REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Camas Ridge Elementary School
1150 East 29th Avenue
Eugene, Oregon

For
4J Facilities Management
March 1, 2021

Project: LaneCoSD-1-01

GeoDesign INC
AN NV5 COMPANY

March 1, 2021

4J Facilities Management
715 West 4th Avenue
Eugene, OR 97402

Attention: Glen Macdonald

Report of Geotechnical Engineering Services
Camas Ridge Elementary School
1150 East 29th Avenue
Eugene, Oregon
Project: LaneCoSD-1-01

GeoDesign, Inc., DBA NV5 (GeoDesign) is pleased to submit this report of geotechnical engineering services for the proposed Camas Ridge Elementary School project located at 1150 East 29th Avenue in Eugene, Oregon. Our services for this project were conducted in accordance with our contract dated December 11, 2020. 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: LaneCoSD-1-01-030121-geor.docx

EXECUTIVE SUMMARY

This report presents the results of our geotechnical engineering evaluation for the proposed Camas Ridge Elementary School project located at 1150 East 29th Avenue in Eugene, Oregon. The proposed project includes construction of new school buildings to replace the aging existing school buildings that were constructed in 1949. The proposed buildings will likely be two stories. Additional site improvements include a new parking lot, paved and unpaved play areas, sports fields, and a bus drop-off area.

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:

- Soil conditions vary significantly across the site. The soil encountered at the footing subgrade level will likely include new structural fill, existing undocumented fill, potentially expansive clay, and sandstone. We recommend the proposed buildings for this project be supported on shallow foundations that bear on granular pads, new structural fill, or sandstone. Where existing undocumented fill or potentially expansive clay is present at the footing subgrade elevation, we recommend that the subgrade soil be removed and replaced with compacted granular pads that are 3 feet thick. The thickness of the granular pads may be reduced if sandstone is encountered at depths shallower than 3 feet BGS. In fill areas, it is also acceptable to support the foundations on at least 3 feet of new structural fill. If less than 3 feet of new structural fill will be placed beneath foundations, we recommend 3-foot-thick granular pads be used beneath foundations. In our opinion, the foundation support options described above will provide adequate mitigation for the layer of expansive soil that is present beneath the site.
- We encountered sandstone at shallow depths in our explorations. It is likely that excavations into the sandstone will be required at some locations to construct the proposed project. The sandstone excavation work will be more difficult than excavating soil and may require additional equipment such as buckets equipped with rock teeth, larger excavators, or pneumatic hammers.
- We recommend that floor slabs be supported on at least 6 inches of imported granular material to aid as a capillary break and provide uniform support. Where potentially expansive clay is present, the thickness should be increased to 12 inches. In our opinion, firm fill may be left in place beneath floor slabs, provided the owner is willing to accept the slightly increased risk of slab settlement occurring over time. If soft fill is present at the slab subgrade elevation, it may be necessary to scarify and re-compact the fill or to remove and replace it with imported structural fill.
- We measured negligible infiltration in our infiltration tests at the site. It appears that on-site stormwater infiltration may not be feasible.
- Our site-specific seismic hazard evaluation indicates that the site should be classified as Site Class C.
- In areas where new fill is placed, the weight of the new fill could cause settlement of underlying soil. In areas where more than 3 feet of new fill will be placed over existing undocumented fill or native clay, we recommend allowing time for the soil to consolidate

under the weight of the new fill before concrete foundations or hardscapes are constructed. We recommend that settlement monitoring using survey hubs be performed to verify that settlement is complete before new overlying structures are constructed.

- The on-site soil will be very difficult to use as structural fill. The on-site soil generally has medium to high plasticity and a high clay content, which will be difficult to moisture condition during most of the year. Based on our experience, this soil will be sensitive to small changes in moisture content and may be difficult, if not impossible, to compact adequately during most of the year or when the moisture content is more than a few percentage points above optimum. The soil will likely require extensive drying before it can be used as structural fill. We recommend not attempting to use the on-site soil as structural fill or as retaining wall backfill.

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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
CSZ	Cascadia subduction zone
DOGAMI	Oregon Department of Geology and Mineral Industries
EPA	U.S. Environmental Protection Agency
fps	feet per second
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
km	kilometers
LiDAR	light detection and ranging
MCE	maximum considered earthquake
MCE_G	maximum considered earthquake geometric mean
mg/kg	milligrams per kilogram
NA	not applicable
ohm-cm	ohm-centimeter
OSHA	Occupational Safety and Health Administration
OSHPD	Office of Statewide Health Planning and Development
OSSC	Oregon Standard Specifications for Construction (2021)
PCC	portland cement concrete
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)
VWP	vibrating wire piezometer

1.0 INTRODUCTION

This report presents the results of our geotechnical engineering evaluation for the proposed Camas Ridge Elementary School project located at 1150 East 29th Avenue in Eugene, Oregon. The proposed project includes construction of new school buildings for 450 students that will replace the aging existing school buildings that were constructed in 1949. The proposed buildings will likely be two stories. Additional site improvements will include a new parking lot, paved and unpaved play areas, sports fields, and a bus drop-off area. The site is approximately 7.7 acres that is currently occupied by the existing Camas Ridge Elementary School, which will be demolished and replaced as part of this project. The site is bordered on the north by E 29th Avenue, on the east by University Street, on the south by E 30th Avenue, and on the west by Harris Street. The site location relative to surrounding physical features is shown on Figure 1.

Foundation loads and grading plans for the proposed project have not yet been finalized. Catena Consulting Engineers informed us that maximum column and wall loads will likely be approximately 150 kips and 12 kips per foot, respectively. KPFF Consulting Engineers informed us that maximum cut and fill heights will likely be approximately 12 feet and 5 feet, respectively. We should be contacted to update our recommendations if the actual structural loads, cuts, or fills will exceed these preliminary 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 project. Specifically, we completed the following scope of services:

- Reviewed readily available, published geologic data and our in-house files for existing information on subsurface conditions in the site vicinity.
- Coordinated and managed the field exploration, including utility locates, coordination with existing tenants, and scheduling subcontractors.
- Conducted a subsurface exploration program that consisted of drilling the following borings:
    - One boring to a depth of 40.2 feet BGS
    - Five borings to depths between 15.4 and 21.5 feet BGS
    - Three borings to a depth of 5.5 feet BGS
- Installed a VWP in one of the borings to measure the depth to groundwater.
- Maintained continuous logs of the explorations and collected samples at representative intervals.
- Conducted a laboratory testing program that consisted of the following tests:
    - Six moisture content determinations in general accordance with ASTM D2216
    - Three particle-size analyses in general accordance with ASTM D1140
    - Three Atterberg limits tests in general accordance with ASTM D4318
    - Two expansion index tests in general accordance with ASTM D4829
    - One suite of corrosivity tests including pH, resistivity, sulfate, and chloride

- Performed three infiltration tests at a depth of 4 feet BGS in general accordance with City of Eugene stormwater and Oregon Department of Environmental Quality requirements.
- Provided recommendations for site preparation and grading, including temporary and permanent slopes, fill placement criteria, suitability of on-site soil for fill, subgrade preparation, trench backfill, and corrosion potential of on-site soil.
- Provided recommendations for wet weather construction.
- Provided foundation support recommendations for the proposed buildings, including preferred foundation type, allowable bearing pressure, lateral resistance parameters, and settlement estimates.
- Provided recommendations for site paving sections, including AC and concrete vehicular and fire lane paving.
- 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 seismic design recommendations in accordance with the procedures outlined in ASCE 7-16 and the 2019 SOSSC.
- Performed a site-specific seismic hazard evaluation as required by 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 Eugene-Springfield area is located at the southern end of the main Willamette Valley. The valley is an expression of a major north-south structural trough between the uplifted Coast Range to the west, made of mostly Tertiary Age marine sedimentary rocks, and the Cascade Range to the east, built chiefly of Tertiary volcanic and volcaniclastic rocks (Frank, 1973; Woodward et al., 1998). The foothills of the two ranges converge as a band of hills around the south side of the trough, underlain by a mixture of sedimentary and volcanic rocks. These southern highlands are broken by the valleys of the Willamette (Coast and Middle forks), McKenzie, and Long Tom rivers and many smaller tributaries, such as Amazon Creek.

According to the Geologic Map of Oregon maintained by DOGAMI (DOGAMI, 2021), the surficial soil at the site is mapped as Eocene/Oligocene Age marine sedimentary rock. The rock consists of sandstone that is part of the Eugene Formation (DOGAMI, 2021).

3.2 SURFACE CONDITIONS

The site is an approximately 7.7-acre parcel that is occupied by the existing Camas Ridge Elementary School. Existing school features include one- and two-story buildings, an AC-paved parking lot, an AC-paved basketball court and area, a sports field surrounded by a track, a playground, and other improvements.

The site slopes gently down from east to west and it appears that previous grading has terraced the site into four different levels. The upper levels on the east contain primarily buildings, the central level contains buildings and AC-paved play areas, and the lower level on the west contains an AC-paved parking lot and a sports field surrounded by a track. Elevations at the site range from approximately 510 feet on the east to 465 feet on the west. Existing slopes at the site are generally graded at 2H:1V or flatter. Sidewalks are located around the perimeter of the site. Vegetation at the site includes landscape trees around the perimeter of the site, scattered trees throughout the site, scattered bushes and shrubs, and a grass lawn sports field. A concrete pedestrian bridge is located on the southeast corner of the site that crosses over E 30th Avenue.

3.3 SUBSURFACE CONDITIONS

We explored subsurface conditions at the site by drilling nine borings (B-1 through B-9). The borings were drilled to depths between 5.5 and 40.2 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 AC pavement or topsoil at the ground surface that is underlain by fill, clay, and sandstone. The following sections provide a detailed description of subsurface conditions encountered at the site.

3.3.1 AC Pavement Section

We encountered AC pavement in borings B-2 and B-5 that we drilled near the center of the site in the walkway and basketball court areas. The AC pavement section consists of 2 to 3 inches of AC and 10 to 12 inches of aggregate base.

3.3.2 Topsoil

We encountered topsoil in the borings we drilled in unpaved areas. The upper approximately 2 to 4 inches of the topsoil consists of a root zone. The topsoil generally extends to a depth of approximately 10 inches.

3.3.3 Fill

We encountered fill beneath the AC pavement and topsoil in borings B-4, B-5, and B-6 that extends to depths of approximately 4.5 to 5 feet BGS. The fill consists of soft to medium stiff, sandy clay and loose, clayey sand. The fill is generally moist; contains trace gravel and organics; and is a mixture of various shades of brown, orange, black, and gray. The clay generally has medium to high plasticity and the sand is generally fine to medium grained. We observed a piece of wire debris in boring B-5. The fill is similar to the native soil we encountered, which suggests that it may be on-site material that was moved during previous grading at the site. The clay fill likely has a very high expansion potential, similar to the native clay at the site. Soil of this type and consistency generally exhibits relatively low strength and high compressibility.

3.3.4 Clay

Beneath the AC pavement, topsoil, and fill we typically observed a layer of native clay that extends to depths between approximately 2.5 and 10.5 feet BGS. We did not observe the clay layer in borings B-1 and B-2, although there may have been a thin layer of clay beneath the topsoil that we did not observe in our sampler. The clay is generally very soft to medium stiff; brown with gray, orange, and black mottles; moist; has medium to high plasticity; contains varying amounts of fine to medium sand; and contains trace gravel and organics. Laboratory testing indicates that moisture contents in the clay layer ranged from 35 to 37 percent at the time of our exploration. Laboratory testing also indicates that the clay has a very high expansion potential. Soil of this type and consistency generally exhibits relatively low to moderate strength and moderate to high compressibility.

3.3.5 Sandstone

Beneath the clay we observed a layer of sandstone that extends to a depth of at least 40.2 feet BGS, the maximum depth explored. The sandstone is generally medium dense to very dense, orange-brown to gray, and moist. The sandstone becomes less weathered with depth. Based on geologic maps, we interpret the sandstone to be part of the Eugene Formation. The expansion potential of sandstone is negligible. Soil of this type and consistency generally exhibits relatively high strength and low compressibility.

3.3.6 Groundwater

We encountered a zone of water in boring B-5 at a depth of approximately 10 feet BGS. We did not encounter water in our other borings. Some nearby well logs on file with the Oregon Water Resources Department reported encountering groundwater at the top of the sandstone layer. In our opinion, it is likely that perched water may occasionally be present at the top of the sandstone layer, with the regional groundwater level located at greater depths. 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 infiltration facilities. We performed the tests at a depth of 4 feet BGS in borings B-7, B-8, and B-9. We performed the tests inside 6-inch-diameter hollow-stem augers using the encased falling head test method. We performed the tests with a water head of approximately 1.5 to 2 feet to simulate conditions in an infiltration swale. We collected representative soil samples below the infiltration test depths 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-7	4	Sandy CLAY	0.0	68
B-8	4	Sandy CLAY	0.0	70
B-9	4	SANDSTONE	0.0	52

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 very low and on-site infiltration may not be feasible. If on-site stormwater infiltration is attempted, we recommend infiltration testing be performed during construction to verify the design infiltration rates are being achieved.

3.5 EXPANSIVE SOIL TESTING

Benchmark Geolabs of McMinnville, Oregon, performed expansion index tests on two soil samples to evaluate the expansion potential of the soil. The tests were performed in accordance with ASTM D4829. The test results are presented in Appendix C and are summarized in Table 2.

Table 2. Expansion Index Test Results

Boring	Depth (feet BGS)	Soil Type	Expansion Index Value	Potential Expansion[1]
B-3	1.5	CLAY with sand	155	Very high
B-7	1.0	CLAY	136	Very high

1. Interpretation from 1997 Edition of the Uniform Building Code

The expansion index tests indicate that the soil tested has a very high expansion potential. It is likely that the clay soil layer above the sandstone could expand or shrink as it undergoes seasonal wetting and drying. Mitigation will be required to prevent damage to new foundations and other structures that are constructed on expansive soil.

3.6 CORROSIVITY

Benchmark Geolabs of McMinnville, Oregon, performed a suite of corrosivity tests on a sample of soil from boring B-3 at a depth of 1.5 feet. The test results are presented in Appendix C and are summarized in Table 3.

Table 3. Corrosivity Test Results

Test	Standard	Result
pH	ASTM G51	5.1
Sulfate	EPA 300.0	3 mg/kg
Chloride	EPA 300.0	6 mg/kg
Saturated resistivity	ASTM G57	568 ohm-cm

We recommend that these corrosivity test results be reviewed by the project structural and civil engineers so they can determine if the proposed buildings and utilities will require corrosion mitigation.

4.0 GEOLOGIC HAZARDS

We evaluated the presence of geologic hazards in 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 is gently sloped, but it is not steep enough that landslides will be a significant hazard. State of Oregon hazard mapping and LiDAR mapping do not show any landslides at the site (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.

Dense sandstone is present beneath the site at relatively shallow depths. The sandstone is not susceptible to liquefaction. Groundwater appears to generally be located at or below the top of the sandstone. The clay above the sandstone is unsaturated and has high plasticity, so it is also not susceptible to liquefaction. As a result, it is our opinion that the soil at the site is not susceptible to liquefaction.

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 closest active mapped faults are located more than 35 km from the site (USGS, 2021). In our opinion, fault surface rupture is not a hazard at the site.

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 is gently sloped, it is our opinion that seismically 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 the building code was not encountered during our subsurface explorations. We conclude the level of amplification determined by the building code seismic design parameters is appropriate and the proposed project can be designed using the levels of ground shaking prescribed by ASCE 7-16 and the 2019 SOSSC.

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 the site is outside the 100-year flood zone (DOGAMI, 2018). As a result, it is our opinion that the risk of flooding at the site is low.

4.4 VOLCANIC HAZARDS

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

4.5 EXPANSIVE SOIL

As discussed in the "Expansive Soil Testing" section, our laboratory testing found that expansive soil is present at the site. Expansive soil is susceptible to expanding and shrinking as it goes through periods of wetting and drying. This soil movement can damage overlying buildings, pavement, and other hardscapes as the soil moves. This is a geologic hazard that will need be mitigated, as discussed in applicable sections of this report.

5.0 DESIGN RECOMMENDATIONS

5.1 FOUNDATION SUPPORT

5.1.1 General

Based on the results of our explorations and analysis, it appears that soil conditions vary significantly across the site. The soil encountered at the footing subgrade level will likely include new structural fill, existing undocumented fill, potentially expansive clay, and sandstone. We recommend the proposed buildings for this project be supported on shallow foundations that bear on granular pads, new structural fill, or sandstone. Where existing undocumented fill or potentially expansive clay is present at the footing subgrade elevation, we recommend the subgrade soil be removed and replaced with compacted granular pads that are 3 feet thick. The thickness of the granular pads may be reduced if sandstone is encountered at depths shallower than 3 feet BGS. We anticipate the required granular pad thicknesses will generally range from 0 to 3 feet across the site, depending on the depth to sandstone. The thickness of the granular pads may need to be increased in some areas if very soft existing fill extends deeper than 3 feet below the foundations. In fill areas, it is also acceptable to support the foundations on at least 3 feet of new structural fill. If less than 3 feet of new structural fill will be placed beneath foundations, we recommend 3-foot-thick granular pads be used beneath foundations. In our opinion, the foundation support options described above will provide adequate mitigation for the layer of expansive soil that is present beneath the site.

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 material; 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.

5.1.2 Bearing Capacity

Spread footings supported on granular pads or sandstone should be sized using an allowable bearing pressure of 3,000 psf. This value may be increased by 50 percent for short-term loads such as wind or seismic forces.

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.

5.1.3 Settlement

We anticipate 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 existing clay or sand. Foundations in contact with crushed rock or sandstone 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 provide uniform support. The thickness should be increased to 12-inches where potentially expansive clay is present. In our opinion, firm fill may be left in place beneath floor slabs, provided the owner is willing to accept the slightly increased risk of slab settlement occurring over time. If soft fill is present at the slab subgrade elevation, it may be necessary to scarify and re-compact the fill or to remove and replace it with imported structural fill. The 12-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.

Some of the soil beneath floor slabs has the potential to expand and contract when the moisture content of the soil fluctuates. To mitigate this, we recommend that foundation drains be installed around the perimeter of the buildings to keep the soil beneath the floor slabs well drained. We also recommend that surface drainage be directed away from buildings. The finished ground surface within 5 feet of buildings should be covered with an impermeable surface in areas where potentially expansive clay is present. This impermeable surface can consist of sidewalk, pavement, other hardscape, a membrane covered with soil, cement-amended soil, or some similar way of keeping surface water at least 5 feet away from the buildings.

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.

5.3 SEISMIC DESIGN PARAMETERS

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

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, tiered 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. We recommend that on-site soil not be used as retaining wall backfill. All potentially expansive clay should be removed from behind retaining walls for a minimum distance of 5 feet behind the walls.

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 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. 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 buildings 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 buildings to keep the soil beneath the floor slabs well drained, which will limit the expansion potential of the soil. 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.

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

New pavement on this project is anticipated to consist of an AC parking lot, an AC or PCC bus drop-off area, and an AC play area. Our pavement design recommendations assume the subgrade has been prepared in accordance with the “Site Preparation” and “Structural Fill” sections. 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.

- A resilient modulus value of 3,000 psi for subgrade based on the soil type.
- 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 in the parking lot will consist of approximately 150 cars per day; 5 two-axle delivery trucks per day; and 1 three-axle delivery truck, garbage truck, or similarly heavy vehicles per day.
- Traffic in the bus drop-off area will consist of approximately 15 school buses and 5 two-axle delivery trucks per day.
- Traffic in the play area will consist of approximately one maintenance pickup truck 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 Pavement Design Sections (Post Construction)

Our pavement design recommendations for the assumptions and loads provided above are presented in Table 4.

Table 4. Recommended Standard Pavement Sections

Pavement Use	AC Thickness[1] (inches)	PCC Thickness[1] (inches)	Aggregate Base Thickness[1,2] (inches)
Parking Lot Drive Aisles – Automobiles and Heavy Vehicles	3.0	NA	12.0
Parking Lot Automobile Parking Only	3.0	NA	10.0
Bus Drop-off Area	4.0	NA	12.0
Bus Drop-off Area	NA	7.0	10.0
Play Area	2.5	NA	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 placed between the aggregate base and the subgrade.

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. If silt or clay is present at the subgrade level, a subgrade geotextile fabric should be used to extend the life of the pavement by preventing fines from gradually migrating into the base rock.

The presence of expansive soil beneath pavement can reduce the life of the pavement. If any potentially expansive soil is present at the subgrade level, it should not be allowed to dry out significantly and may require some over-excavation. Other mitigation measures may include details that prevent water from entering the subgrade, which will prevent the soil moisture content from fluctuating.

Although we have presented both AC and PCC pavement design sections for the bus drop-off area, it is our opinion that AC pavement will perform better over the life of the project since it is more flexible than the PCC and can better tolerate soil movement.

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 PCC

The PCC should be Class 4000 ¾-inch or 1½-inch paving concrete. Joints should be placed at a maximum spacing of 12 feet. The length-to-width ratio of any panel should be at least 0.75 and should not exceed 1.25. PCC paving should only occur when ground temperatures are 40 degrees Fahrenheit or warmer.

5.7.3.4 Subgrade Geotextile Fabric

A subgrade geotextile fabric should be placed as a barrier between the subgrade and granular material. The geotextile 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).

5.8 PLACEMENT OF NEW FILL

In areas where new fill is placed, the weight of the new fill could cause settlement of underlying soil. In areas where more than 3 feet of new fill will be placed over existing undocumented fill or native clay, we recommend allowing time for the soil to consolidate under the weight of the new fill before concrete foundations or hardscapes are constructed. We recommend that settlement monitoring using survey hubs be performed to verify that settlement is complete before new overlying structures are constructed.

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 and shrubs that remain in landscape areas after cuts are performed. Root material should be removed from all building, pavement, and structural fill 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 will be required to remove existing buildings, floor slabs, utilities, AC pavement, and other existing improvements from the site. These features should be completely removed from beneath new structures. 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 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.

6.1.3 Subgrade Evaluation

A member of our geotechnical 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 demolish existing structures, as well as to construct new foundations, utilities, stormwater infiltration facilities, and other improvements. Conventional earthmoving equipment in proper working condition should be capable of making the necessary excavations in soil, although excavations in sandstone will be more difficult and may require additional equipment such as buckets equipped with rock teeth, larger excavators, or pneumatic hammers. We anticipate temporary excavation sidewalls will generally stand vertical to a depth of approximately 4 feet, provided water seepage does not occur.

Excavations deeper than 4 feet will require shoring or should be sloped. Sloped excavations may be used to vertical depths of 15 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 generally be above the groundwater level. However, some perched water could still seep into the site excavations, especially after periods of heavy rain. We anticipate dewatering methods consisting of pumping water from excavation sumps will generally be adequate. If possible, we recommend 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

The on-site fine-grained soil will be very difficult to use on site as structural fill. The soil has medium to high plasticity and a high clay content, which will be difficult to moisture condition during most of the year. Based on our experience, this soil will be sensitive to small changes in moisture content and may be difficult, if not impossible, to compact adequately during most of the year or when the moisture content is more than a few percentage points above optimum. The soil will likely require extensive drying before it can be used as structural fill. We recommend not attempting to use the on-site fine-grained soil as structural fill. If used as structural fill, the on-site fine-grained soil should be placed in lifts with a maximum uncompacted thickness of 8 to 12 inches and compacted to not less than 92 percent of the material’s maximum dry density, as determined by ASTM D1557. We recommend using imported granular material for structural fill if the moisture content of the on-site soil cannot be reduced.

The on-site sand could also be difficult to use as structural fill because of the clay that is mixed with it. The sand will still likely require moisture conditioning, but may be easier to dry than the on-site fine-grained soil. If used as structural fill, the on-site sand 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 moisture condition the sand 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 soil cannot be reduced.

We did not perform compaction testing on the soil collected during our field exploration program because we did not encounter material that is well suited for use as structural fill, because of the significant variability in the soil types at the site, and the uncertainty regarding which soil types would actually be used as structural fill. During construction, we recommend compaction tests be performed on the soil that will be attempted to use as structural fill. This could include tests on the clay, sand, and sandstone material that is at the site. It will likely be necessary to perform multiple compaction tests on each soil type to account for soil variability.

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 Eugene trench backfill requirements should be followed for any public utilities that are installed. Our trench backfill recommendations for private utilities are provided below.

Trench backfill for the utility pipe base and pipe zone should consist of durable, well-graded, granular material that has a maximum particle size of 1 inch, has less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and contains no organic material or other deleterious materials. Backfill above the pipe zone should meet the requirements above, except that the maximum particle size may be increased to 1½ inches.

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

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 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, and performing laboratory compaction and field moisture-density tests.

8.0 LIMITATIONS

We have prepared this report for use by 4J Facilities Management 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

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Associate Engineer

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Principal Engineer

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

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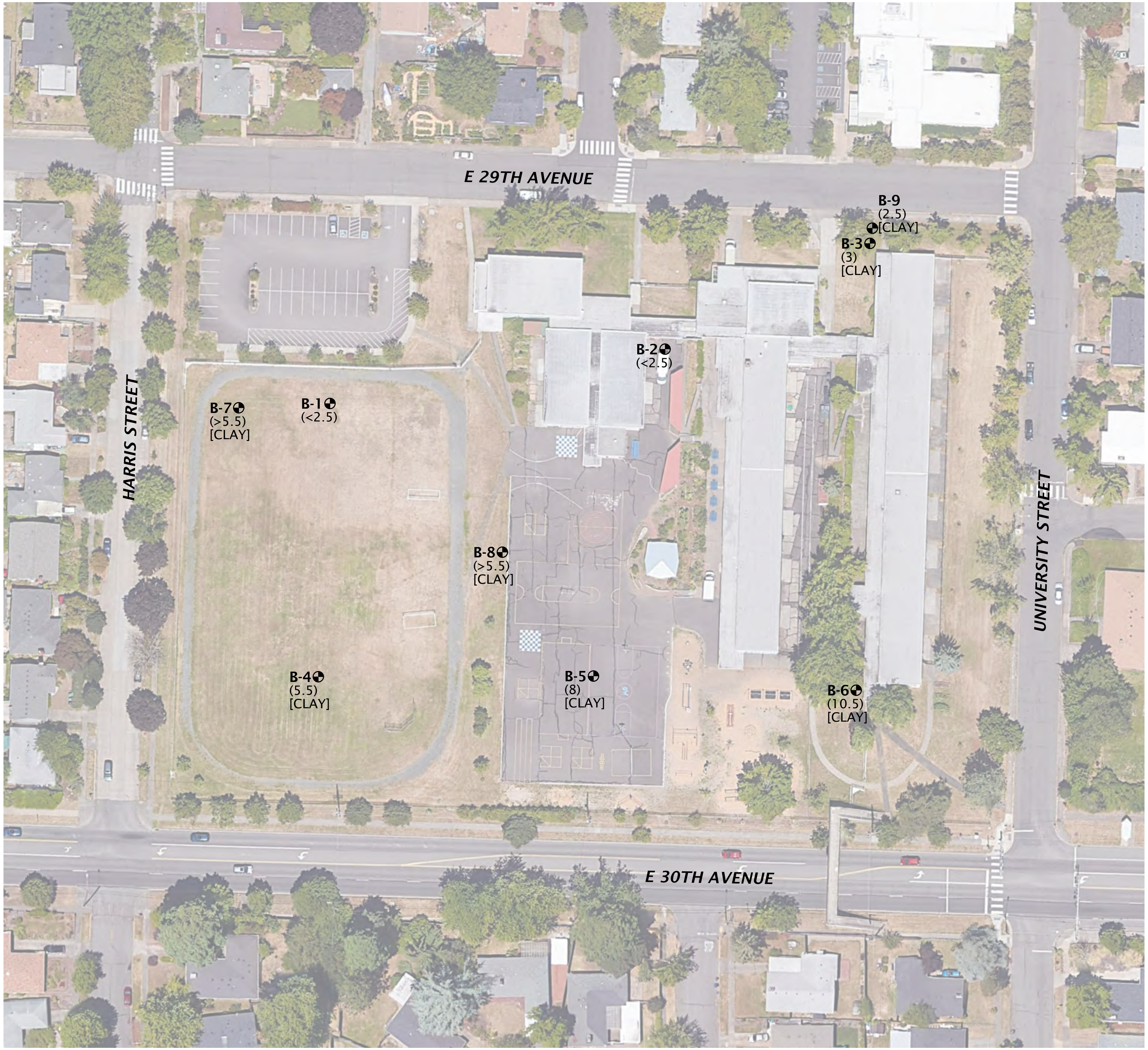
FIGURES

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

GEODESIGN INC AN NV5 COMPANY	LANECOSD-1-01	**VICINITY MAP**	
	MARCH 2021	CAMAS RIDGE ELEMENTARY SCHOOL EUGENE, OR	**FIGURE 1**



LEGEND:

B-1◐	BORING
(5.5)	DEPTH TO SANDSTONE (FEET BGS)
[CLAY]	POTENTIALLY EXPANSIVE CLAY OBSERVED DURING EXPLORATIONS

N

0 80 160

(SCALE IN FEET)

SITE PLAN BASED ON AERIAL PHOTOGRAPH
OBTAINED FROM GOOGLE EARTH PRO®,
FEBRUARY 24, 2021

FIGURE 2

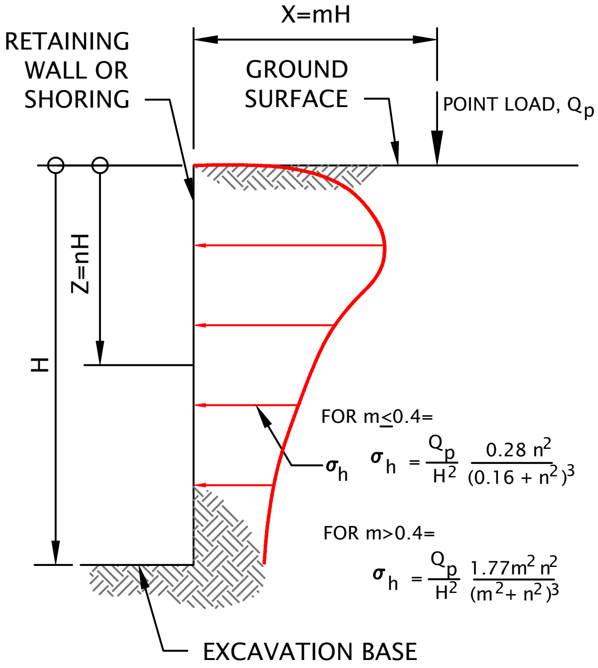
SITE PLAN

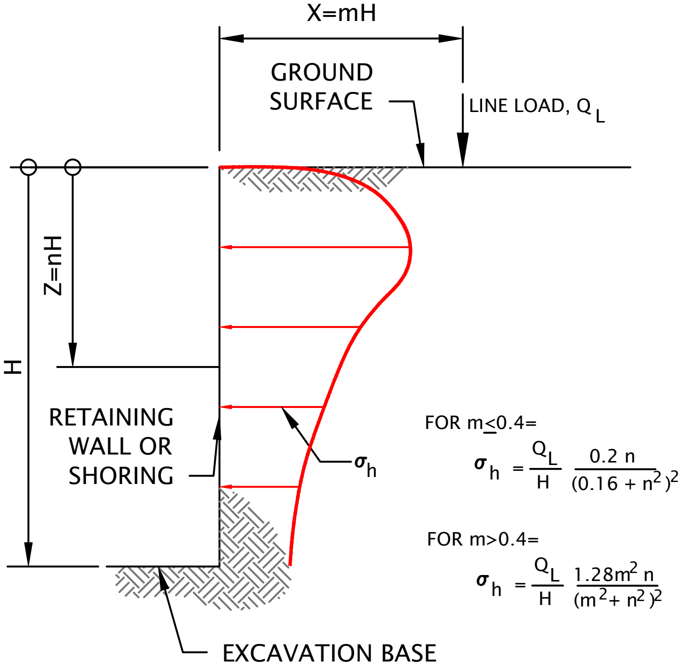
CAMAS RIDGE ELEMENTARY SCHOOL
EUGENE, OR

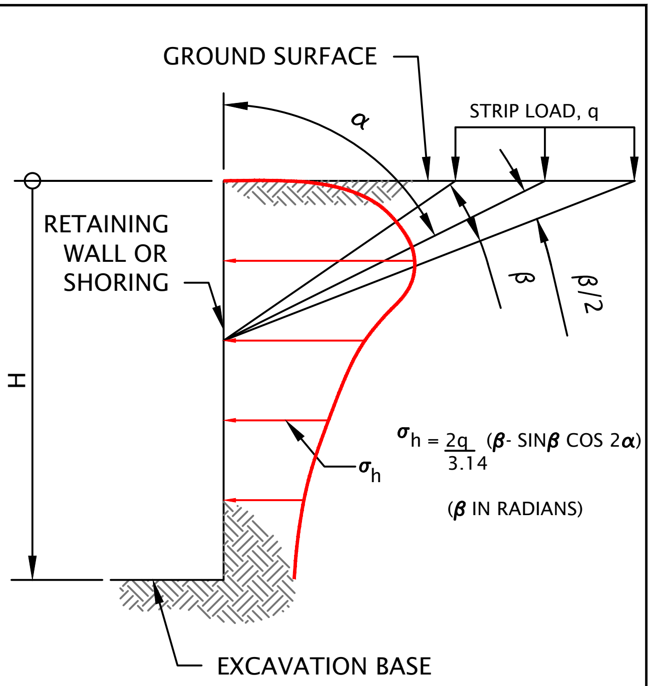
LANECOSD-1-01

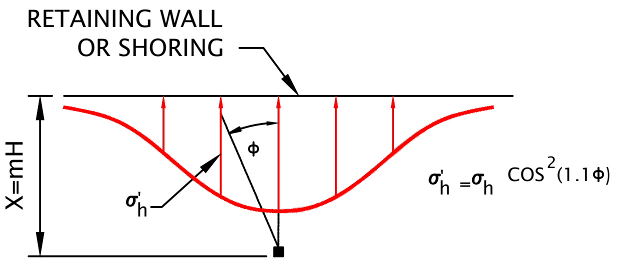
MARCH 2021

GEODESIGN℠
AN NV5 COMPANY









DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

LINE LOAD PARALLEL TO WALL

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
AN NV5 COMPANY

LANECOSD-1-01

MARCH 2021

SURCHARGE-INDUCED LATERAL EARTH PRESSURES

CAMAS RIDGE ELEMENTARY SCHOOL
EUGENE, OR

FIGURE 3

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We conducted a subsurface exploration program that consisted of drilling nine borings (B-1 through B-9) at the approximate locations shown on Figure 2. The borings were drilled to depths between 5.5 and 40.2 feet BGS. The borings were drilled using hollow-stem auger and mud rotary drilling methods. Drilling services were provided by Western States Soil Conservation, Inc. of Hubbard, Oregon, on January 20 and 21, 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 by using the Google Earth computer program and a topographic map of the site.

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 hammer used by Western States Soil Conservation, Inc. was 85 percent. The results of the hammer calibration testing 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.

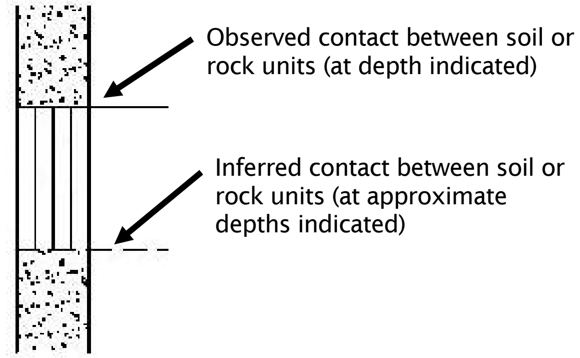
PARTICLE-SIZE ANALYSIS

We completed particle-size analyses 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

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

GEODESIGN INC AN NV5 COMPANY	**EXPLORATION KEY**	**TABLE A-1**

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

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SOIL CLASSIFICATION SYSTEM

TABLE A-2

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

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	BLOW COUNT (0 50 100)	INSTALLATION AND COMMENTS
0.0	Very dense, orange-brown with orange and black mottled, silty SAND (SM), trace organics; moist, sand is fine, moderate cementation (topsoil to 10 inches, 4-inch-thick root zone; weathered sandstone).	474.0				
2.5				◨	23-50/5' ▲	
5.0				◨	19-43-50/4" ▲	
7.5	trace gravel; gravel is fine and rounded at 7.5 feet			◨	85 ▲	
10.0	blue-gray with orange and brown mottles, without gravel at 10.0 feet			◨	38-50/3" ▲	
12.5						
15.0	blue-gray (sandstone) at 15.0 feet			◨	28-50/5" ▲	
	Exploration completed at a depth of 15.9 feet. Hammer efficiency factor is 85 percent.	458.1 15.9				
17.5						
20.0						
22.5						
25.0						
27.5						
30.0						

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: H. Herinckx
COMPLETED: 01/21/21

BORING METHOD: hollow-stem auger (see document text)
BORING BIT DIAMETER: 3 7/8 inches

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AN NV5 COMPANY

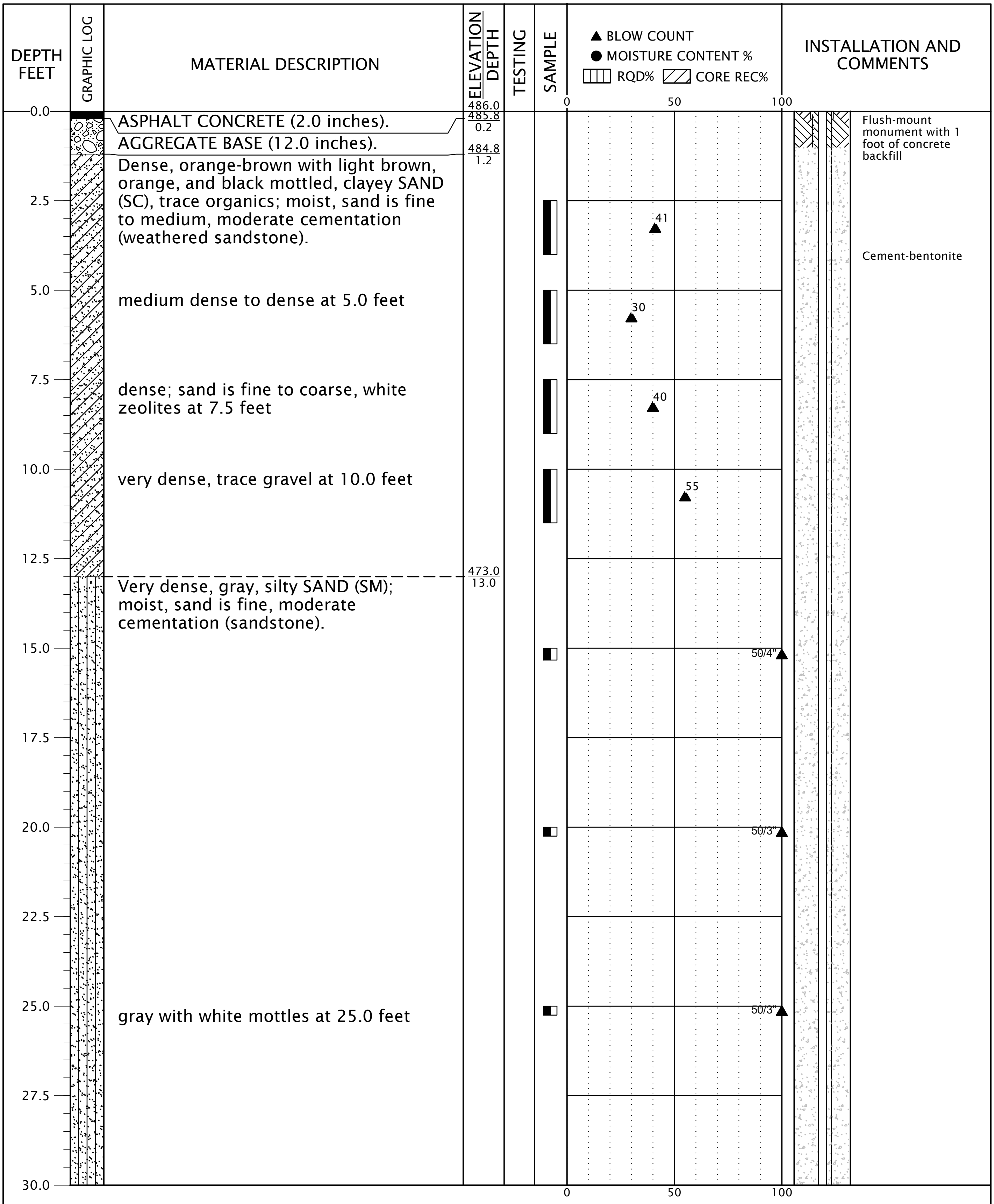
LANECOSD-1-01

MARCH 2021

BORING B-1

CAMAS RIDGE ELEMENTARY SCHOOL
EUGENE, OR

FIGURE A-1



ASPHALT CONCRETE (2.0 inches).

AGGREGATE BASE (12.0 inches).

Dense, orange-brown with light brown, orange, and black mottled, clayey SAND (SC), trace organics; moist, sand is fine to medium, moderate cementation (weathered sandstone).

medium dense to dense at 5.0 feet

dense; sand is fine to coarse, white zeolites at 7.5 feet

very dense, trace gravel at 10.0 feet

Very dense, gray, silty SAND (SM); moist, sand is fine, moderate cementation (sandstone).

gray with white mottles at 25.0 feet

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: H. Herinckx

COMPLETED: 01/20/21

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

GEODESIGN INC
AN NV5 COMPANY

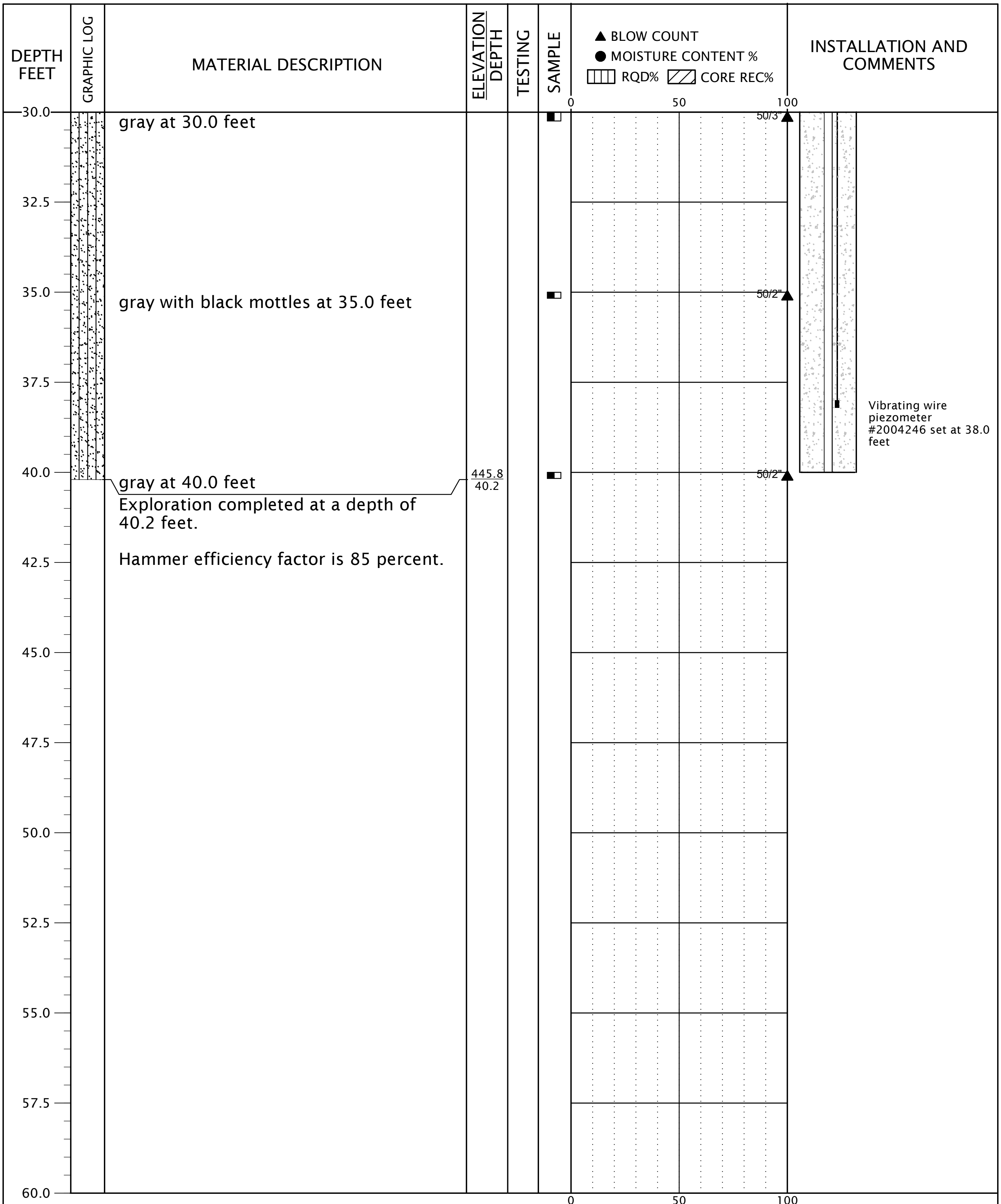
LANECOSD-1-01

MARCH 2021

BORING B-2

CAMAS RIDGE ELEMENTARY SCHOOL
EUGENE, OR

FIGURE A-2



gray at 30.0 feet

gray with black mottles at 35.0 feet

gray at 40.0 feet

Exploration completed at a depth of 40.2 feet.

Hammer efficiency factor is 85 percent.

Vibrating wire piezometer #2004246 set at 38.0 feet

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: H. Herinckx

COMPLETED: 01/20/21

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

GEODESIGN INC
AN NV5 COMPANY

LANECOSD-1-01

MARCH 2021

BORING B-2
(continued)

CAMAS RIDGE ELEMENTARY SCHOOL
EUGENE, OR

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
0.0	▨	Very soft, brown with gray mottled CLAY with sand (CL), trace gravel and organics; moist (topsoil to 10 inches, 2-inch-thick root zone).	499.0				Expansion index test and corrosion tests performed on soil from 1.5 feet.
2.5		Medium dense, orange-brown with orange and black mottled SAND with clay (SP-SC), trace organics; moist, moderate cementation (weathered sandstone).	496.0 3.0		▮	25	
5.0		very dense at 5.0 feet			▮	61	
7.5		Medium dense, orange-brown with orange and black mottled, clayey SAND (SC); moist, sand is fine, white zeolites (weathered sandstone).	492.0 7.0		▮	27	
10.0		dense; sand is fine to coarse at 10.0 feet			▮	39	
12.5							
15.0		very dense, brown; sand is fine at 15.0 feet			▮	81	
17.5							
20.0					▮	24-39-50/6"	
22.5		Exploration completed at a depth of 21.5 feet. Hammer efficiency factor is 85 percent.	477.5 21.5				
25.0							
27.5							
30.0							

DRILLED BY: Western States Soil Conservation, Inc.   LOGGED BY: H. Herinckx   COMPLETED: 01/21/21

BORING METHOD: hollow-stem auger (see document text)   BORING BIT DIAMETER: 3 7/8 inches

GEODESIGN INC
AN NV5 COMPANY

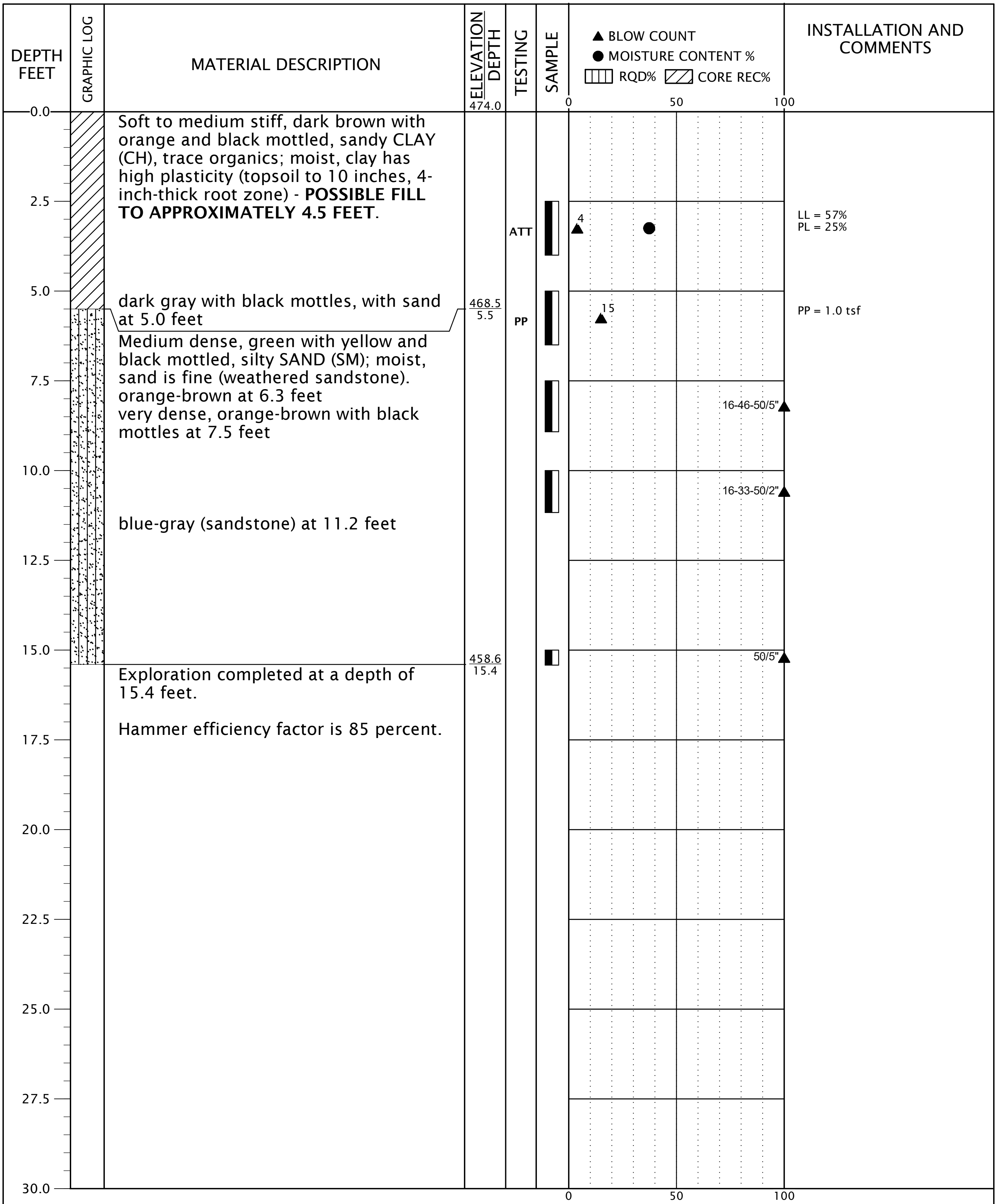
LANECOSD-1-01

MARCH 2021

BORING B-3

CAMAS RIDGE ELEMENTARY SCHOOL
EUGENE, OR

FIGURE A-3



Soft to medium stiff, dark brown with orange and black mottled, sandy CLAY (CH), trace organics; moist, clay has high plasticity (topsoil to 10 inches, 4-inch-thick root zone) - **POSSIBLE FILL TO APPROXIMATELY 4.5 FEET**.

dark gray with black mottles, with sand at 5.0 feet

Medium dense, green with yellow and black mottled, silty SAND (SM); moist, sand is fine (weathered sandstone).
orange-brown at 6.3 feet
very dense, orange-brown with black mottles at 7.5 feet

blue-gray (sandstone) at 11.2 feet

Exploration completed at a depth of 15.4 feet.

Hammer efficiency factor is 85 percent.

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: H. Herinckx
COMPLETED: 01/20/21

BORING METHOD: hollow-stem auger (see document text)
BORING BIT DIAMETER: 3 7/8 inches

GEODESIGN INC
AN NV5 COMPANY

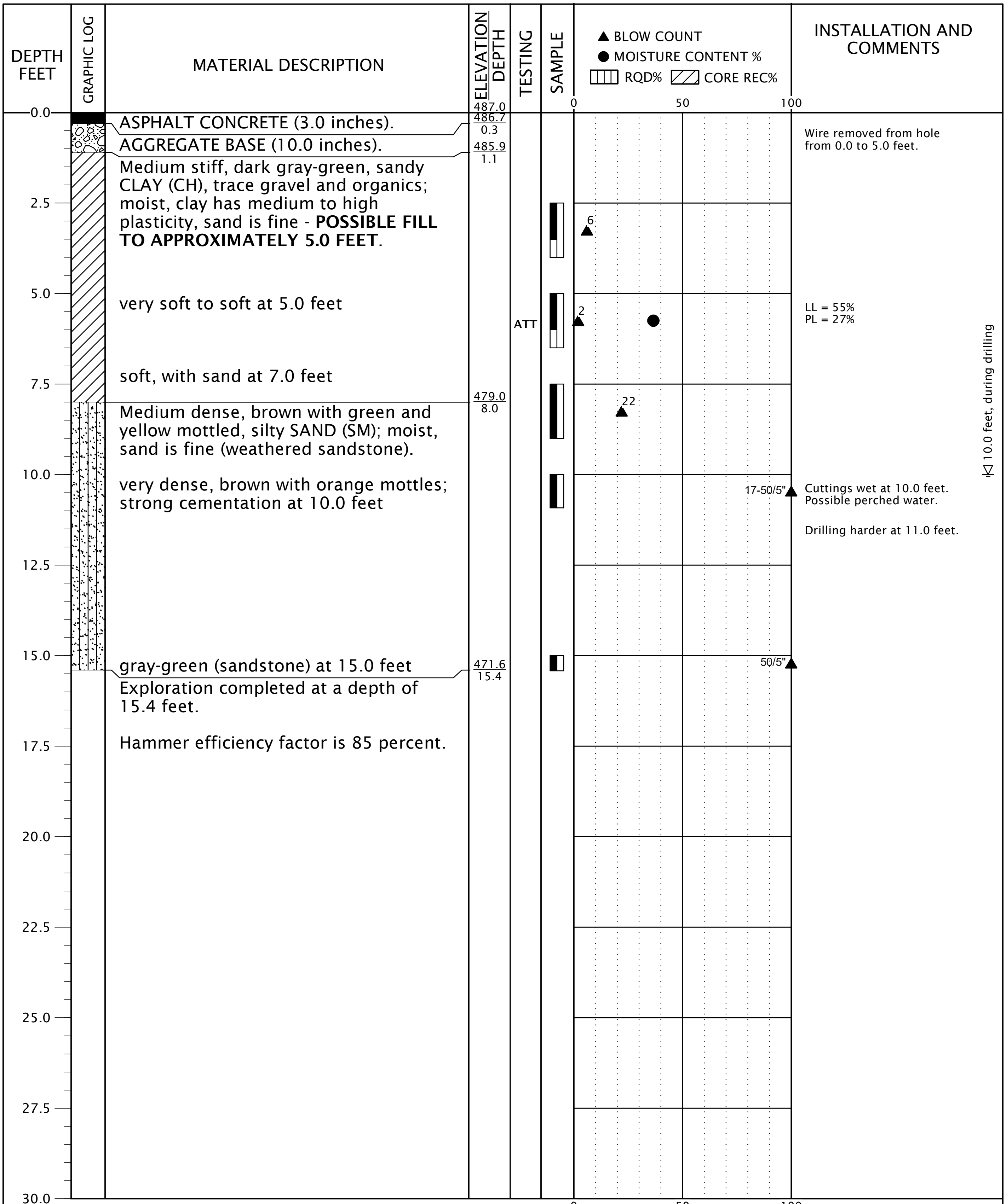
LANECOSD-1-01

MARCH 2021

BORING B-4

CAMAS RIDGE ELEMENTARY SCHOOL
EUGENE, OR

FIGURE A-4



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (3.0 inches).	487.0 486.7 0.3			
	AGGREGATE BASE (10.0 inches).	485.9 1.1			Wire removed from hole from 0.0 to 5.0 feet.
	Medium stiff, dark gray-green, sandy CLAY (CH), trace gravel and organics; moist, clay has medium to high plasticity, sand is fine - **POSSIBLE FILL TO APPROXIMATELY 5.0 FEET**.				
2.5				6	
5.0	very soft to soft at 5.0 feet		ATT	2	LL = 55% PL = 27%
7.5	soft, with sand at 7.0 feet				
	Medium dense, brown with green and yellow mottled, silty SAND (SM); moist, sand is fine (weathered sandstone).	479.0 8.0		22	
10.0	very dense, brown with orange mottles; strong cementation at 10.0 feet			17-50/5"	Cuttings wet at 10.0 feet. Possible perched water. Drilling harder at 11.0 feet.
12.5					
15.0	gray-green (sandstone) at 15.0 feet	471.6 15.4		50/5"	
	Exploration completed at a depth of 15.4 feet.				
17.5	Hammer efficiency factor is 85 percent.				
20.0					
22.5					
25.0					
27.5					
30.0					

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: H. Herinckx COMPLETED: 01/20/21

BORING METHOD: hollow-stem auger (see document text) BORING BIT DIAMETER: 3 7/8 inches

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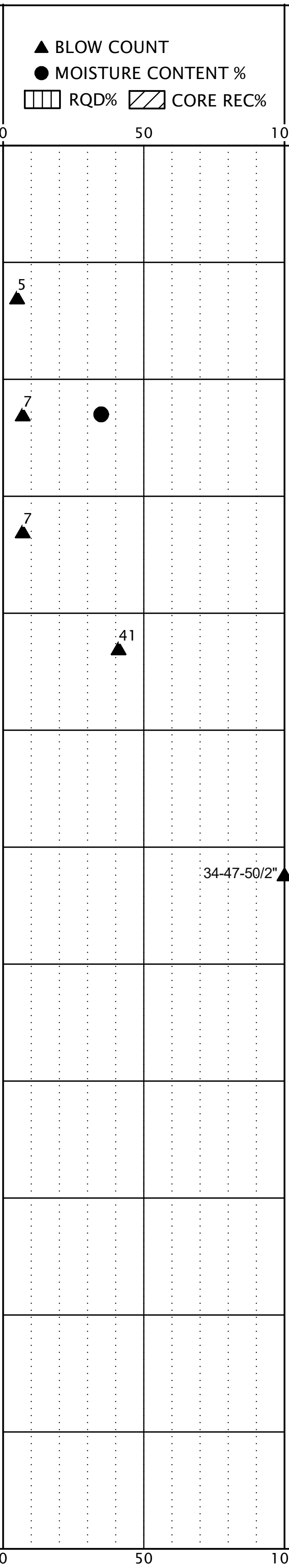
MARCH 2021

BORING B-5

CAMAS RIDGE ELEMENTARY SCHOOL
EUGENE, OR

FIGURE A-5



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	Loose, dark brown, clayey SAND with gravel (SC), trace organics roots); moist, sand is fine to medium, gravel is coarse and subangular (topsoil to 10 inches, 4-inch-thick root zone) - **FILL**.	499.0			
2.5				5	
4.5	Medium stiff, dark brown with black and orange mottled, sandy CLAY (CH), trace organics; moist, clay has high plasticity.	494.5 4.5			
5.0			ATT PP	7	PP = 1.5 tsf LL = 87% PL = 26%
7.5	dark gray, trace sand, without organics at 7.5 feet		PP	7	PP = 1.5 tsf
10.5	Dense, green with yellow mottled, silty SAND (SM); moist, sand is fine. orange-brown (weathered sandstone) at 11.3 feet	488.5 10.5		41	
13.5	Very dense, brown with orange mottled, clayey SAND (SC); moist, sand is fine (weathered sandstone).	485.5 13.5			
15.0				34-47-50/2"	
16.2	Exploration completed at a depth of 16.2 feet. Hammer efficiency factor is 85 percent.	482.8 16.2			

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: H. Herinckx

COMPLETED: 01/21/21

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

BORING BIT DIAMETER: 3 7/8 inches

GEODESIGN INC AN NV5 COMPANY

LANECOSD-1-01

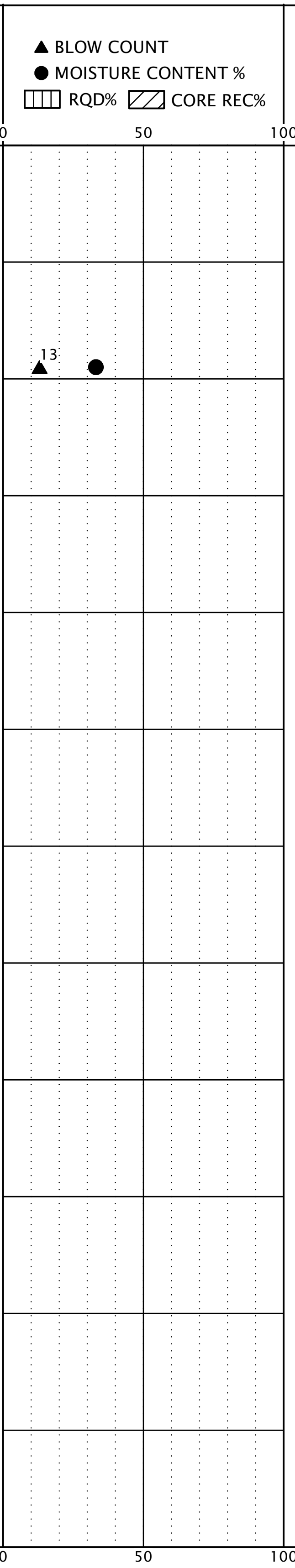
MARCH 2021

BORING B-6

CAMAS RIDGE ELEMENTARY SCHOOL
EUGENE, OR

FIGURE A-6

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	INSTALLATION AND COMMENTS
0.0	▨	Stiff, brown with orange and gray mottled, sandy CLAY (CL), trace organics; moist, sand is fine (topsoil to 10 inches, 4-inch-thick root zone).	472.0			Expansion index test performed on soil from 1.0 foot.
				P200	▮	Infiltration test at 4.0 feet. P200 = 68%
5.5		Exploration completed at a depth of 5.5 feet. Hammer efficiency factor is 85 percent.	466.5 5.5			



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: H. Herinckx

COMPLETED: 01/21/21

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

BORING BIT DIAMETER: 6 inches

GEODESIGN INC
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LANECOSD-1-01

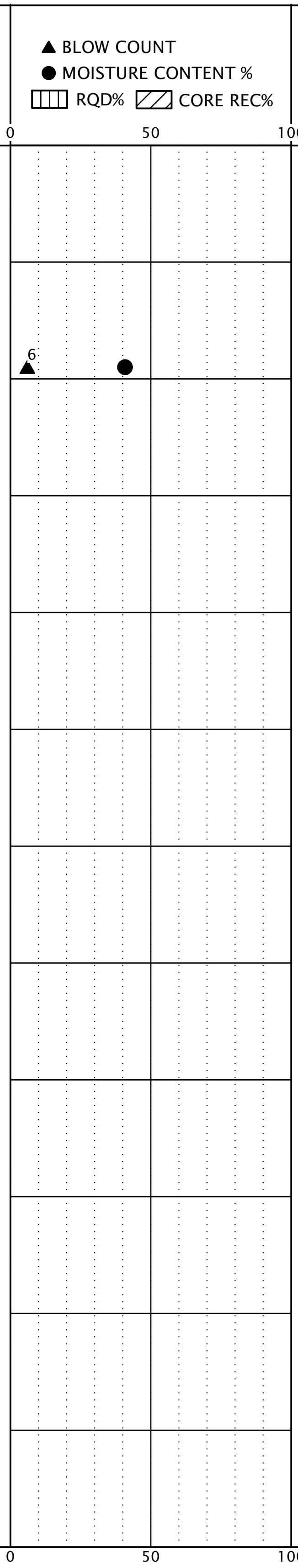
MARCH 2021

BORING B-7

CAMAS RIDGE ELEMENTARY SCHOOL
EUGENE, OR

FIGURE A-7

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC%	INSTALLATION AND COMMENTS
0.0	▨	Medium stiff, dark brown with orange and black mottled, sandy CLAY (CL), trace organics; moist, fragment of light brown sandstone (topsoil to 10 inches, 4-inch-thick root zone).	485.0				
2.5							
5.0				P200	▮	▲ 6 ●	Infiltration test at 4.0 feet. P200 = 70%
		Exploration completed at a depth of 5.5 feet. Hammer efficiency factor is 85 percent.	479.5 5.5				
7.5							
10.0							
12.5							
15.0							
17.5							
20.0							
22.5							
25.0							
27.5							
30.0							

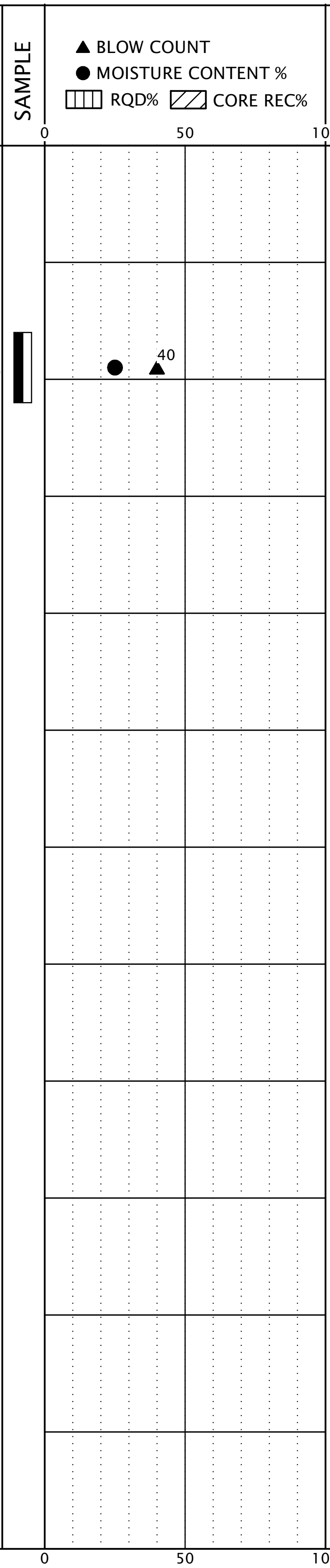


DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: H. Herinckx COMPLETED: 01/20/21

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

GEODESIGN INC AN NV5 COMPANY	LANECOSD-1-01	**BORING B-8**	
	MARCH 2021	CAMAS RIDGE ELEMENTARY SCHOOL EUGENE, OR	**FIGURE A-8**

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Brown, sandy CLAY to CLAY with sand (CL); moist (topsoil to 10 inches, 2-inch-thick root zone).	499.0		Soil description based on cuttings.
2.5	Dense, orange-brown with orange and black mottled, clayey SAND (SC) to sandy CLAY (CL), trace organics; moist, sand is fine to coarse, white zeolites (weathered sandstone).	496.5 2.5	P200	Infiltration test at 4.0 feet. P200 = 52%
5.5	Exploration completed at a depth of 5.5 feet. Hammer efficiency factor is 85 percent.	493.5 5.5		



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: H. Herinckx

COMPLETED: 01/21/21

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

BORING BIT DIAMETER: 6 inches

GEODESIGN INC
AN NV5 COMPANY

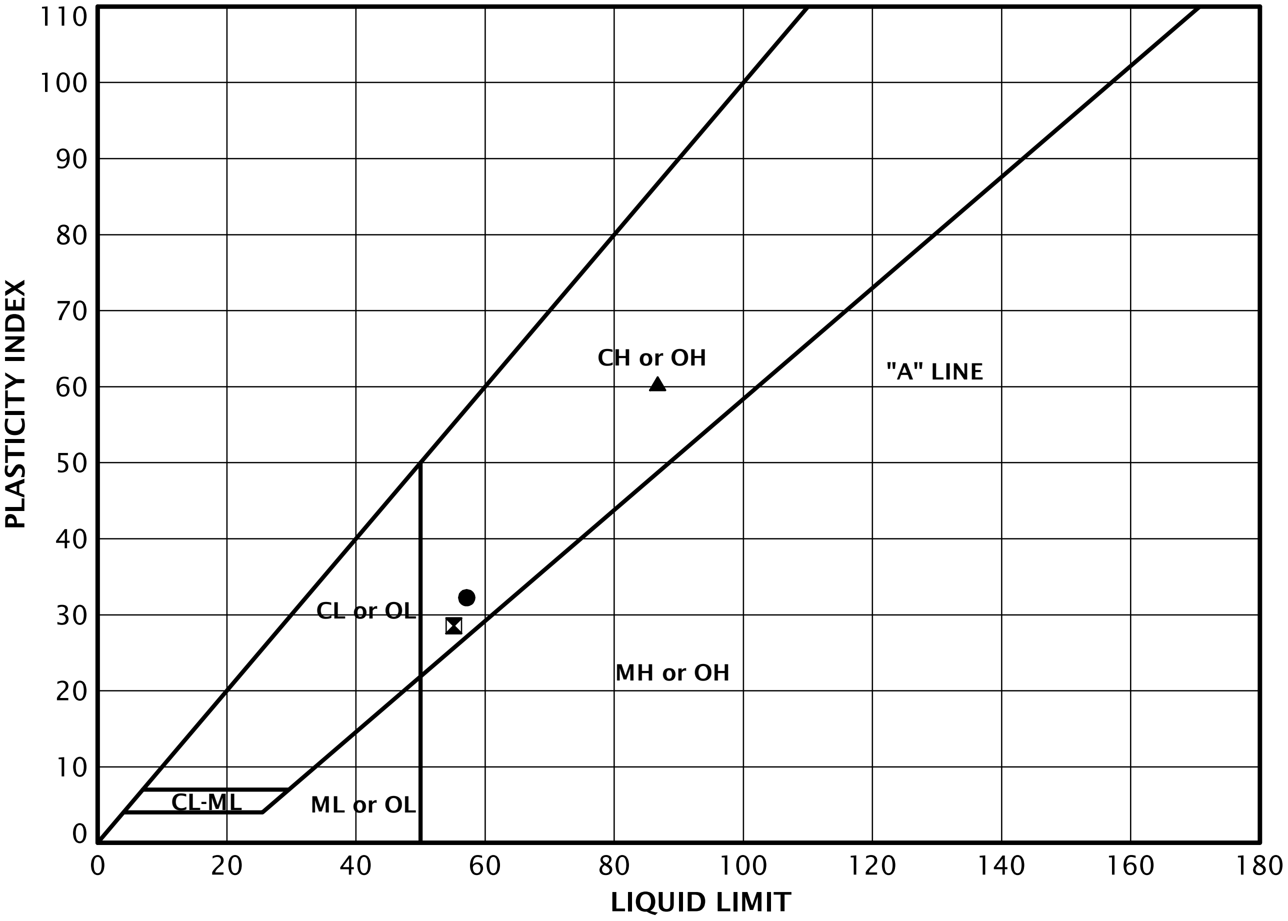
LANECOSD-1-01

MARCH 2021

BORING B-9

CAMAS RIDGE ELEMENTARY SCHOOL
EUGENE, OR

FIGURE A-9



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-4	2.5	37	57	25	32
☒	B-5	5.0	37	55	27	28
▲	B-6	5.0	35	87	26	61

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LANECOSD-1-01

MARCH 2021

ATTERBERG LIMITS TEST RESULTS

CAMAS RIDGE ELEMENTARY SCHOOL
EUGENE, OR

FIGURE A-10

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-4	2.5	471.5	37					57	25	32
B-5	5.0	482.0	37					55	27	28
B-6	5.0	494.0	35					87	26	61
B-7	4.0	468.0	33				68			
B-8	4.0	481.0	41				70			
B-9	4.0	495.0	25				52			

GEODESIGN INC
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LANECOSD-1-01

MARCH 2021

SUMMARY OF LABORATORY DATA

CAMAS RIDGE ELEMENTARY SCHOOL
EUGENE, OR

FIGURE A-11

Summary of SPT Test Results

Project: WSSC-8-04, Test Date: 12/27/2018

EMX: Maximum Energy

ETR: Energy Transfer Ratio - Rated

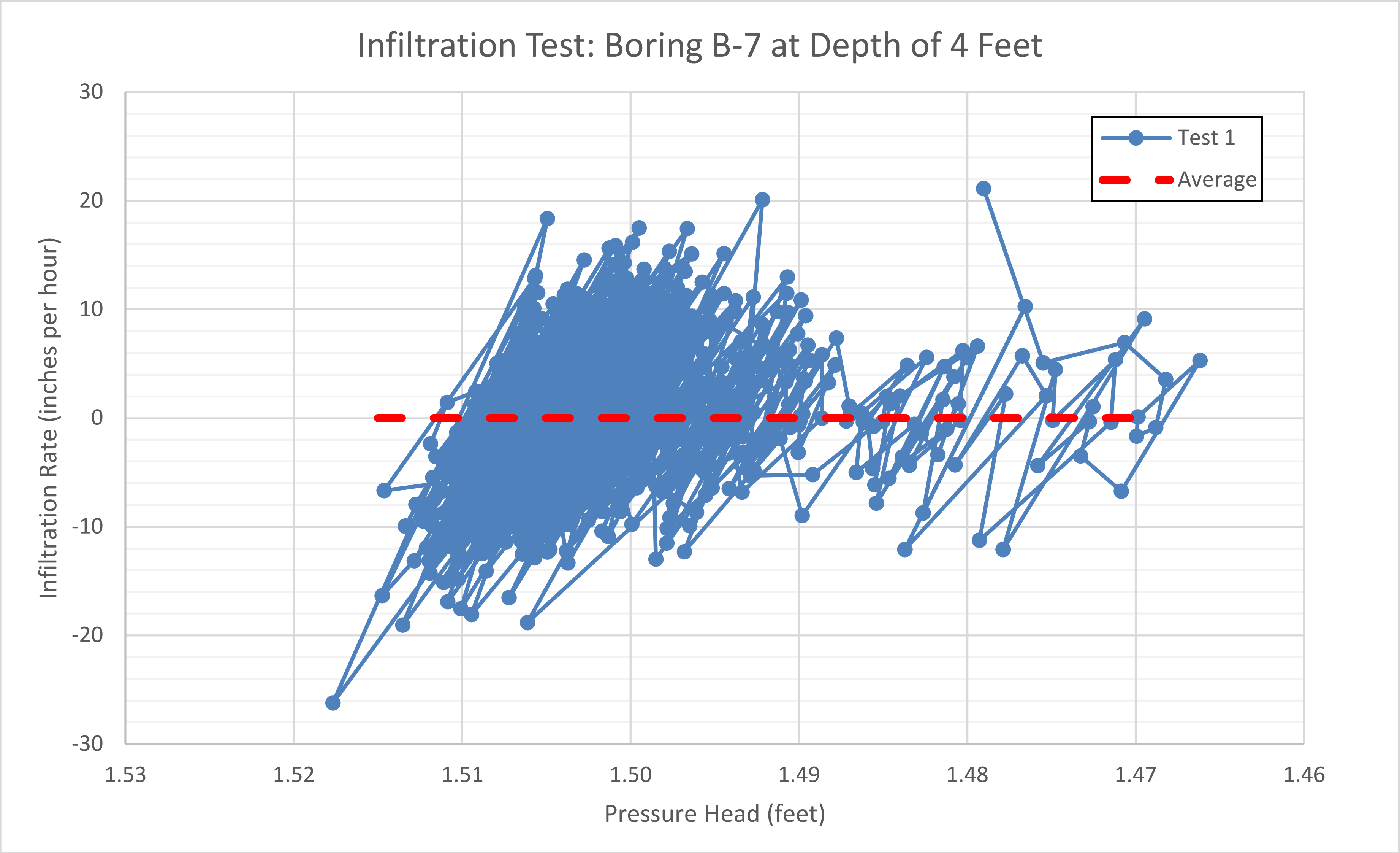
Start Depth ft	Final Depth ft	**N Value**	**N60 Value**	Average EMX ft-lb	Average ETR %
25.00	26.50	**0**	**0**	0.00	0.0
30.00	31.50	**0**	**0**	0.00	0.0
35.00	36.50	**0**	**0**	0.00	0.0
40.00	41.50	**31**	**43**	297.64	85.0
			Overall Average Values:	297.64	85.0
			Standard Deviation:	3.78	1.1
			Overall Maximum Value:	303.37	86.7
			Overall Minimum Value:	289.04	82.6

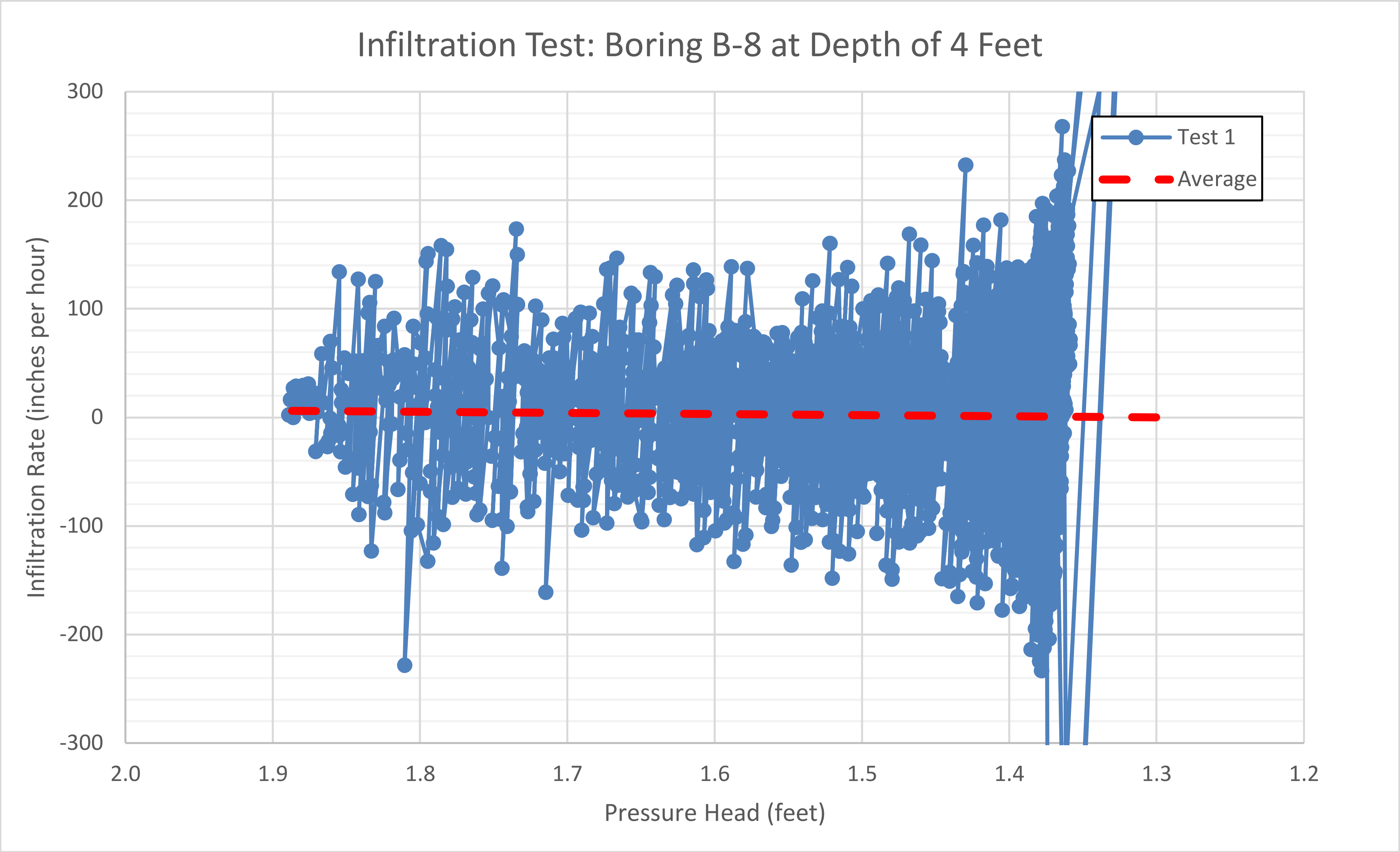
APPENDIX B

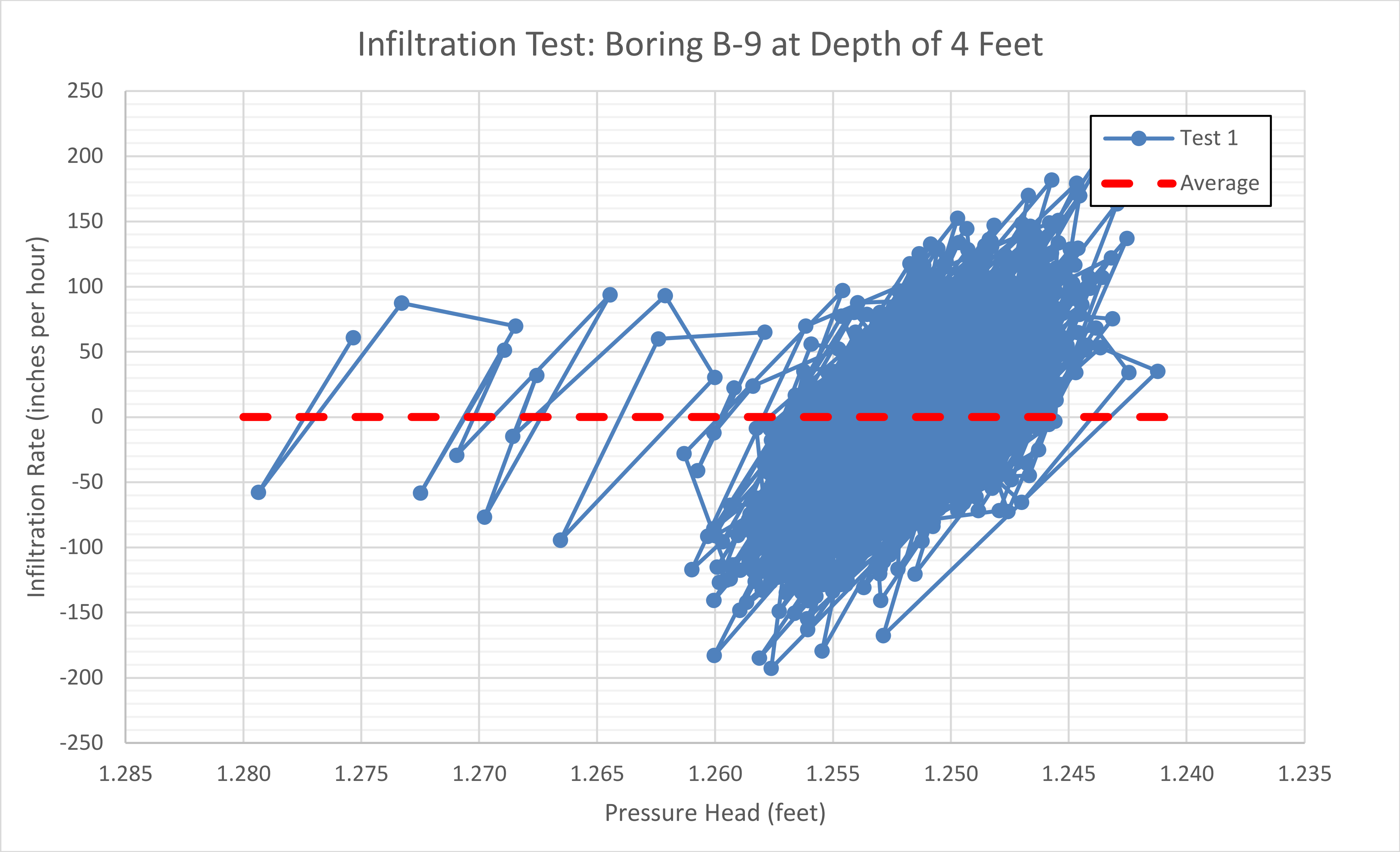
APPENDIX B

INFILTRATION TESTING

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 in borings B-7, B-8, and B-9 with a water head of approximately 1.5 to 2 feet to simulate infiltration swale conditions. We collected water level readings using an electronic water level indicator data logger. The apparent scatter in the data is due to the high frequency of readings collected by the data logger and the negligible infiltration that was taking place. We added a trend line to the plot to show the average of the data.


Infiltration Test: Boring B-7 at Depth of 4 Feet
Infiltration Rate (inches per hour)
30
20
10
0
-10
-20
-30
1.53
1.52
1.51
1.50
1.49
1.48
1.47
1.46
Pressure Head (feet)
Test 1
Average


Infiltration Test: Boring B-8 at Depth of 4 Feet
300
200
100
0
-100
-200
-300
Infiltration Rate (inches per hour)
2.0
1.9
1.8
1.7
1.6
1.5
1.4
1.3
1.2
Pressure Head (feet)
Test 1
Average


Infiltration Test: Boring B-9 at Depth of 4 Feet
Infiltration Rate (inches per hour)
250
200
150
100
50
0
-50
-100
-150
-200
-250
1.285
1.280
1.275
1.270
1.265
1.260
1.255
1.250
1.245
1.240
1.235
Pressure Head (feet)
Test 1
Average

APPENDIX C

APPENDIX C

BENCHMARK GEOLABS LABORATORY TESTING

Benchmark Geolabs of McMinnville, Oregon, performed expansion index testing and corrosivity testing on samples that we collected. The results of their laboratory testing are presented in this appendix.

BENCHMARK
GEOLABS

Expansion Index

ASTM D-4829

BGL Job No.:	038-021	**Boring:**	B-03	**Date:**	2/21/2021
Client:	GeoDesign, Inc.	**Sample:**	0	**By:**	PJ
Project Name:	Camas Ridge Elementary	**Depth:**	1.5		
Project No:	LaneCoS-1-01				

Visual Description: Grayish Brown CLAY, trace organics

Processing:

Percent Passing #4 Sieve

Total Air Dry Weight:	2282
Wt. Retained on #4 Sieve:	0
% Retained	0.0
% Passing #4 Sieve:	100.0

Sample Dimensions

Height (in.)= 1.004 **Diameter (in.) =** 4.012

Moisture Calcs

	Initial	**Final**
Tare #		
Wet Wt. + Tare, (gm)	211.31	913.25
Dry Wt. + Tare, (gm)	184.5	777.2
Tare Wt., (gm)	43.6	502.18
Wt. Of Water, (gm)	26.8	136.05
% Water	19.0	49.5

Remolding:

	Initial	**Final**	
Ring & Sample:	696.55	779.2	grams
Ring:	368.09	368.1	grams
Remolded Wet Wt.:	328.46	411.1	grams
Wet Density	98.6	106.8	pcf
Dry Density	82.8	71.4	pcf
% Sat. = $\frac{(2.7)(\text{dry dens.})(m/c)}{168.48 - (\text{dry dens.})}$	49.7	98.3	ASTM Saturation range 48-52%

Expansion Test:

Date	Time	Dial	Delta h, %
2/19/2021	10:38	0.0000	0.000
	10:40	-0.0451	4.492
	10:54	-0.1022	10.179
	11:17	-0.1258	12.530
	13:04	-0.1466	14.602
	15:01	-0.1510	15.040
2/20/2021	7:31	-0.1558	15.518
	9:44	-0.1561	15.548
	11:21	-0.1561	15.548
		Total Dial	15.5

Tested with 1 psi Surcharge

Remarks:

Expansion Index — **Results**

$$\frac{\text{initial dial - final dial}}{\text{initial sample height}} \times 1000$$

EI = 155

BENCHMARK GEOLABS

SERVICE - ACCURACY - QUALITY

Expansion Index

ASTM D-4829

BGL Job No.:	038-021	**Boring:**	B-07	**Date:**	2/20/2021
Client:	GeoDesign, Inc.	**Sample:**	0	**By:**	PJ
Project Name:	Camas Ridge Elementary	**Depth:**	1		
Project No:	LaneCoS-1-01				

Visual Description: Grayish Brown CLAY trace roots

Processing:

Percent Passing #4 Sieve

Total Air Dry Weight:	1510
Wt. Retained on #4 Sieve:	12.1
% Retained	0.8
% Passing #4 Sieve:	99.2

Sample Dimensions

Height (in.)= 1.004 **Diameter (in.) =** 4.012

Moisture Calcs

	Initial	**Final**
Tare #		
Wet Wt. + Tare, (gm)	132.67	524.47
Dry Wt. + Tare, (gm)	117.5	391.6
Tare Wt., (gm)	43.3	127.23
Wt. Of Water, (gm)	15.1	132.88
% Water	20.4	50.3

Remolding:

	Initial	**Final**	
Ring & Sample:	687.66	766.9	grams
Ring:	368.09	368.1	grams
Remolded Wet Wt.:	319.57	398.8	grams
Wet Density	95.9	105.3	pcf
Dry Density	79.7	70.1	pcf
% Sat. = $\frac{(2.7)(\text{dry dens.})(\text{m/c})}{168.48 - (\text{dry dens.})}$	49.4	96.7	ASTM Saturation range 48-52%

Expansion Test:

Date	Time	Dial	Delta h, %
2/17/2021	9:17	0.0000	0.000
	9:37	-0.1040	10.359
	11:00	-0.1311	13.058
	14:38	-0.1345	13.396
	17:49	-0.1354	13.486
2/18/2021	8:00	-0.1368	13.625
	9:08	-0.1368	13.625
		Total Dial	13.6

Tested with 1 psi Surcharge

Remarks:

Expansion Index **Results**

$$\frac{\text{initial dial - final dial}}{\text{initial sample height}} \times 1000$$

EI =	**136**

BENCHMARK GEOLABS

Corrosivity Tests Summary

BGL # 038-021 **Date:** 2/22/2021 **Tested By:** PJ **Checked:** PJ
Client: GeoDesign, Inc. **Project:** Camas Ridge Elementary **Proj. No:** LaneCoSD-1-01
Remarks:

Sample Location or ID			**Resistivity @ 15.5 °C (Ohm-cm)**			**Chloride**	**Sulfate**		**pH**	**ORP**		**Sulfide**	**Moisture**	
			As Rec.	**Min**	**Sat.**	**mg/kg**	**mg/kg**	**%**		(Redox)		Qualitative	At Test	**Soil Visual Description**
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	**%**	
Boring	**Sample, No.**	**Depth, ft.**	ASTM G57	Cal 643	ASTM G57	EPA 300.0	EPA 300.0	EPA 300.0	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
B-03	0	1.5	-	-	568	6	3	0.0003	5.1	-	-	-	43.3	Grayish Brown CLAY w/ roots

APPENDIX D

APPENDIX D

SITE-SPECIFIC SEISMIC HAZARD EVALUATION

INTRODUCTION

The information in this appendix summarizes the results of our site-specific seismic hazard study for the new Camas Ridge Elementary School. The proposed project includes construction of new school buildings for 450 students that will replace the aging existing school buildings that were constructed in 1949. The proposed buildings will likely be two stories. Additional site improvements will include a new parking lot, paved and unpaved play areas, sports fields, and a bus drop-off area. 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 D-1 shows the locations of faults with potential Quaternary movement within a 35-km radius of the site (USGS, 2020a). As shown on the figure, there are no mapped crustal faults within a 35-km radius of the site. Figure D-2 shows the interpreted locations of seismic events that occurred between 1904 and 2020 (USGS, 2020b).

DESIGN EARTHQUAKE

Deaggregation at the approximate fundamental building period of 0.2 second using the USGS Unified Hazard tool (USGS, 2021 [latitude = 44.024915, longitude = -123.076439]) indicates the CSZ comprises approximately 90 percent of the seismic hazard at the site. The remaining approximately 10 percent of the seismic hazard at the site is comprised of deep intraplate events and local fault events.

SEISMIC DESIGN PARAMETERS

We determined the seismic site class by developing a soil profile and assigning shear wave velocities based on measurements from other nearby sites in similar soil conditions. One of these sites was located approximately 2 miles northwest of Camas Ridge Elementary School in downtown Eugene. Another site was located approximately 3.5 miles southeast of Camas Ridge Elementary School in Goshen. Our soil profile and a summary of our calculations are presented in Table D-1. Based on our soil profile and calculations, the site can be classified as seismic Site Class C based on the average shear wave velocity (V_{S30}) being between 1,200 and 2,500 fps.

Table D-1. Site Class Determination

Soil Type	Depth Below Foundation[1] (feet)	Interval (feet)	Shear Wave Velocity (fps)	Interval/Shear Wave Velocity (second)
Fill and Native Clay[1]	0 to 5	5	500	0.0100
Weathered Sandstone	5 to 10	5	800	0.0063
Weathered Sandstone	10 to 15	5	1,000	0.0050
Sandstone	15 to 25	10	1,200	0.0083
Sandstone	25 to 50	25	1,600	0.0156
Sandstone	50 to 100	50	1,800	0.0278
Sum	NA	100	NA	0.0730
Average shear wave velocity in the upper 100 feet below the foundation, Vs_{30} (fps)	NA			1,370
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 D-2 can be used for design of the building. These parameters were obtained from the SEAOC/OSHPD seismic design map tool (SEAOC/OSHPD, 2021).

Table D-2. 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 = 0.694 g	S_1 = 0.398 g
Site Class	C	
Site Coefficient	F_a = 1.222	F_v = 1.500
Spectral Acceleration Parameters	S_{MS} = 0.848 g	S_{M1} = 0.597 g
Design Spectral Acceleration Parameters	S_{DS} = 0.566 g	S_{D1} = 0.398 g
Spectral PGA	0.330 g	
Design Spectral PGA	0.220 g	
MCE_G PGA Adjusted for Site Class Effects[1]	PGA_M = 0.396 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.

REFERENCES

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

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USGS, 2020a. *Quaternary Fault and Fold Database for the United States.* Obtained from website https://www.usgs.gov/natural-hazards/earthquake-hazards/faults, accessed November 10, 2020.

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USGS, 2021. Unified Hazard Tool. Obtained from website https://earthquake.usgs.gov/hazards/interactive/, accessed February 24, 2021.

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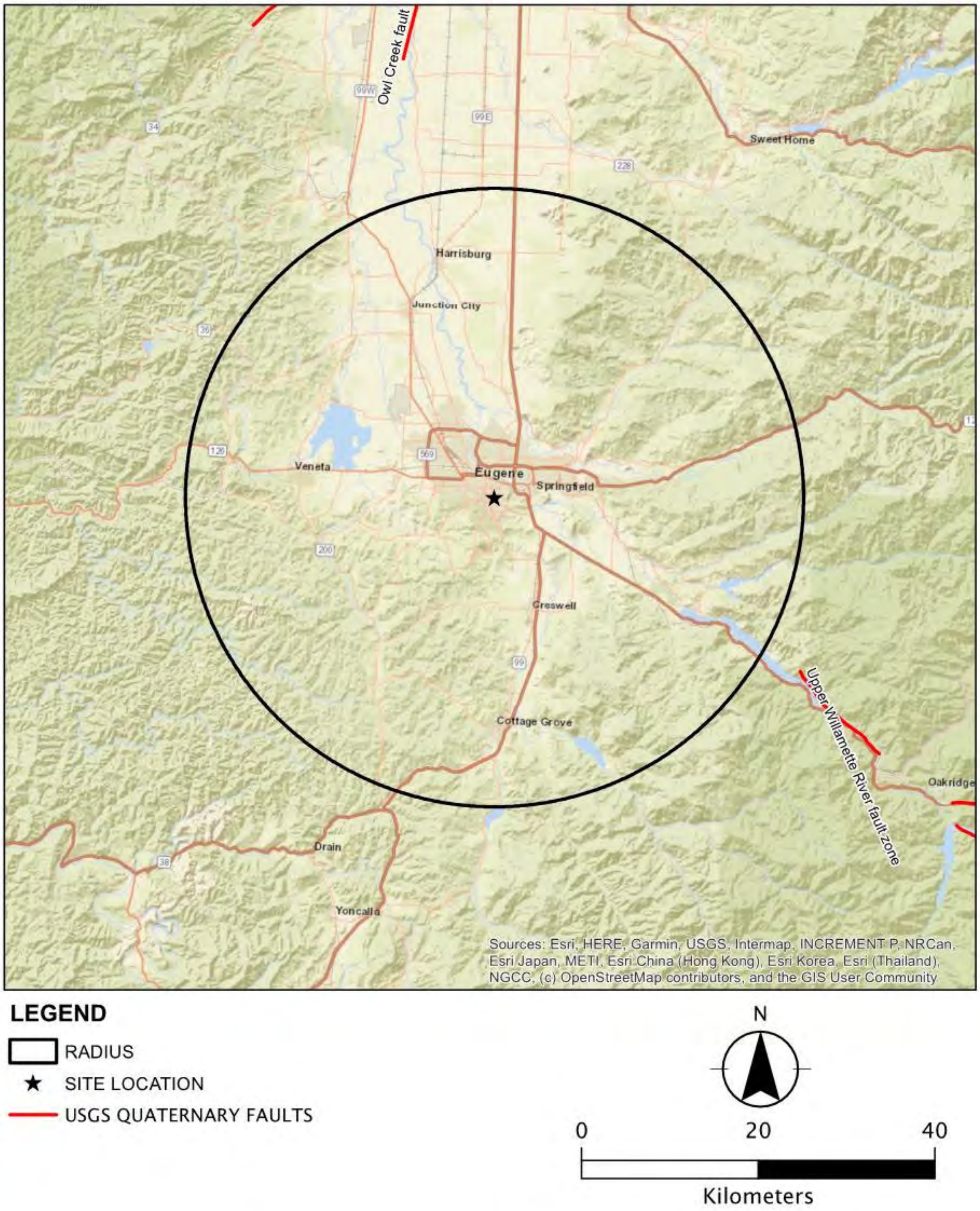

Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community

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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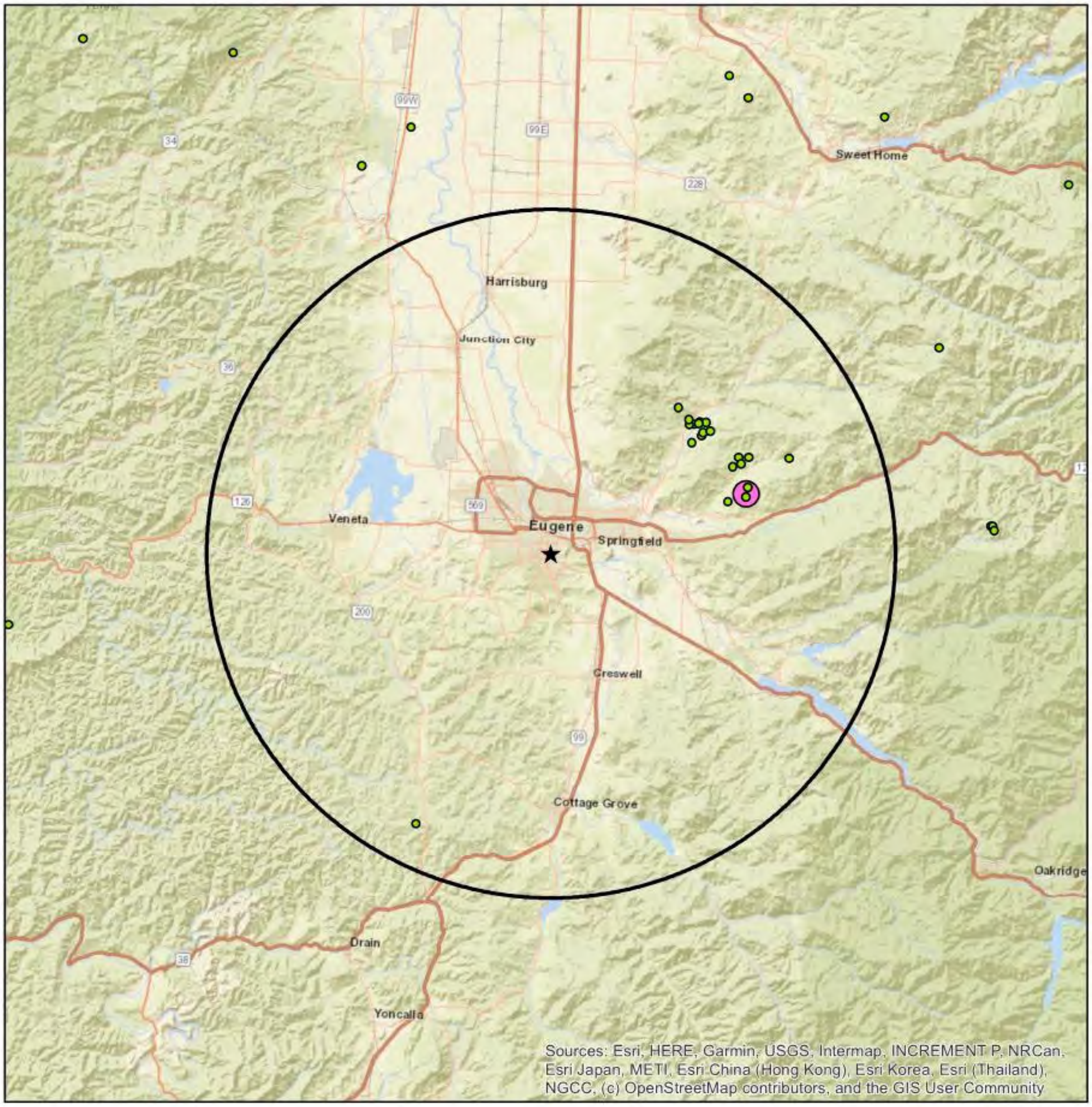
	MARCH 2021	CAMAS RIDGE ELEMENTARY SCHOOL EUGENE, OR	**FIGURE D-1**



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

▭ RADIUS

★ SITE LOCATION

INSTRUMENTAL EARTHQUAKE MAGNITUDE

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

N

0 20 40

Kilometers

USGS, accessed November 9, 2020, 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 1904-2020.

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MARCH 2021

HISTORICAL SEISMICITY MAP

CAMAS RIDGE ELEMENTARY SCHOOL
EUGENE, OR

FIGURE D-2

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