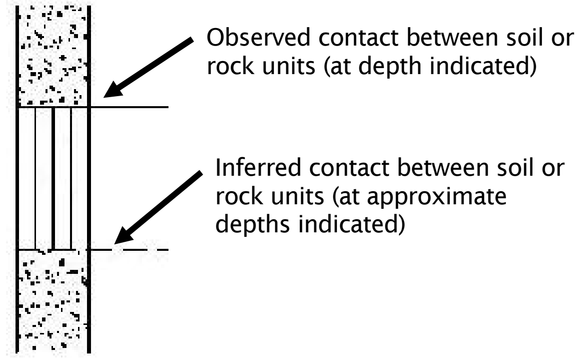
We completed particle-size analysis on select soil samples in general accordance with ASTM D1140. The testing consisted of determining the soil percentages passing various U.S. Standard sieves. The percent fines is the ratio of the dry weight of the material passing the U.S. Standard No. 200 sieve to the dry weight of the overall sample. The test results are presented in this appendix.

ATTERBERG LIMITS TESTING

We determined the Atterberg limits of select soil samples in general accordance with ASTM D4318. Atterberg limits include the liquid limit, plastic limit, and the plasticity index of soil. These index properties are used to classify soil and for correlation with other engineering properties of soil. 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 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-O.D. 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

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	Standard Penetration 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 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.					
	Silt and Clay In:			Sand and Gravel In:	
Percent	Fine-Grained Soil	Coarse-Grained Soil	Percent	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 %

GEODESIGN INC
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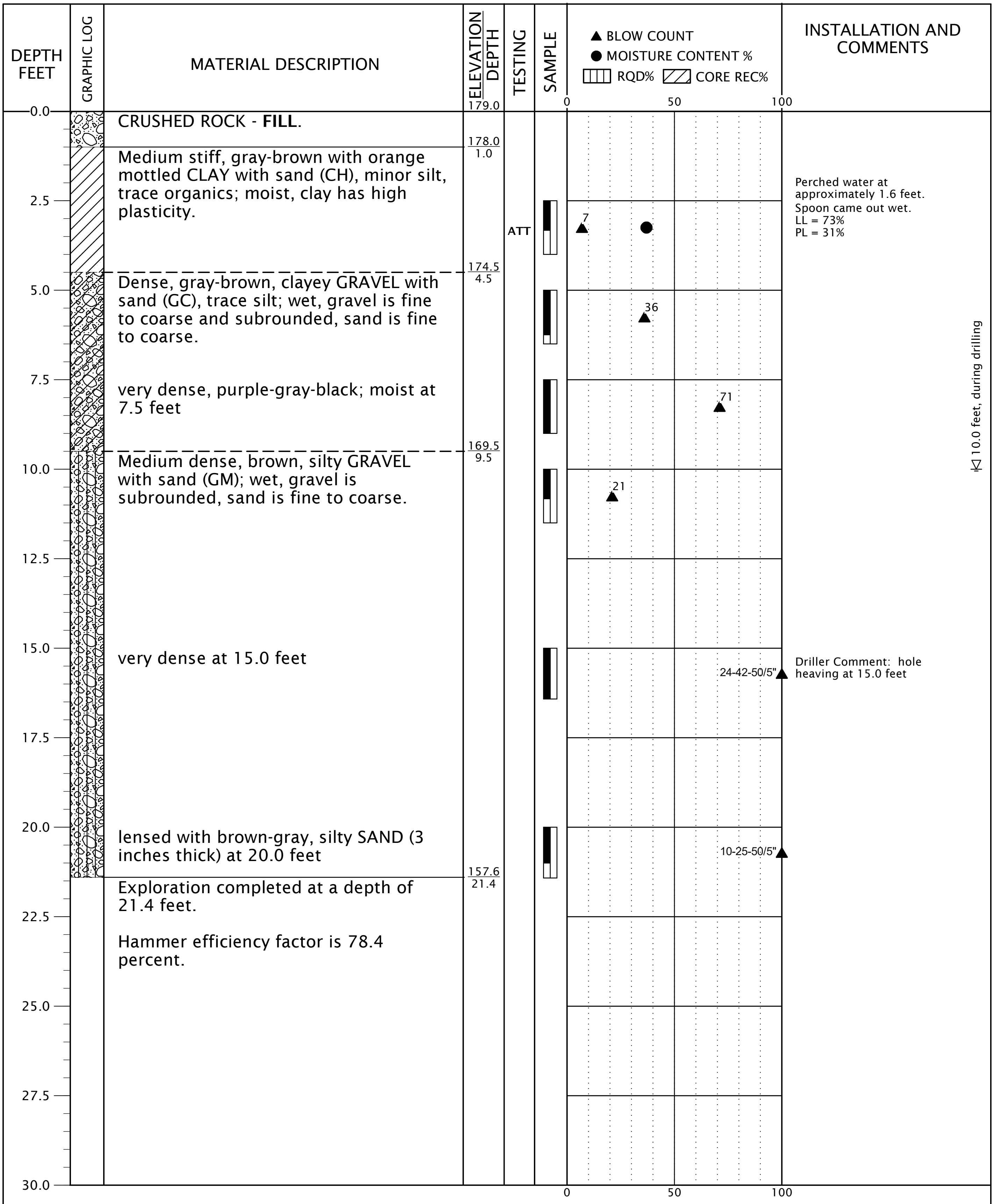
SOIL CLASSIFICATION SYSTEM

TABLE A-2



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	CRUSHED ROCK - **FILL**.	179.0			
1.0	Medium stiff, gray-brown with orange mottled CLAY with sand (CH), minor silt, trace organics; moist, clay has high plasticity.	178.0 1.0	ATT	7	Perched water at approximately 1.6 feet. Spoon came out wet. LL = 73% PL = 31%
4.5	Dense, gray-brown, clayey GRAVEL with sand (GC), trace silt; wet, gravel is fine to coarse and subrounded, sand is fine to coarse.	174.5 4.5		36	
7.5	very dense, purple-gray-black; moist at 7.5 feet			71	
9.5	Medium dense, brown, silty GRAVEL with sand (GM); wet, gravel is subrounded, sand is fine to coarse.	169.5 9.5		21	⊻ 10.0 feet, during drilling
15.0	very dense at 15.0 feet			24-42-50/5"	Driller Comment: hole heaving at 15.0 feet
20.0	lensed with brown-gray, silty SAND (3 inches thick) at 20.0 feet			10-25-50/5"	
21.4	Exploration completed at a depth of 21.4 feet. Hammer efficiency factor is 78.4 percent.	157.6 21.4			

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: H. Herinckx
COMPLETED: 03/17/21

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

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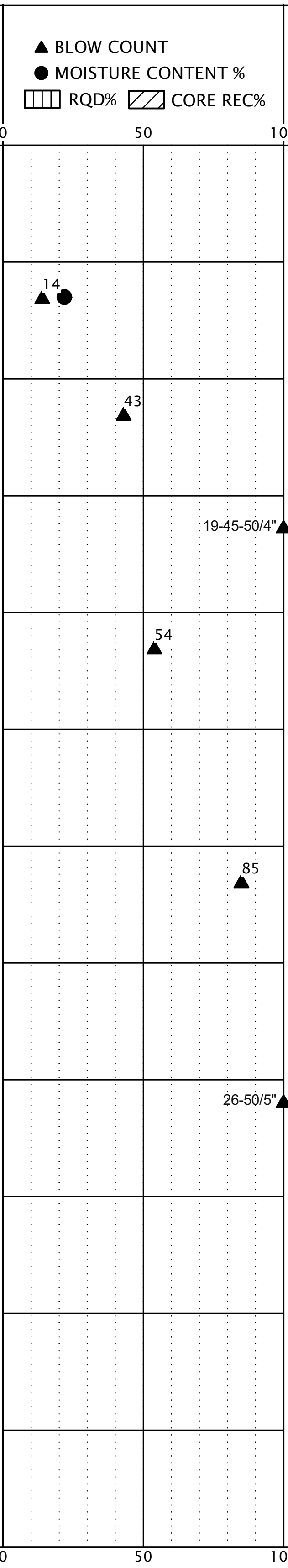
MAY 2021

BORING B-1

SALEM PUBLIC WORKS OPERATIONS BUILDING
SALEM, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT / MOISTURE CONTENT %	INSTALLATION AND COMMENTS
0.0	CRUSHED ROCK with asphalt concrete chunks - **FILL**.	179.0			
	Stiff, brown-gray with orange mottled CLAY with sand (CH), minor silt and gravel, trace organics; moist, clay has high plasticity, sand is fine to coarse.	178.0 1.0			
2.5			ATT	14	LL = 71% PL = 29%
	Dense, brown with orange mottled, clayey GRAVEL with sand (GC), trace silt; moist, gravel is fine to coarse and subrounded, sand is fine to coarse.	174.5 4.5			
5.0				43	
	Very dense, brown, silty GRAVEL with sand (GM); moist, gravel is fine to coarse and subrounded, sand is fine to coarse.	172.0 7.0			
7.5				19-45-50/4"	
10.0	moist to wet at 10.0 feet			54	∇ 10.0 feet, during drilling
12.5					
15.0				85	
17.5					
20.0	interbedded silty SAND (0.25 inch thick) at 20.0 feet			26-50/5"	
	Exploration completed at a depth of 20.9 feet. Hammer efficiency factor is 69.2 percent.	158.1 20.9			
22.5					
25.0					
27.5					
30.0					



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: H. Herinckx
COMPLETED: 03/23/21

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

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BORING B-2

SALEM PUBLIC WORKS OPERATIONS BUILDING
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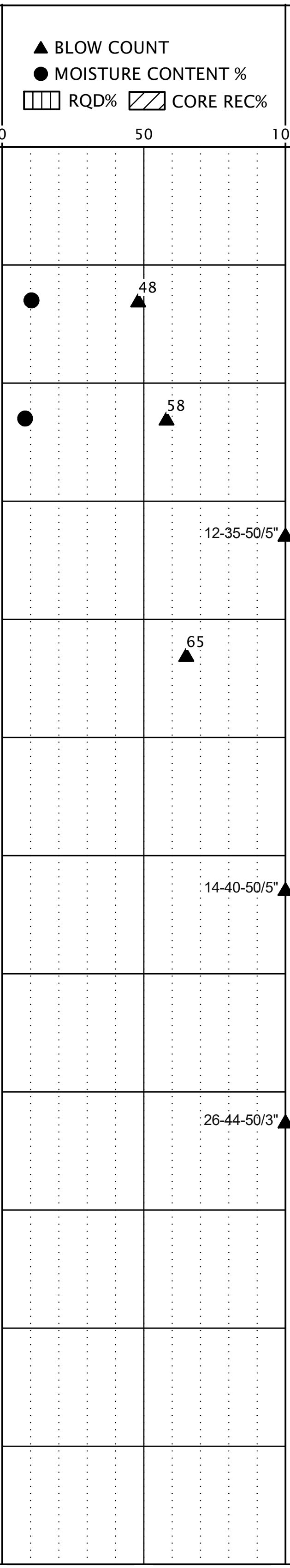
FIGURE A-2



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	CRUSHED ROCK - **FILL**.	179.0			
1.0	Dense, brown with orange mottled, clayey GRAVEL with sand (GC), trace silt and organics; moist, gravel is fine to coarse and subrounded, sand is fine to coarse.	178.0 1.0			
2.5			P200	48	P200 = 16%
4.5	Very dense, brown, silty GRAVEL with sand (GM); moist, gravel is fine to coarse and subrounded to subangular.	174.5 4.5			
5.0			P200	58	P200 = 12%
7.5				12-35-50/5"	
10.0	moist to wet at 10.0 feet			65	
13.0					⊻ 13.0 feet, during drilling
15.0	brown with yellow mottles; wet, interbedded silty SAND at 15.0 feet			14-40-50/5"	
20.0	without interbedded silty SAND at 20.0 feet			26-44-50/3"	
21.3	Exploration completed at a depth of 21.3 feet. Hammer efficiency factor is 69.2 percent.	157.7 21.3			

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: H. Herinckx

COMPLETED: 03/23/21

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

BORING BIT DIAMETER: 4 1/4 inches

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BORING B-3

SALEM PUBLIC WORKS OPERATIONS BUILDING
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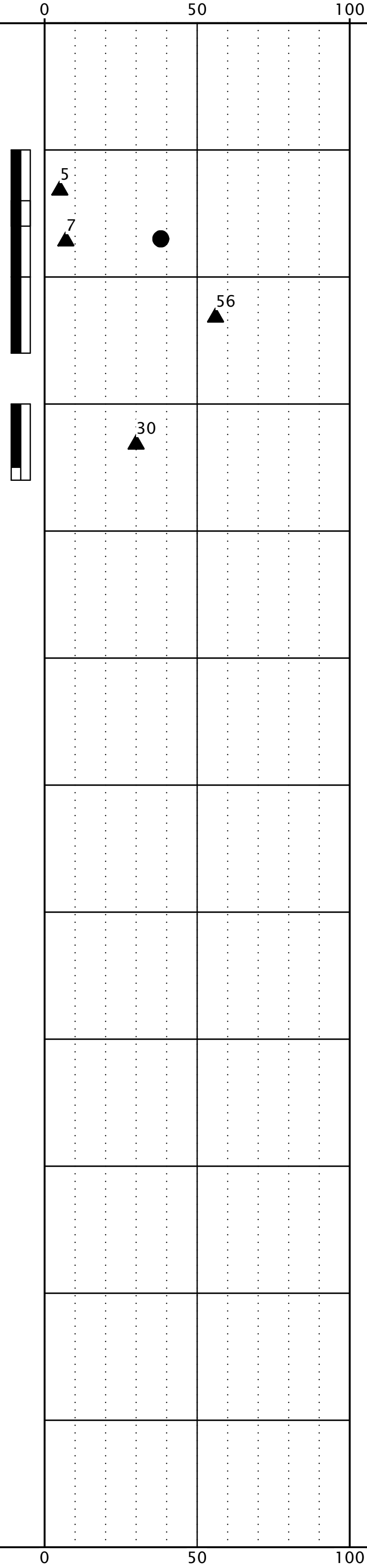
FIGURE A-3

BORING LOG - GDI-NV5 - 1 PER PAGE SALEM-59-01-B1_6.GPJ GDI_NV5.GDT PRINT DATE: 5/19/21:KT

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

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Medium stiff, brown-orange with orange mottled, sandy SILT (ML), minor clay, trace gravel and organics; moist, sand is fine, gravel is fine (3.5-inch-thick root zone).	179.0		Grass at the ground surface.
2.5				Infiltration test at 3.5 feet. P200 = 66%
5.0	Very dense, brown, silty GRAVEL with sand (GM), trace organics; moist, gravel is fine to coarse and subrounded to subangular, sand is fine to coarse.	174.5 4.5	P200	
7.5	medium dense to dense at 7.5 feet			More moisture at 8.5 feet. Left hole open for approximately 20 minutes. No measurable water.
10.0	Exploration completed at a depth of 9.0 feet. Hammer efficiency factor is 78.4 percent.	170.0 9.0		
12.5				
15.0				
17.5				
20.0				
22.5				
25.0				
27.5				
30.0				



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: H. Herinckx
COMPLETED: 03/17/21

BORING METHOD: hollow-stem auger (see document text)
BORING BIT DIAMETER: 6 1/4 inches

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BORING B-4

SALEM PUBLIC WORKS OPERATIONS BUILDING
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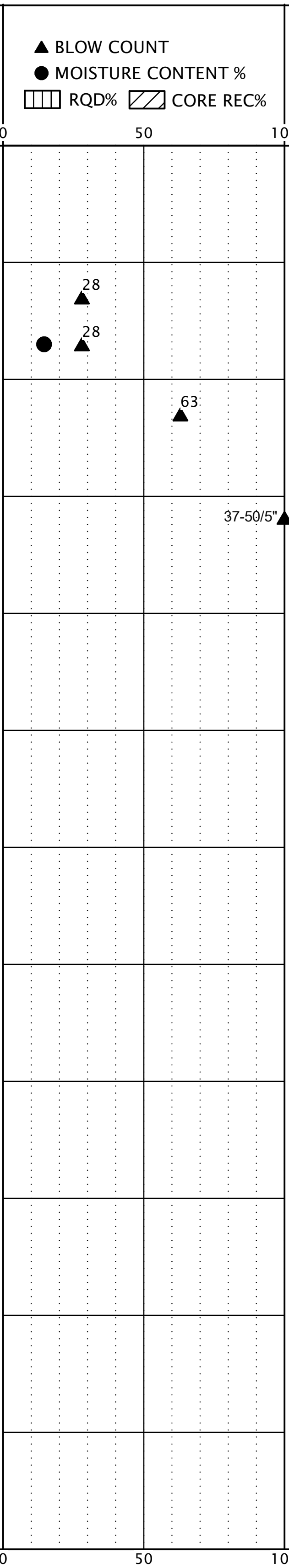
FIGURE A-4

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	CRUSHED ROCK - **FILL**.	179.0		
1.0	Medium dense, brown with orange mottled, clayey GRAVEL with sand (GC), trace silt and organics; moist, gravel is fine to coarse and subrounded to subangular, sand is fine to coarse.	178.0 1.0		
3.5			P200	Infiltration test at 3.5 feet. P200 = 18%
5.0	very dense at 5.0 feet			
8.4	Exploration completed at a depth of 8.4 feet. Hammer efficiency factor is 78.4 percent.	170.6 8.4		



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: H. Herinckx
COMPLETED: 03/17/21

BORING METHOD: hollow-stem auger (see document text)
BORING BIT DIAMETER: 6 1/4 inches

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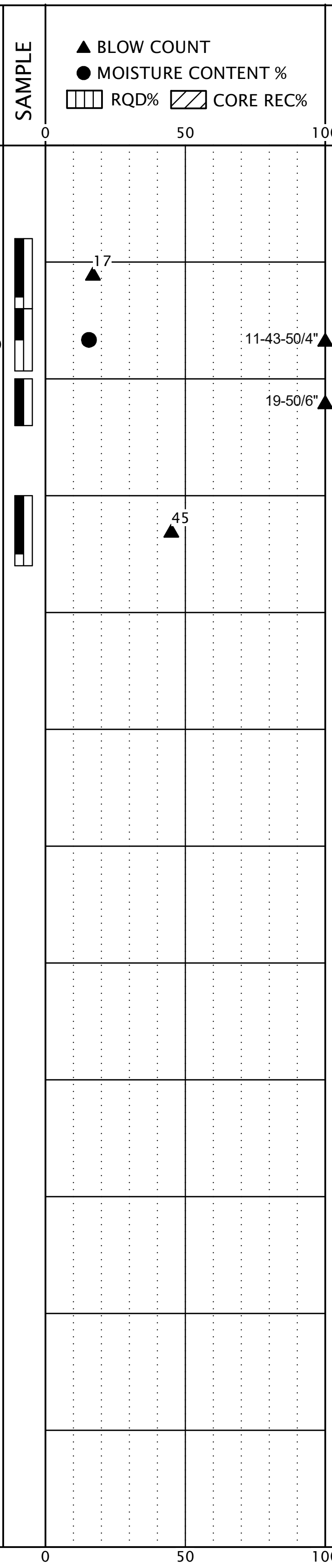
BORING B-5

SALEM PUBLIC WORKS OPERATIONS BUILDING
SALEM, OR

FIGURE A-5

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Medium dense, brown, silty GRAVEL with sand (GM), trace organics; moist, gravel is fine to coarse and subrounded to subangular, sand is fine to coarse (3-inch-thick root zone).	178.0		Grass at the ground surface.
2.5				At 2.5 feet, no recovery. Blow counts of 5/17/30. Pushed SPT at 2.0 feet when drilling for infiltration test.
	very dense at 3.5 feet		P200	Infiltration test at 3.5 feet. P200 = 16%
5.0				
7.5	dense at 7.5 feet			
	Exploration completed at a depth of 9.0 feet. Hammer efficiency factor is 78.4 percent.	169.0 9.0		
10.0				
12.5				
15.0				
17.5				
20.0				
22.5				
25.0				
27.5				
30.0				



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: H. Herinckx

COMPLETED: 03/17/21

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

BORING BIT DIAMETER: 6 1/4 inches

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BORING B-6

SALEM PUBLIC WORKS OPERATIONS BUILDING
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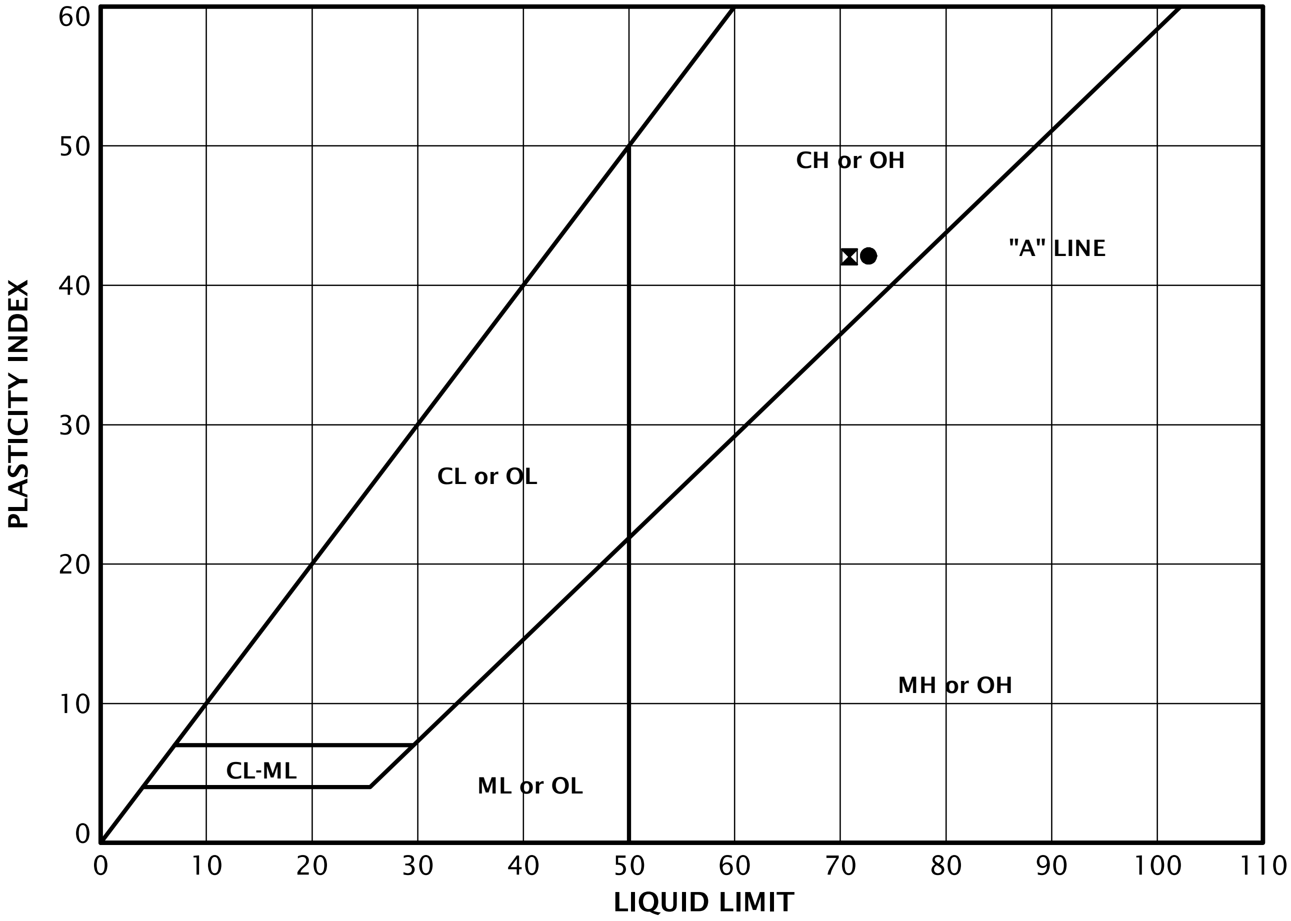
FIGURE A-6



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-1	2.5	37	73	31	42
⧗	B-2	2.5	22	71	29	42

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ATTERBERG LIMITS TEST RESULTS

SALEM PUBLIC WORKS OPERATIONS BUILDING
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FIGURE A-7

ATTERBERG_LIMITS 7 SALEM-59-01-B1_6.GPJ GEODESIGN.GDT PRINT DATE: 5/19/21:KT

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	176.5	37					73	31	42
B-2	2.5	176.5	22					71	29	42
B-3	2.5	176.5	10				16			
B-3	5.0	174.0	8				12			
B-4	3.5	175.5	38				66			
B-5	3.5	175.5	15				18			
B-6	3.5	174.5	16				16			

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SUMMARY OF LABORATORY DATA

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FIGURE A-8

Summary of SPT Test Results

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

EMX: Maximum Energy

ETR: Energy Transfer Ratio - Rated

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

Summary of SPT Test Results

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

EMX: Maximum Energy

ETR: Energy Transfer Ratio - Rated

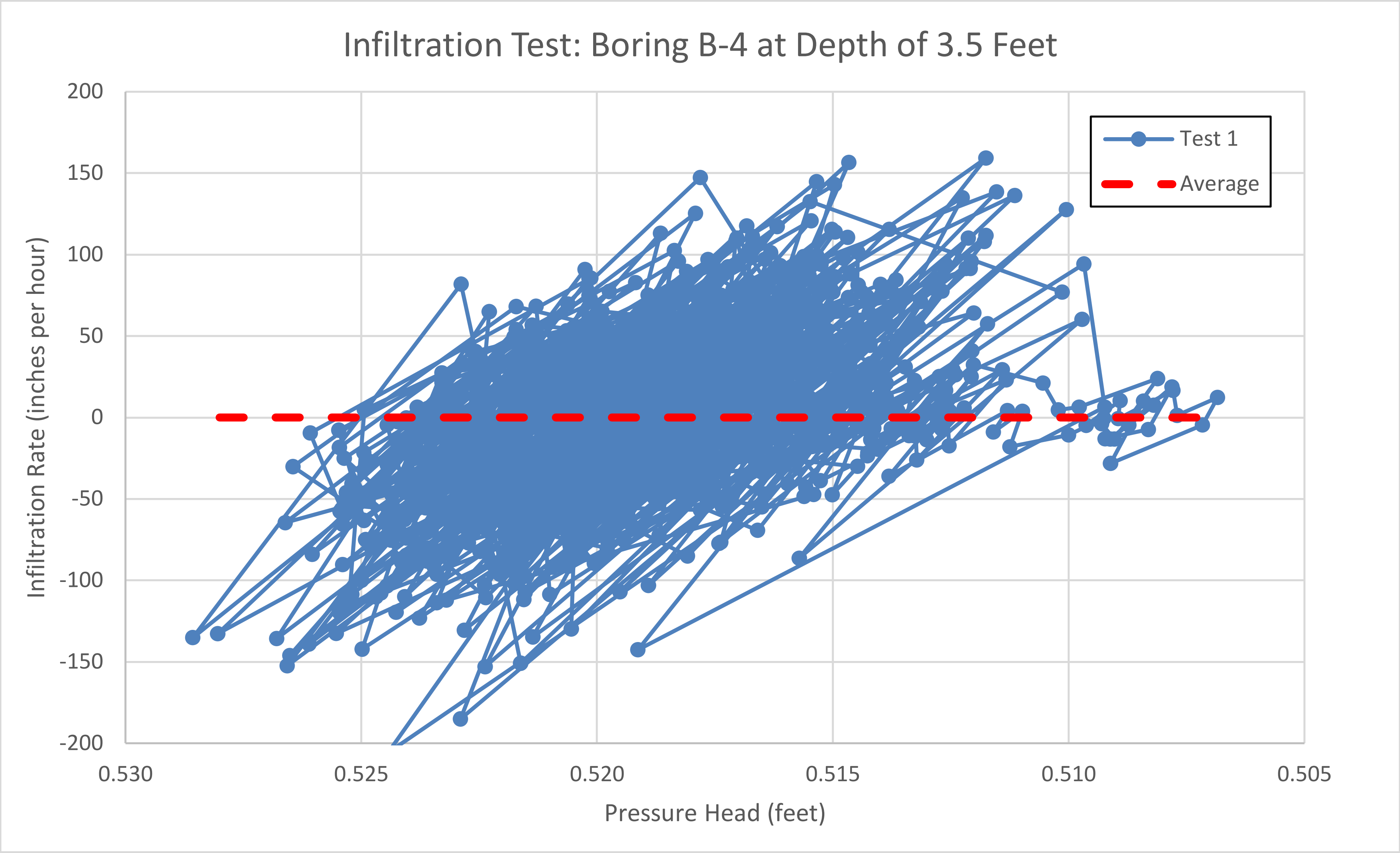
Start Depth ft	Final Depth ft	**N Value**	**N60 Value**	Average EMX ft-lb	Average ETR %
27.50	29.00	**30**	**34**	243.05	69.4
30.00	31.50	**26**	**29**	253.78	72.5
32.50	34.00	**29**	**33**	238.01	68.0
35.00	36.50	**37**	**42**	241.80	69.1
37.50	39.00	**44**	**50**	237.44	67.8
			Overall Average Values: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

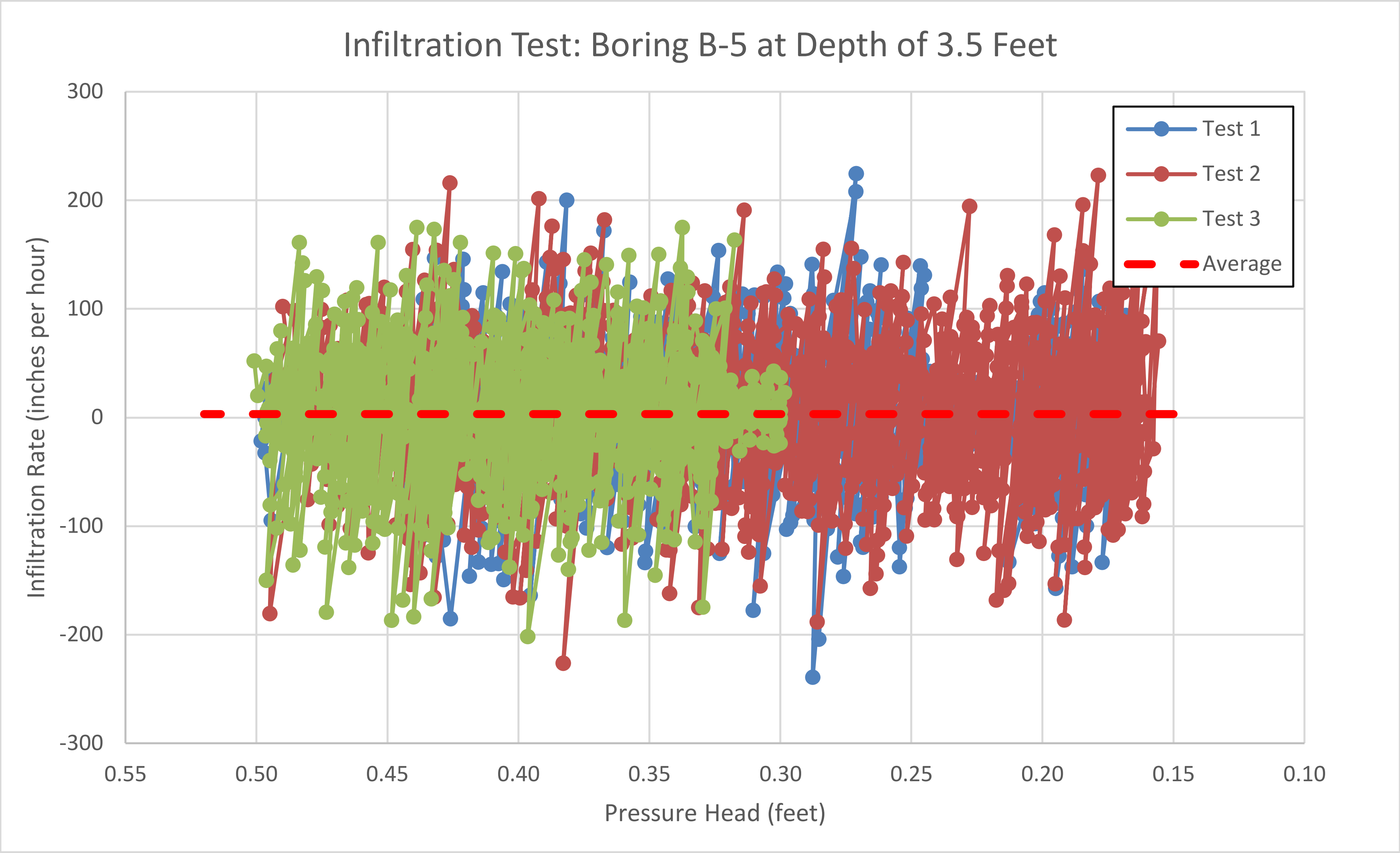
APPENDIX B

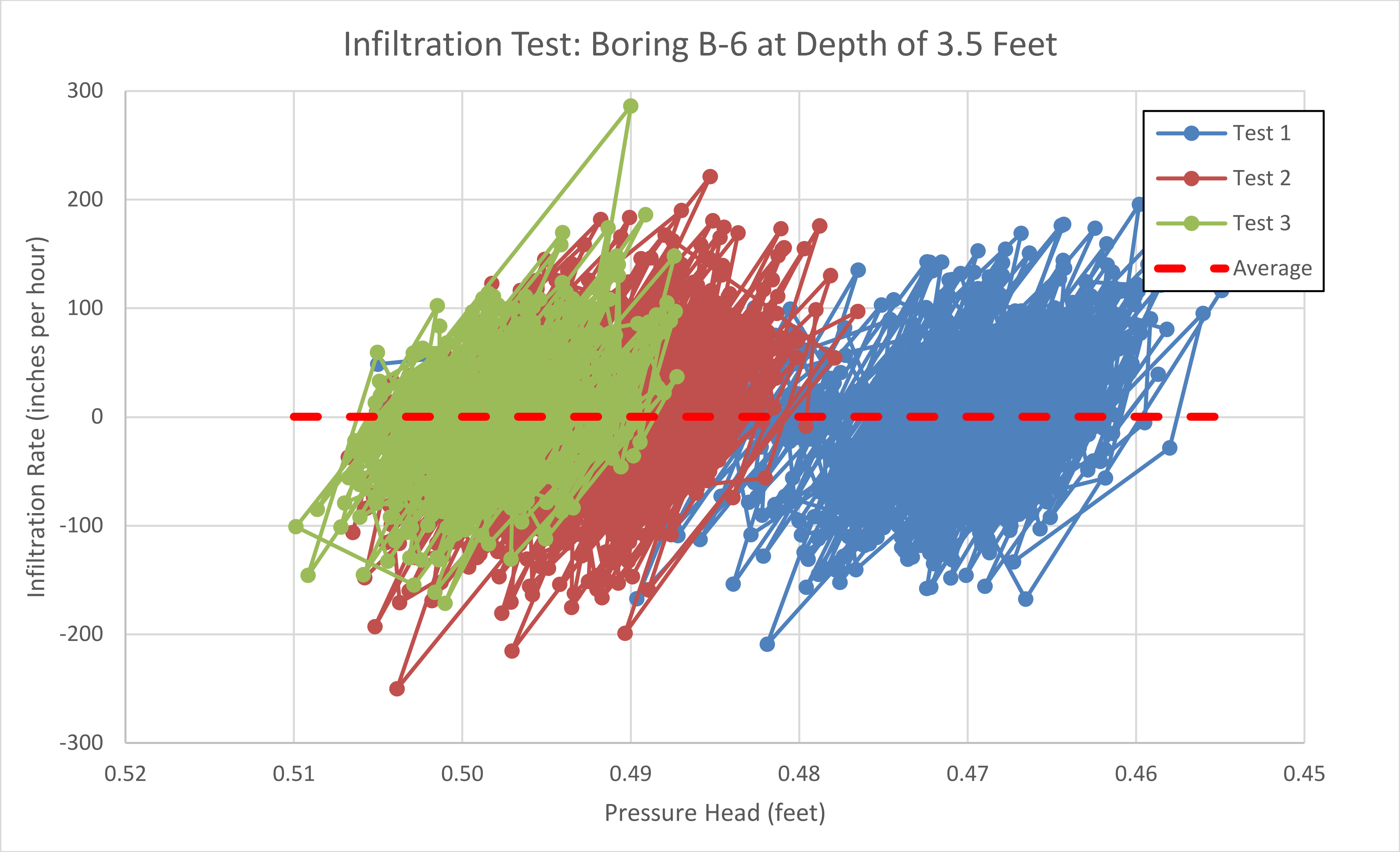
APPENDIX B

INFILTRATION TEST DATA

Plots showing the infiltration tests we performed on the site are presented in this appendix. We performed the infiltration tests inside hollow-stem augers using the encased falling head test method. We performed the tests with a water head of approximately 6 inches. We collected water level readings using an electronic water level data logger. The apparent scatter in the data is due to the high frequency of readings collected by the data logger. We added a trend line to the plots to show the average of the data.


Infiltration Test: Boring B-4 at Depth of 3.5 Feet
200
150
100
50
0
-50
-100
-150
-200
Infiltration Rate (inches per hour)
0.530
0.525
0.520
0.515
0.510
0.505
Pressure Head (feet)
Test 1
Average


Infiltration Test: Boring B-5 at Depth of 3.5 Feet
300
200
100
0
-100
-200
-300
Infiltration Rate (inches per hour)
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
Pressure Head (feet)
Test 1
Test 2
Test 3
Average


Infiltration Test: Boring B-6 at Depth of 3.5 Feet
Infiltration Rate (inches per hour)
300
200
100
0
-100
-200
-300
0.52
0.51
0.50
0.49
0.48
0.47
0.46
0.45
Pressure Head (feet)
Test 1
Test 2
Test 3
Average

APPENDIX C

APPENDIX C

SITE-SPECIFIC SEISMIC HAZARD EVALUATION

INTRODUCTION

The information in this appendix summarizes the results of our site-specific seismic hazard study for the proposed Salem Public Works Operations Building. The proposed project includes the construction of a new two-story, at-grade office building. This seismic hazard evaluation was performed in accordance with the requirements of the 2019 SOSSC (Section 1803.6.1).

SITE CONDITIONS

REGIONAL GEOLOGY

A detailed description of the regional geology is presented in the main report.

GEOLOGIC HAZARDS

A discussion of potential seismic hazards that could affect the proposed project is presented in the main report.

SURFACE AND SUBSURFACE CONDITIONS

Detailed descriptions of the site surface and subsurface conditions are presented in the main report.

SEISMIC SETTING

Earthquake Source Zones

Three earthquake scenarios were considered for this study that are consistent with the local seismic setting. Two of the possible earthquake sources are associated with the CSZ, and the third event is a shallow, local crustal earthquake that could occur in the North American Plate. The three earthquake scenarios are discussed below.

Regional Events

The CSZ is the region where the Juan de Fuca Plate is being subducted beneath the North American Plate. This subduction is occurring in the coastal region between Vancouver Island and northern California. Evidence has accumulated suggesting that this subduction zone has generated eight great earthquakes in the last 4,000 years, with the most recent event occurring approximately 300 years ago (Weaver and Shedlock, 1991). The fault trace is mapped approximately 50 to 120 km off the Oregon Coast. Two types of subduction zone earthquakes are possible and considered in this study:

1. An interface event earthquake on the seismogenic part of the interface between the Juan de Fuca Plate and the North American Plate on the CSZ. This source is reportedly capable of generating earthquakes with a moment magnitude of between 8.5 and 9.0.

2. A deep intraplate earthquake on the seismogenic part of the subducting Juan de Fuca Plate. These events typically occur at depths between 30 and 60 km. This source is capable of generating an event with a moment magnitude of up to 7.5.

Local Events

A significant earthquake could occur on a local fault near the site within the design life of the facility. Such an event would cause ground shaking at the site that could be more intense than the CSZ events, although the duration would be shorter. Figure C-1 shows the locations of faults with potential Quaternary movement within a 35-km radius of the site (USGS, 2020). Figure C-2 shows the interpreted locations of seismic events that occurred between 1904 and 2021 (USGS, 2021a). The most significant faults in the site vicinity are the Waldo Hills fault, Salem-Eola Hills homocline, and the Turner and Mill Creek faults. A discussion of these faults is provided below and a summary of these faults is presented in Table C-1.

Waldo Hills Fault

The northeast-striking, southeast-dipping Waldo Hills reverse fault offsets Miocene rocks of the CRBG along the northwestern margin of the Waldo Hills in the central Willamette Valley. The Waldo Hills fault is coincident with a steep, linear range front that marks the northwestern margin of the Waldo Hills and the eastern margin of the central Willamette Valley, but no fault scarps on surficial quaternary deposits have been described along its trace. The Waldo Hills fault has a mapped length of 12 km. The Waldo Hills fault is mapped as a high-angle, normal fault with a northwest dip direction. The fault slip rate category is less than 0.2 millimeters per year (Personius, 2002a).

Salem-Eola Hills Homocline

The northwest-striking Salem-Eola Hills homocline deforms Miocene rocks of the CRBG along the Salem Hills and Eola Hills in the central Willamette Valley, at the southwestern margin of deposition of these rocks in this part of Oregon. In the late Miocene, the fold acted as a tectonic dam, causing the obstruction of the ancestral Willamette River and deposition of a thick sequence of basin-fill sediment in the southern Willamette Valley. Older undated gravels of probable Quaternary age that occupy a bedrock channel in the Salem water gap slope northward approximately 25 times steeper than the present channel of the Willamette River; this increase in slope probably reflects uplift or faulting in the Salem Hills and Eola Hills. A broad convexity in the modern channel profile of the Willamette River that is roughly coincident with the location of the Salem and Eola Hills may also be caused by deformation on the Salem-Eola Hills homocline, but the channel convexity could also be caused by differential channel incision due to varying channel lithology and be unrelated to ongoing tectonism. The Salem-Eola Hills homocline has a mapped length of 32 km. The Salem-Eola Hills homocline is mapped as a northeast-dipping homocline or monocline. The fault slip rate category is less than 0.2 millimeters per year (Personius, 2002b).

Turner and Mill Creek Faults

The northeast-striking Turner and Mill Creek faults offsets Miocene rocks of the CRBG in the Salem Hills and Waldo Hills of the central Willamette Valley. This fault has the same strike and displacement direction as the Corvallis fault, but there is no evidence that these structures are continuous across the Willamette River. The Mill Creek fault is coincident with a gentle, embayed range front along the southern margin of the Waldo Hills and may deform middle Pleistocene (?) deposits near the Mill Creek water gap. The fault was originally mapped and named as two separate structures, the Turner and Mill Creek faults. The Turner fault is named after the town of Turner in the Salem Hills. The Mill Creek fault is named after Mill Creek, which parallels part of the fault trace. Several recent studies include both faults as a single fault. The Turner and Mill Creek faults have a mapped length of 18 km. The Mill Creek fault is mapped as a near-vertical fault and basin relations suggest a strong component of left-lateral strike slip. The fault slip rate category is less than 0.2 millimeters per year (Personius, 2002c).

Table C-1. Significant Crustal Faults Within the Site Vicinity

Fault Name	Proximity to Site (km)	Estimated Displacement Description	Estimated Age
Waldo Hills	5.3	May deform older Quaternary deposits, but the fault does not appear to deform middle or late Quaternary deposits	Quaternary (<1.6 million years before present)
Salem-Eola Hills Homocline	8.2	May deform older Quaternary deposits along the Salem water gap and also may deform the modern channel of the Willamette River	Quaternary (<1.6 million years before present)
Turner and Mill Creek	11.3	Faults may deform older Quaternary deposits, but do not appear to deform late Quaternary surfaces	Quaternary (<1.6 million years before present)

DESIGN EARTHQUAKE

Deaggregation at the approximate fundamental building period of 0.2 second using the USGS Unified Hazard tool (USGS, 2021b [latitude = 44.923307, longitude = -123.015912]) indicates the CSZ comprises approximately 67 percent of the seismic hazard at the site. The remaining 33 percent is comprised local events and the deep intraplate events. All of the local faults contribute less than 1 percent to the overall seismic hazard.

SEISMIC DESIGN PARAMETERS

We determined the seismic site class using shear wave velocities that were measured in similar soil conditions approximately 1.5 miles southeast of the site at the intersection of Interstate 5 and the North Santiam Highway in Salem (Mabey and Madin, 1992). A copy of the published shear wave velocity profile we used is shown on Figure 5 of the article that is presented in Appendix D.

Based on our calculations, the site can be classified as seismic Site Class C. The calculations we used to determine the site class are provided in Table C-2.

Table C-2. Site Class Determination

Soil Type	Depth Below Foundation[1] (meters)	Interval (meters)	Shear Wave Velocity (mps)	Interval/Shear Wave Velocity (second)
Gravel[1]	5	5	375	0.013333
Gravel	7	2	450	0.004444
Gravel	9	2	225	0.008889
Gravel	11	2	400	0.005000
Gravel	13	2	600	0.003333
Gravel	15	2	740	0.002703
Gravel	17	2	350	0.005714
Gravel	19	2	500	0.004000
Gravel	21	2	500	0.004000
Gravel	23	2	500	0.004000
Gravel	25	2	200	0.010000
Gravel	27	2	200	0.010000
Gravel	29	2	550	0.003636
Gravel	30.5	1.5	475	0.003158
Sum	NA	30.5	NA	0.082211
Average shear wave velocity in the upper 100 feet below the foundation, Vs_{30} (mps)	NA			371
Site Class	NA			C

[1] Assumes base of foundations is at the existing ground surface elevation.

In our opinion, amplification factors prescribed by ASCE 7-16 for a seismic Site Class C are appropriate for design and a site-response analysis is not required. The parameters in Table C-3 can be used for design of the proposed building. These parameters were obtained from the SEAOC/OSHPD seismic design map tool (SEAOC/OSHPD, 2021).

Table C-3. Seismic Design Parameters per ASCE 7-16

Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
Spectral Acceleration (MCE)	S_s = 0.816 g	S_1 = 0.410 g
Site Class	C	
Site Coefficient	F_a = 1.200	F_v = 1.500
Spectral Acceleration Parameters	S_{MS} = 0.980 g	S_{M1} = 0.615 g
Design Spectral Acceleration Parameters	S_{DS} = 0.653 g	S_{D1} = 0.410 g
Spectral PGA	0.379 g	
Design Spectral PGA	0.253 g	
MCE_G PGA Adjusted for Site Class Effects[1]	PGA_M = 0.455 g	

1. From ASCE 7-16. Minimum PGA value to use when evaluating liquefaction and soil strength loss, as required by ASCE 7-16 Section 11.8.3.

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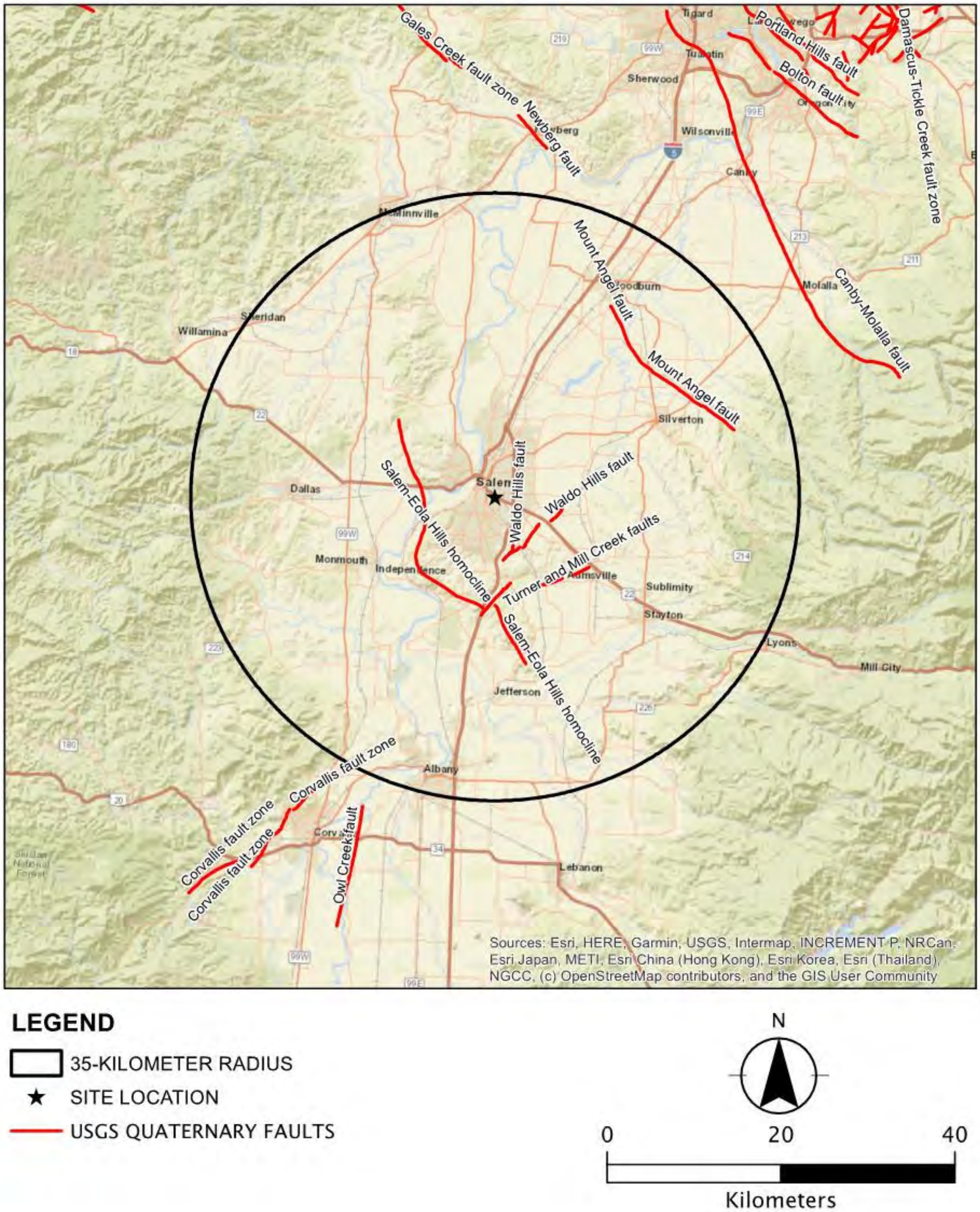

USGS, accessed November 10, 2020, Quaternary Fault and Fold Database of the United States, U.S. Geological Survey, Available: https://www.usgs.gov/natural-hazards/earthquake-hazards/faults

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QUATERNARY FAULT MAP

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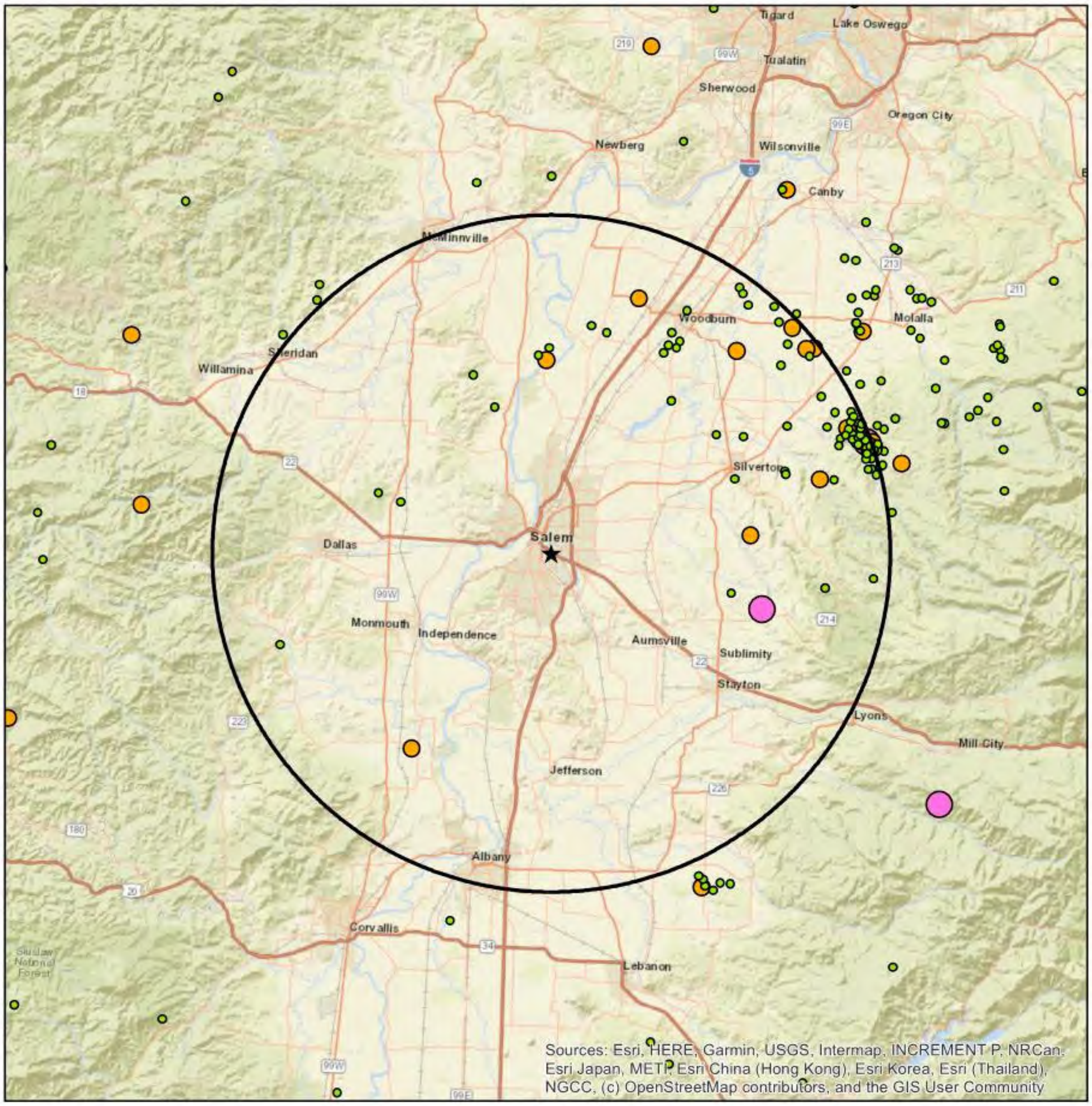
FIGURE C-1



Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

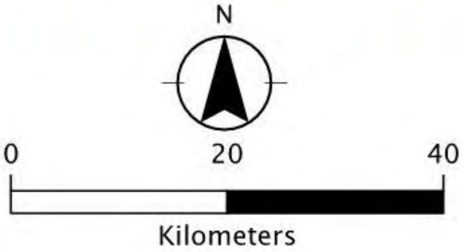
LEGEND

▭ 35-KILOMETER RADIUS

★ SITE LOCATION

INSTRUMENTAL EARTHQUAKE MAGNITUDE

- ● 2.0 - 3.0
- ● 3.0 - 4.0
- ● 4.0 - 6.0
- ● > 6.0



USGS, accessed April 19, 2021, Earthquake Hazards Program, US Earthquake Information by State, U.S. Geological Survey, Available: https://www.usgs.gov/natural-hazards/earthquake-hazards/earthquakes

Earthquakes in figure are from 1904-2021.

Salem-59-01-FC1_C2.docx Print Date: 5/19/21

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HISTORICAL SEISMICITY MAP

SALEM PUBLIC WORKS OPERATIONS BUILDING
SALEM, OR

FIGURE C-2

APPENDIX D

APPENDIX D

NEARBY SHEAR WAVE VELOCITY MEASUREMENT

This appendix includes the article we used to determine the seismic site class for the site. This article is from Oregon Geology, published by the Oregon Department of Geology and Mineral Industries, Volume 54, Number 3, May 1992. We used the shear wave velocity profile shown on Figure 5 of this article. This shear wave velocity profile was drilled approximately 1.5 miles southeast of the site in similar soil conditions.

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IN THIS ISSUE:

Shear wave velocity measurements in the Willamette Valley and the Portland Basin, Oregon

by Matthew A. Mabey and Ian P. Madin, Oregon Department of Geology and Mineral Industries

ABSTRACT

For the purpose of mapping the hazard represented by amplification of earthquake ground shaking by the sediment column, the Oregon Department of Geology and Mineral Industries (DOGAMI) has begun measuring shear wave velocities in the Willamette Valley and Portland Basin. These measurements are made by recording the time it takes a shear wave generated at the surface to reach a geophone located in a borehole. The shear wave velocity of unconsolidated sediments will be used to model how the sediments will respond to earthquake ground shaking.

INTRODUCTION

In order to fulfill its obligation to assess earthquake hazards in the state of Oregon, DOGAMI is developing hazard maps of areas in the state. The initial efforts are focused on the population centers in the Willamette Valley and Portland Basin. One hazard that is being mapped is the potential for amplification of ground shaking by the sediment column at a given site. A critical parameter for assessing the amount of amplification that takes place is the shear modulus of the soil. The shear wave velocity is one way to measure the shear modulus of a soil. The measurement of the travel time for shear waves generated at the surface down to a geophone located in a borehole gives a direct measurement of both average and interval shear wave velocities in the sediments between the source and receiver. These measurements have been made at seven sites so far. The shear wave velocity can then be used to develop a dynamic model of how the sediment column responds to ground shaking.

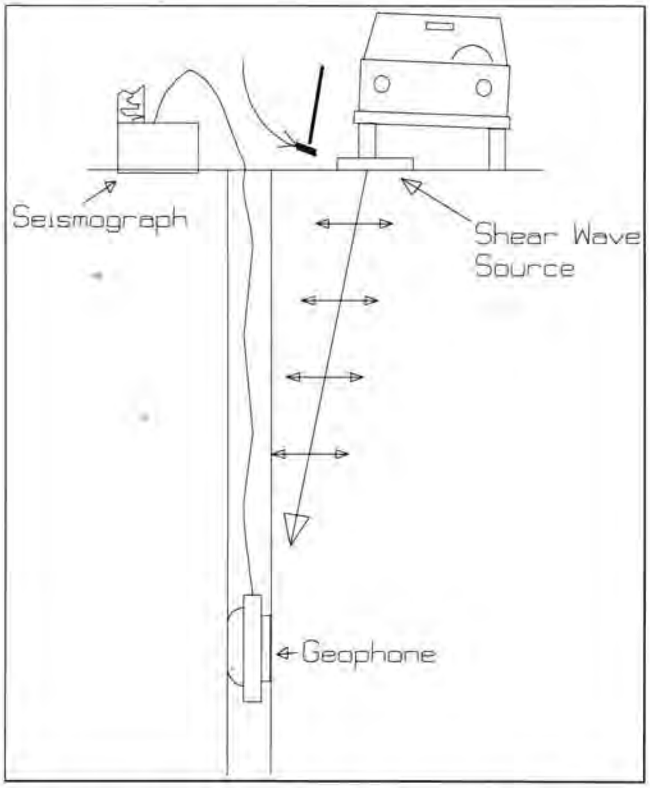

Figure 1. Diagram representing the procedure for collecting shear wave velocity data.

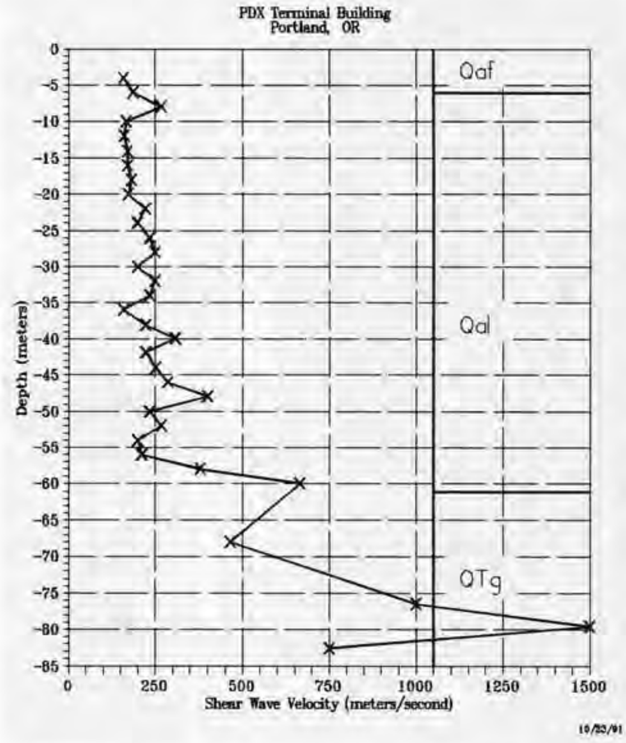

Figure 2. Shear wave velocity profile at Portland International Airport. This profile was measured in a hole immediately to the north of the main terminal building.

MEASUREMENT PROCEDURE

The shear wave velocities are measured by means of a procedure that is in common use (Stokee, 1991). The following is a brief description of some of the specifics of the implementation used for the data presented here. A Bison Series 5000 seismograph is used to record the vibrations. This seismograph is a 12-channel instrument and is equipped with digital filters. It is also a signal stacking recorder so that multiple recordings of a source-receiver configuration can be summed to increase the signal-to-noise ratio. An Oyo Geospace "Borehole Pick" (down-hole geophone) is used to detect the vibrations. This geophone is a three-component instrument that records vibrations in two orthogonal horizontal directions and in the vertical direction.

The source used to generate the shear waves is a beam struck by a sledge hammer (Figure 1 depicts the logging process). The beam is laid on the ground. Parking the wheel of a truck on top of the beam holds the beam firmly in place against the ground. Originally, a wooden beam was used, but the wood was found to deteriorate too rapidly under repeated hammer blows. The system being used now, which seems to perform very well and is very durable, is a 4-ft length of 4-in. by 4-in. steel I-beam with a 1-in. steel plate welded to one end. When the sledge hammer is hit against the end of the horizontal beam, vibrations that are predominantly horizontal shear waves are transmitted into the ground. The beam is placed 3 m horizontally away from the borehole to avoid generating tube waves in the borehole.

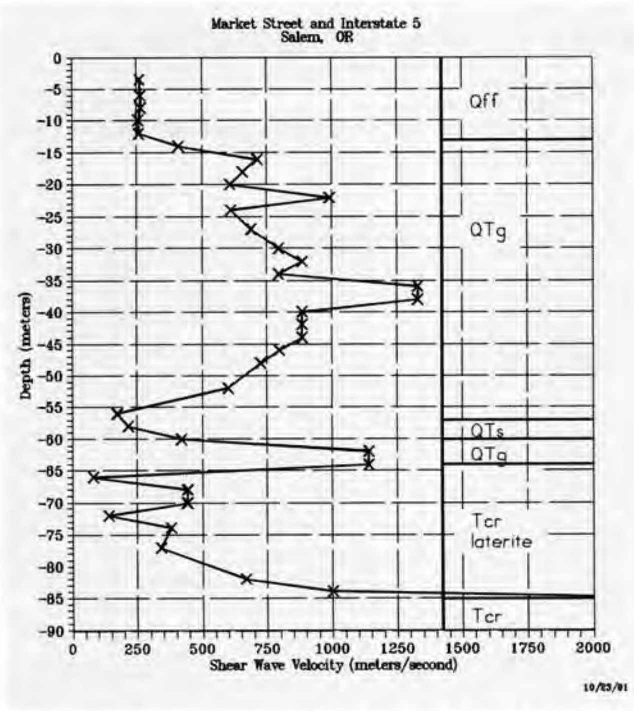

Figure 3. Shear wave velocity profile at Market Street and Interstate 5, Salem. This profile was measured in a hole drilled northeast of the overpass where Interstate 5 crosses Market Street.

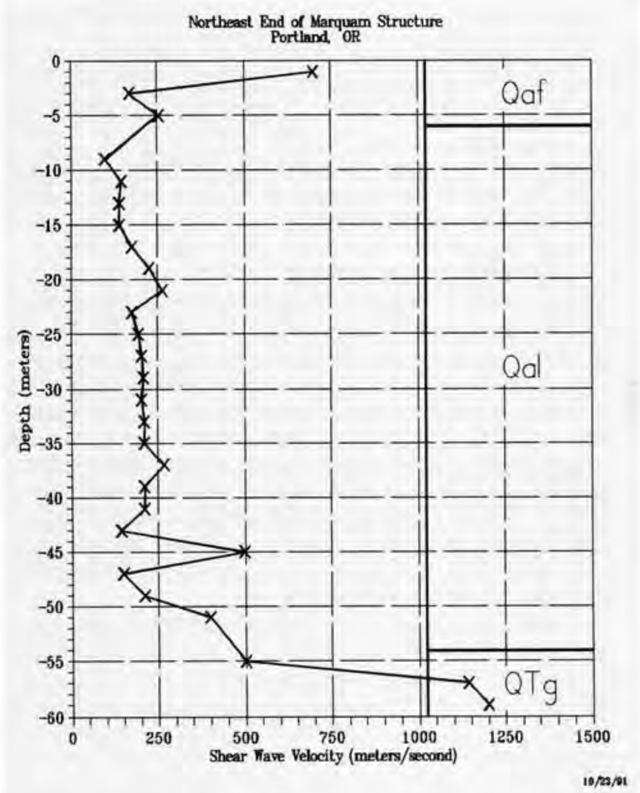

Figure 4. Shear wave velocity profile at the northeast end of the Marquam Bridge structure, Portland. This profile was measured in a hole drilled between the railroad tracks and Interstate 5, on the east side of the Willamette River, immediately north of SE Stark Street.

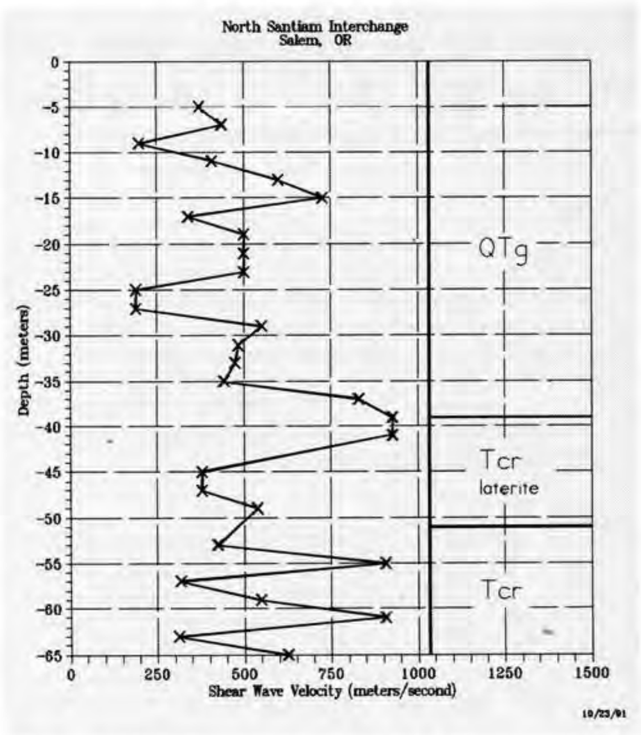

Figure 5. Shear wave velocity profile at the interchange where Interstate 5 crosses the North Santiam Highway near Salem. The profile was measured in a hole drilled to the southwest of the intersection.

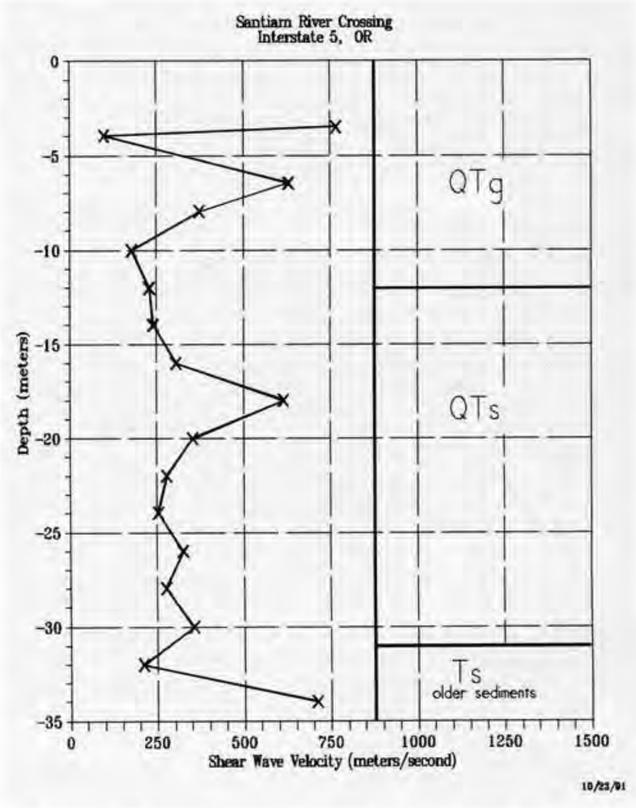

Figure 6. Shear wave velocity profile at Santiam River crossing of Interstate 5. The profile was measured in a hole drilled between the two lanes of Interstate 5, immediately south of the river.

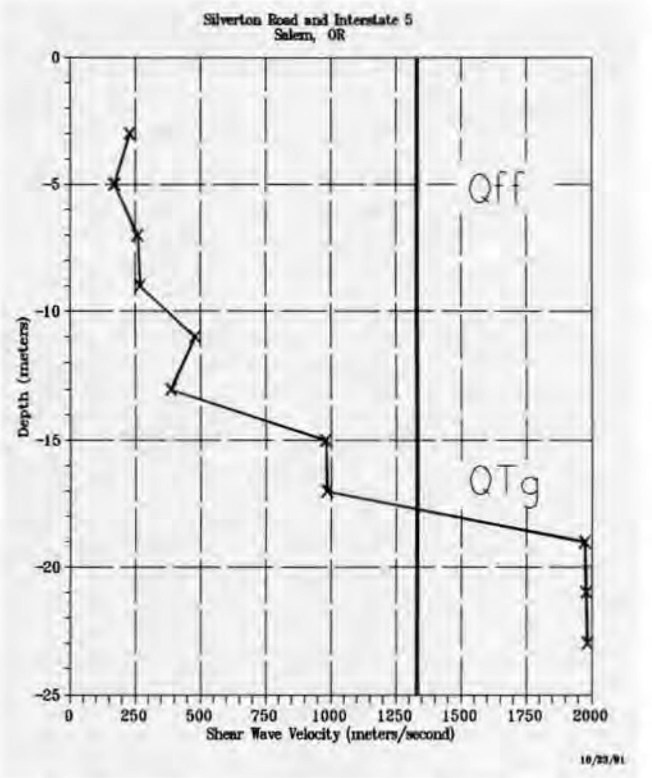

Figure 7. Shear wave velocity profile at Silverton Road and Interstate 5 in Salem. The profile was measured in a hole drilled to the southeast of the overpass where Silverton Road crosses under Interstate 5.

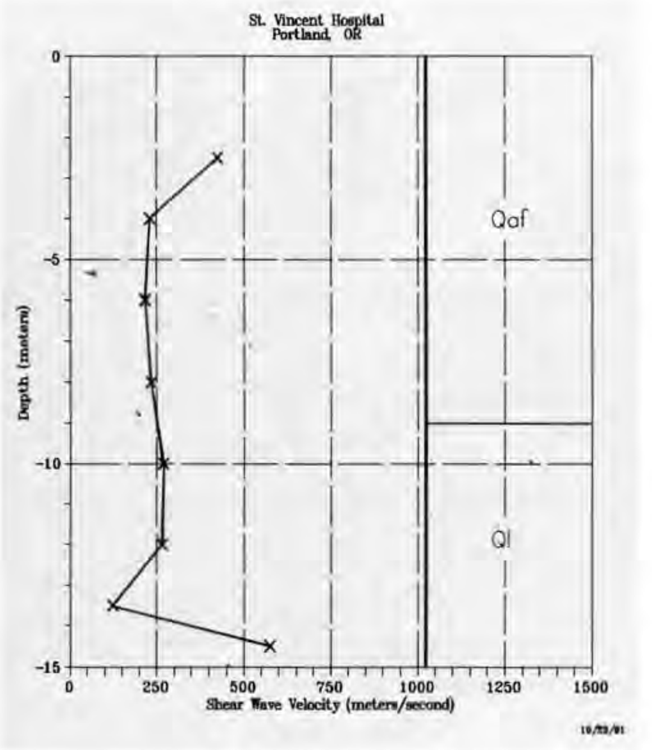

Figure 8. Shear wave velocity profile at St. Vincent Hospital, Portland. The profile was measured in a hole drilled beside the helipad that is located south of the administration building.

The generated vibrations are recorded as they arrive at the geophone, which has been lowered down a borehole. The practice has been to lower the geophone to the bottom of the hole to start. The geophone is secured in place by a pressurized rubber bladder that pushes a metal plate on one side of the geophone against the side of the borehole.

The boreholes have been completed by grouting 2- to 3-in. inside diameter PVC casing in them. The 2-in. diameter represents the smallest casing into which the geophone will fit. Casing larger than 3 in. in diameter could be logged if shims are attached to the geophone. This has not been done to date.

After a recording has been made at a given level, the pressure is released from the bladder, and the geophone is free to be raised to a higher level in the hole. Presently the recordings are being made at 2-m intervals. This process is repeated until the geophone is within 1 or 2 m of the surface.

The data recorded by the seismograph are downloaded to a laptop computer, so that they can be preserved in digital format on floppy disk. This also allows for computer-based digital processing of the data. The files are downloaded as multiplexed (intermixed in a specific pattern) time series of the three recorded channels.

DATA REDUCTION

The data that have been stored on disk are processed on a computer and yield an interval velocity profile of the soil column. The first step is to "demultiplex" the data into three separate time series or traces representing the three components of the geophone. The three recorded traces can then be displayed on a computer screen.

Arrival picks for the shear wave based on a single component of the geophone were found to be dependent on which trace was being used. Correlating waveforms from level to level was also difficult. Creating a vector sum of the two horizontal components allows extremely good correlation of the waveforms at different levels, and a single, unequivocal arrival pick is the result.

A cross-correlation function is used to aid the interpreter's choice of arrival picks and correlations of traces at different levels. The travel times are automatically corrected by the computer program for the geometric effect of the wave path varying with the depth of the measurement. An interval travel time for each 2-m logging interval is the result.

VELOCITY LOGS

Figures 2 through 8 are the results from the seven holes logged so far. These figures are plots of the measured interval shear wave velocity versus depth. Also plotted are generalized lithology logs for the holes. The lithologic units depicted correspond to the Quaternary units mapped in DOGAMI Open-File Report O-90-2 (Madin, 1990). The shear wave velocities reported here should not be viewed as a substitute for site-specific measurements. As additional shear wave velocity data are collected, the resulting profiles will be published in *Oregon Geology.*

ACKNOWLEDGMENTS

The Oregon Department of Transportation, GeoEngineers, Inc., and Rittenhouse-Zeman and Associates, Inc., provided access to the cased bore holes used to collect these data, and this indispensable assistance is gratefully acknowledged.

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Stokee, K.H., 1991, Application of stress waves to soil measurements: Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Mo., March 11-15, Proceedings, S. Prakash, ed., v. III, Rolla, Mo., University of Missouri-Rolla, in press. ☐

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