Design Review Preliminary Stormwater Management Report

Edwards Elementary School Additions

Prepared for: BRIC Architects
Prepared by: Andrew Chung, PE
Project Engineer: Mark Wharry, PE

February 2022 | KPFF Project #2100142

kpff

KPFF’S COMMITMENT TO SUSTAINABILITY

As a member of the US Green Building Council, KPFF is committed to the practice of sustainable design and the use of sustainable materials in our work.

When hardcopy reports are provided by KPFF, they are prepared using recycled and recyclable materials, reflecting KPFF’s commitment to using sustainable practices and methods in all of our products.

KPFF'S COMMITMENT TO SUSTAINABILITY

As a member of the US Green Building Council, KPFF is committed to the practice of sustainable design and the use of sustainable materials in our work.

When hardcopy reports are provided by KPFF, they are prepared using recycled and recyclable materials, reflecting KPFF's commitment to using sustainable practices and methods in all of our products.

Table of Contents

Tables and Figures

Appendices

PROJECT OVERVIEW AND DESCRIPTION

Purpose of this Report

This preliminary report describes the stormwater management design strategies for the proposed development. The basis of this report is the City of Newberg Public Works Design and Construction Standards and the requirements outlined therein. The purpose of the proposed stormwater management facilities is to protect existing stormwater infrastructure and improve the overall health of the watershed.

Project Location

The property is located on the west side of the Edwards Elementary School site at 715 E 8th Street in Newberg, Oregon. Currently, the project site drains to the north and south with no stormwater management facilities (see Vicinity Map). The limits of work are approximately 3.71 acres. Within the limits of work is 1.08 acres of existing impervious area is to be removed. Existing impervious area includes existing buildings, asphalt pavement, concrete curb/sidewalks, and compacted gravel. The proposed development within the limits of work includes 2.25 acres of new impervious area, including new building structures, new concrete pavement, and new asphalt. This equates to a 1.17 acre net increase in impervious area. Per City of Newberg Design Standards, this project is required to treat and flow control a minimum of 1.17 acres of impervious area. See Appendix A and Appendix B for net impervious area calculations and area of management.

Existing vs. Post-construction Conditions

The existing site consists of an existing elementary school and parking lot sloping to the south with roughly a 6-foot grade difference. The proposed condition will consist of two new additions to the school site, covered play, pedestrian plaza, north parking lot, and revised bus drop off at front entry. Stormwater from the site will be collected and discharged to the new public storm main extension in E 8th Street. Based on site topography, some runoff will have to be directed to the north. Shallow storm infrastructure in E 6th Street makes it infeasible to route runoff through management facilities. The storm main in E 6th Street will be extended for collection of some runoff. See Methodology section for net increase in impervious area being managed.

METHODOLOGY

Drainage and Conveyance

The proposed onsite development consists of approximately 2.25 acres of impervious area. Shallow existing storm infrastructure and site topography makes it infeasible to collect and manage the entire redeveloped area. As shown in Appendix B2, 1.40 acres of impervious area will be treated and flow-controlled equating to managing more than the required 1.17 acre minimum. The remaining area will be collected by storm piping and directed to the public storm main. The proposed conveyance system will be designed according to the City of Newberg Public Works Design and Construction Standards Section 4.5.4. Flow rates will be calculated using the Santa Barbara Unit Hydrograph (SBUH) method. Pipe size and conveyance capacity will utilize Manning's Equation. Runoff curve numbers will be per NRCS TR-55 and are shown in the stormwater management assumptions in Appendix C. Storm pipes will be sized to convey the 25-year design storm.

Appendix C storm assumptions note that the water quantity flow control requirements should a detention system be implemented.

Infiltration

The SCS soil map in Appendix A shows type C/D soils throughout the site. Infiltration test results suggest 0.25 in/hr. A factor of safety of 2 will be applied to the open bottom underground detention facility.

Proposed Stormwater Management

KPFF proposes one (1) vegetated rain garden with 3H:1V side slope that will hold the water quality volume and infiltrate through the growing media into the underground detention facility. This is located at the south end of the south addition. This facility is sized to treat approximately 1.16 acres of impervious area. Overflow structures will be installed to direct stormwater runoff above the water quality design storm to the underground detention facility. A two filter Water Quality Catch Basin (WQCB) is proposed to treat the redeveloped front entry drop off. This structure also connects to the underground detention facility. Due to shallow existing stormwater infrastructure, a 24-inch perforated detention pipe system is proposed. Preliminary calculations suggest 8,220 cf of underground detention will be required. Peak flows leaving the site will be controlled by an orifice flow control tee in a 60-inch manhole. These facilities are preliminary sized based on the current preliminary grading plan. A final storm report at the time of permit submittal will demonstrate adequate sizing to manage the water quality storm, flow control peak runoffs, and safely convey peak flows from the City of Newberg design storms.

Stormwater Quality Treatment Standards

The water quality design storm is 1.0 inch in 24 hours. All water quality facilities will be designed to capture and treat this 24-hour design storm event. The facilities will be sized to prevent flooding/ponding during the 100-year design storm.

Stormwater Quantity Standards

Appendix C notes the City of Newberg standards for flow control. Peak flows will be controlled to meet pre-developed peak flows for half the 2-year, 2-year, 10-year, and 25-year design storms. Safe overflow of the 100-year design storm will also be evaluated on the treatment facility and underground detention facility.

ANALYSIS

Stormwater Management

The stormwater management facilities will be sized utilizing the AutoDesk Storm and Sewer Analysis 2021 program. This program will run SBUH peak flow calculations from a prescribed basin area to adequately size storm facilities to pass design storms up to the 100-years. The software will size the rain garden, WQCB, underground detention facility, orifice flow control tee, and conveyance systems. The computer software will input the City of Newberg design storms (Table 1) to calculate peak flows.

TABLE 1: City of Newberg Design Storms

Design Storm	24 hr rain fall (in)
Water Quality	1
Half the 2-year	1.25
2-year	2.5
10-year	3.5
25-Year	4.0
100-Year	4.5

Downstream Analysis

A downstream analysis will be performed per Section 4.5.9 of the City Design standards. This analysis will assess the condition and capacity of the downstream storm drainage system up to a 1/4 mile past the project site.

CONCLUSION

Based on the requirements of the City of Newberg and the engineering assumptions and calculations detailed in this preliminary report, all facility components will have enough capacity to treat and convey runoff from an area that exceeds the net increase in impervious area and not have adverse effects on the site or stormwater infrastructure. Runoff from the area hatched in Appendix B2 will be routed to the water quality facilities for treatment with overflow to a new underground detention facility and connect to the new public stormwater main extension in E 8th Street. A public stormwater main extension in E 6th Street will collect some runoff from the proposed redevelopment. Final design calculations, report, and downstream analysis will be provided at final permit submittal.

2100142-pm

Appendix A

Existing Impervious Areas

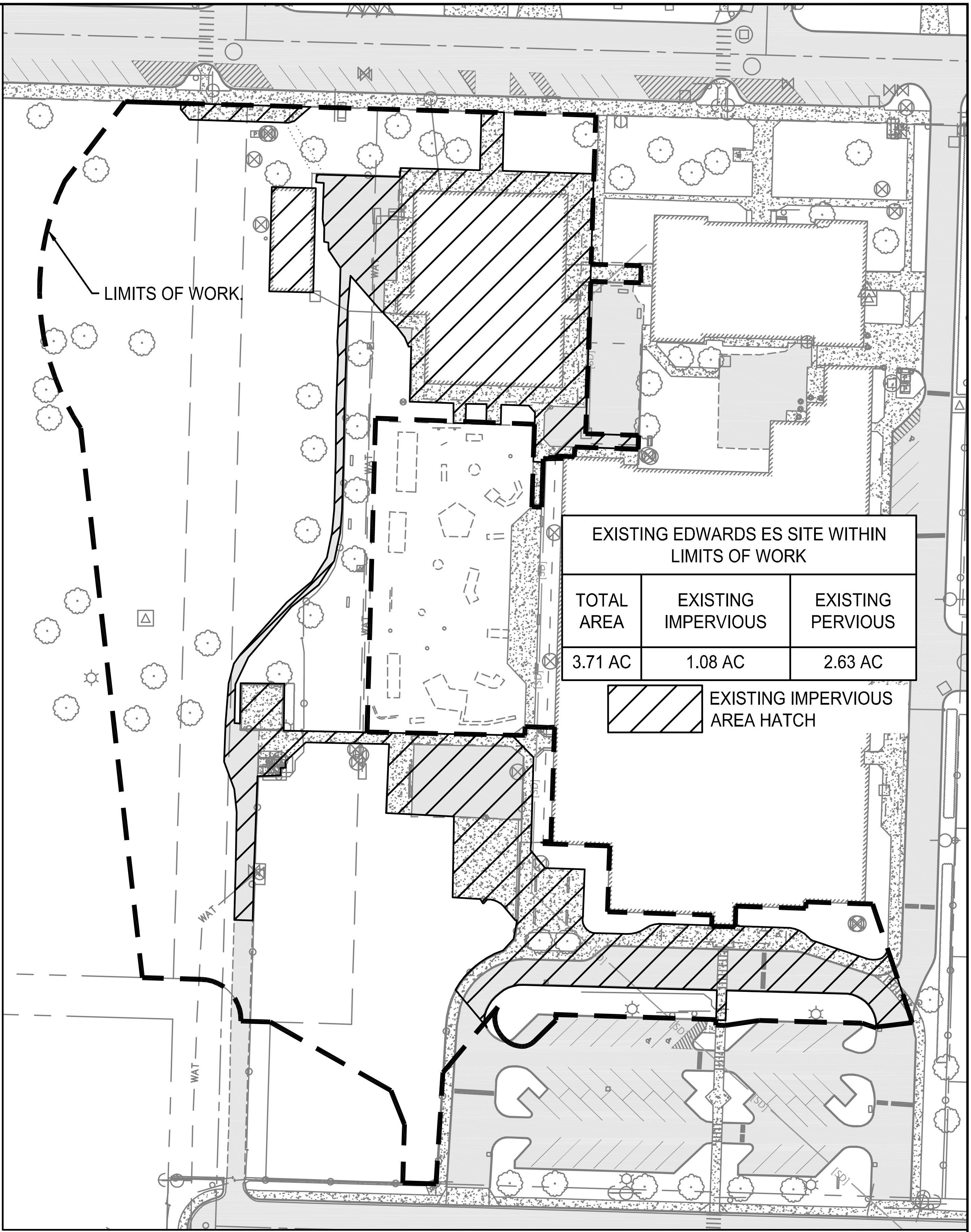
Proposed Impervious Areas

Vicinity Map

Geotechnical Report

SCS Soil Map

Page left blank for double-sided printing.



EXISTING EDWARDS ES SITE WITHIN LIMITS OF WORK		
TOTAL AREA	EXISTING IMPERVIOUS	EXISTING PERVIOUS
3.71 AC	1.08 AC	2.63 AC

▨ EXISTING IMPERVIOUS AREA HATCH

EDWARDS ELEMENTARY SCHOOL EXISTING IMPERVIOUS AREA

SCALE: 1" = 80'

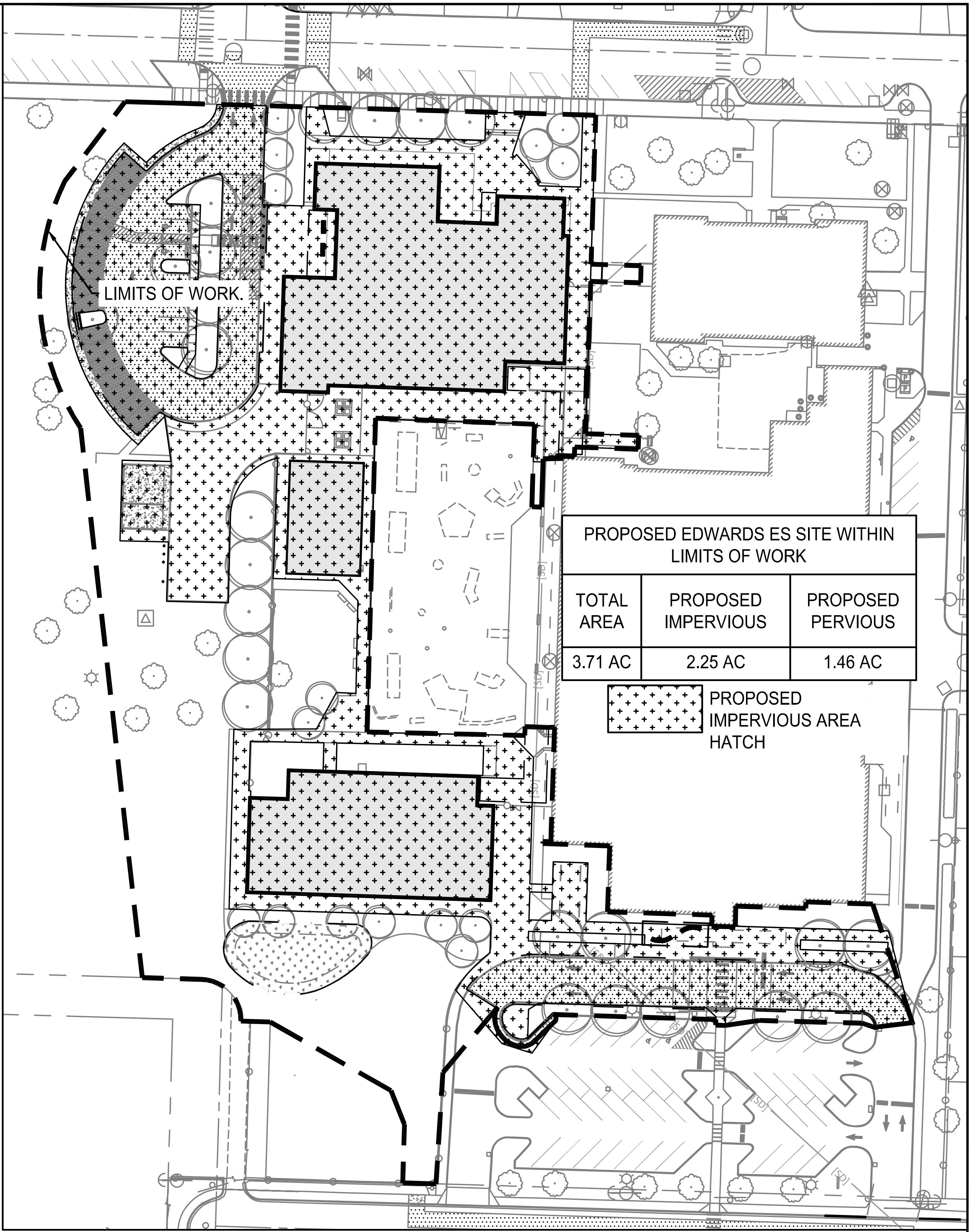
SHEET NO.

EXH-A1

File: N:\c\p\2021\2100142-NSD-Edwards-ES\PROJ-INFO\Planning-Design\03-STORM\20210701-AREA-TAKEOFF.dwg TAB:EXH-A1

11X17-EXHIBIT

Plotted: 11/29/21 at 4:36pm By: AChung



PROPOSED EDWARDS ES SITE WITHIN LIMITS OF WORK		
TOTAL AREA	PROPOSED IMPERVIOUS	PROPOSED PERVIOUS
3.71 AC	2.25 AC	1.46 AC

▨ PROPOSED IMPERVIOUS AREA HATCH

EDWARDS ELEMENTARY SCHOOL PROPOSEDIMPERVIOUS AREA

SCALE: 1" = 80'

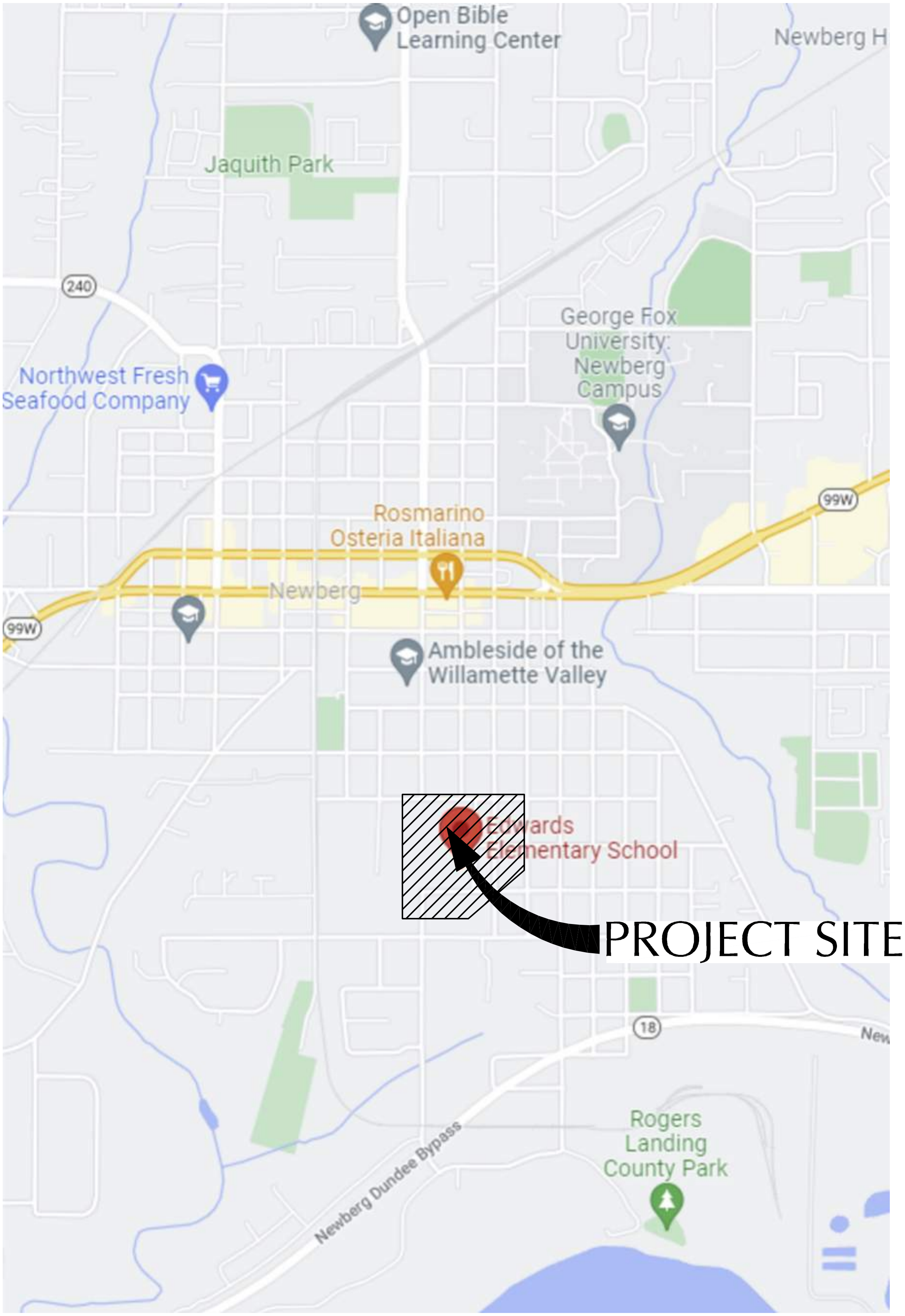
SHEET NO.

EXH-A2

File: N:\c\p\2021\2100142-NSD-Edwards-ES\PROJ-INFO\Planning-Design\03-STORM\20210701-AREA-TAKEOFF.dwg TAB:EXH-A2

11X17-EXHIBIT

Plotted: 2/10/22 at 3:14pm By: AChung



EDWARDS ELEMENTARY SCHOOL VICINITY MAP

SCALE: NTS

SHEET NO.

EXH-A3

File: N:\c\p\2021\2100142-NSD-Edwards-ES\PROJ-INFO\Planning-Design\03-STORM\20210701-AREA-TAKEOFF.dwg TAB:EXH-A3

11X17-EXHIBIT

Plotted: 11/29/21 at 4:48pm By: AChung

Geotechnical Investigation and Site-Specific Seismic-Hazard Evaluation Edwards Elementary School 2022 Addition and Improvements

715 E 8th Street
Newberg, Oregon

October 13, 2021

Prepared for
Cornerstone Management Group, Inc.
Newberg School District
714 E 6th Street
Newberg, OR 97132

Prepared by

GRI

9750 SW Nimbus Avenue
Beaverton, OR 97008-7172
(503) 641-3478 | www.gri.com

TABLE OF CONTENTS

TABLES

APPENDICES

FIGURES

As requested, GRI completed a geotechnical investigation for the Edwards Elementary School 2022 Addition and Improvements Project located in Newberg, Oregon. The Vicinity Map, Figure 1, shows the general location of the site. The purpose of our investigation was to evaluate subsurface conditions at the site and develop geotechnical recommendations for use in the design and construction of the proposed improvements. The investigation included a review of available existing geotechnical information for the site and surrounding areas, subsurface explorations, laboratory testing, and engineering analyses. This report describes the work accomplished and provides conclusions and recommendations for use in the design and construction of the proposed project.

1 PROJECT DESCRIPTION

Based on the information provided by Cornerstone Management Group, Inc., we understand improvements to the school will include demolition of portions of the existing school with two new single-story wood-frame or masonry buildings, a new covered play area, playground and sports field improvements, additional parking areas, and new pavement areas for parent drop-off and bus loop. Additionally, stormwater facility improvements are being considered for the project.

We anticipate the structure will be designed in accordance with the 2018 International Building Code with modifications by the 2019 Oregon Structural Specialty Code (OSSC), which references the 2016 American Society of Civil Engineers (ASCE) 7-16 document, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-16), for seismic design. We anticipate the new structures will be on-grade and supported on conventional column- and wall-type spread footings; however, the structural loads for the buildings have not been provided at this time. Based on our understanding of the project, we anticipate the maximum column and wall loads will be less than approximately 200 kips and 3 kips/foot, respectively.

We anticipate minor cuts and fills will be required to reach final site grades for the proposed site improvements. New parking will be provided at the southeast corner of the school property and be accessed from East 8th Street. A new parent drop-off will be provided north of the school buildings and be accessed from East 6th Street. We anticipate the entrance driveways and parking lots will be paved with asphalt concrete (AC), and areas subjected to heavy traffic loads and hardscape areas such as trash enclosures and sidewalks will be paved or surfaced with portland cement concrete (PCC).

2 SITE DESCRIPTION

2.1 General

The Edwards Elementary School property is bordered by East 6th Street to the north, East 7th Street and residential housing to the east, East 8th Street and East 9th Street to the south, and South Blaine Street to the west. School buildings, parking lots, and driveways occupy the eastern portion of the campus and grass athletic fields occupy the central and western portion of the site. A review of satellite imagery and our observations on site indicates the ground surface gently slopes downward to the west.

2.2 Geology

Published geologic mapping indicates the site is mantled with Missoula flood deposits, locally referred to in the project area as the Willamette Silt Formation. In general, Willamette Silt is composed of beds and lenses of clay, silt, and sand. Stratification within this formation commonly consists of 4- to 6-inch-thick beds, although in some areas, the clay, silt, and sand are massive, and the bedding is indistinct or nonexistent (Wells et al., 2018). The Hillsboro Formation, which typically consists of stiff to very stiff brown to gray clay, commonly underlies the Willamette Silt in this area (Ma et al., 2009).

3 SUBSURFACE CONDITIONS

3.1 General

Subsurface materials and conditions at the site were investigated on September 1, through September 3, 2021, with four machine-drilled geotechnical borings, designated B-1 through B-4; three machine-drilled pavement borings, designated PB-1 through PB-3; three machine-drilled infiltration test borings, designated I-1 through I-3; two hand-augered borings, designated HA-1 and HA-2; one cone penetration test (CPT) probe, designated CPT-1; five Kessler dynamic cone penetration tests (KDCPs), designated KDCP-1 through KDCP-5; and falling-head infiltration testing in borings I-1 through I-3. The approximate locations of the explorations are shown on Figure 2. The machine-drilled borings were advanced to depths ranging from about 6.5 feet to 41.5 feet. The hand-augered borings were advanced to a depth of approximately 5.3 feet. The CPT probe was advanced to a depth of approximately 64 feet. The KDCPs were advanced to a depth of about 3 feet. The falling-head infiltration tests were performed at depths ranging from approximately 4.7 feet to 10 feet.

A discussion of the field-exploration and laboratory-testing programs completed for this investigation is provided in Appendix A. Logs of the machine-drilled and hand-augered borings are provided on Figures 1A through 12A, logs of the CPT probe are provided on Figures 13A and 14A, and logs of the KDCPs are provided on Figures 15A through 19A. The terms and symbols used to describe the soil encountered in the explorations are defined in Tables 2A and 3A and on the attached legend.

3.2 Sampling

Disturbed and undisturbed soil samples were generally obtained from the machine-drilled borings at 2.5-foot intervals of depth in the upper 15 feet and at 5-foot intervals below 15 feet. Disturbed soil samples were generally obtained from the hand-augered borings at 2-foot intervals of depth or where subsurface conditions changed. Standard penetration tests (SPTs) were conducted while collecting disturbed samples from the drilled borings. The SPT N-values provide a measure of relative density of granular soils and the relative consistency of cohesive soils. Additional details of the sampling and SPTs are provided in Appendix A.

3.3 Soils

For the purpose of discussion, the soils disclosed by our investigation have been grouped into the following categories based on their physical characteristics and engineering properties:

a. GRAVEL and CLAY (Fill)
b. SILT and CLAY (Willamette Silt)
c. CLAY and SAND (Hillsboro Formation)

The following paragraphs provide a description of the soil units encountered in the explorations completed by GRI for this investigation.

a. GRAVEL and CLAY (Fill)

Fill consisting of gravel underlain by clay were encountered at the ground surface in the hand-augered boring HA-2 and extends to a depth of about 4 feet. The gravel fill is silty and contains some fine- to coarse-grained sand, and the gravel is subrounded. The clay fill is silty and contains some fine- to coarse- grained sand, is black, and has an organic odor. The natural moisture content of the clay fill is about 26%.

b. SILT and CLAY (Willamette Silt)

Silt and clay, interpreted to be the Willamette Silt Formation, was encountered beneath the fill in hand-augered boring HA-2, and at the ground surface in all other explorations. Interpretation of CPT probe CPT-1 indicates the Willamette Silt Formation extends to a depth of about 35.2 feet and the silt and clay of the Willamette Silt Formation extends to a depth of about 40 feet, 35 feet, and 38 feet in borings B-1, B-3, and B-4, respectively. Pavement borings PB-1 through PB-3 were terminated in the silt and clay at depths of about 14 feet to 16.5 feet. The infiltration-test borings were terminated in the silt and clay at depths of about 6.5 feet to 11.5 feet, and the hand-augered borings were terminated in the silt and clay at a depth of about 5.3 feet. The silt typically contains a trace to some fine-grained sand, though the silt may contain sandy layers, or the sand may be fine to medium grained. An interbedded sand layer was encountered in borings B-3 and B-4 at depths of about 13.5 feet to 16.2 feet. The silt contains a trace clay to clayey and ranges in color from brown to dark gray and may be mottled rust. The clay is typically silty and contains up to a trace to a trace of fine-grained sand. A 4- to 5-inch-thick, heavily rooted zone was encountered at the ground surface in the silt and clay. Based on SPT N-values, Torvane shear-strength values, and CPT tip-resistance values, the relative consistency of the silt and clay ranges from very soft to hard and is typically soft to medium stiff.

The natural moisture content of the silt and clay ranges from about 12% to 42%. Results of Atterberg-limits testing completed on select clay samples are provided on Figure 20A and indicate the clay has medium plasticity. One-dimensional consolidation testing was completed on select samples of silt obtained from boring B-1 at depths of about 14.5 feet and 26.5 feet and boring B-3 at a depth of about 8.5 feet. Test results indicate the silt soil is moderately to heavily overconsolidated and has relatively low compressibility in the preconsolidated range of pressures and a moderate to high compressibility in the normally consolidated ranges of pressures, see Figures 21A through 23A.

c. CLAY and SAND (Hillsboro Formation)

Clay and sand interpreted to be the Hillsboro Formation were encountered beneath the Willamette Silt at depths of about 40 feet, 35 feet, and 38 feet in borings B-1, B-3, and B-4, respectively, and from the interpretation of CPT probe CPT-1 at a depth of about 35.2 feet below the ground surface. The Hillsboro Formation extends to the maximum depth explored of about 64 feet in CPT probe CPT-1. Borings B-1, B-3, and B-4 were terminated in the Hillsboro Formation clay and sand at depths of about 41.5 feet, 36.5 feet, and 41.5 feet, respectively. The clay contains some silt to silty, some fine-grained sand to sandy, and is gray to dark gray. The sand is clayey, contains a trace silt, and is gray mottled yellow-brown. Based on SPT N-values, Torvane shear-strength values, and CPT tip-resistance values, the relative consistency of the clay is stiff to hard and based on SPT N-values and CPT tip-resistance values, the sand is medium dense to dense.

The natural moisture content of the clay ranges from about 26% to 27%, and the natural moisture content of the sand is about 28%. One-dimensional consolidation testing was completed on a select sample of clay obtained from boring B-4 at a depth of about 38.5 feet. Test results indicate the silt soil is moderately overconsolidated and has a relatively low compressibility in the preconsolidated range of pressures and a moderate compressibility in the normally consolidated ranges of pressures, see Figure 24A.

3.4 Groundwater

Based on nearby well logs and published U.S. Geological Survey (USGS) groundwater studies in the vicinity of the project area, the estimated depth to the regional groundwater level in the area ranges from about 30 feet to 50 feet below the ground surface (USGS, 2021).

The geotechnical borings B-1 through B-4 were completed using mud-rotary drilling techniques, which do not allow the direct measurement of groundwater levels. Groundwater was not observed in the machine-drilled borings completed using hollow-stem auger techniques, borings PB-1 through PB-3, borings I-1 through I-3, or the hand-augered borings, HA-1 and HA-2, within the depths of exploration ranging from about 5.3 feet to 16.5 feet. CPTs are in-situ tests, which do not allow for the direct measurement of groundwater levels. Our experience in similar soils and the project vicinity indicates perched groundwater likely occurs above the static groundwater levels indicated by USGS in the site vicinity throughout the year in the silt and clay soils that mantle the site. We anticipate the local perched groundwater level typically occurs within depths of about 10 feet to 15 feet below the ground surface during the normally dry summer months and may approach the ground surface during the wet winter and spring months or during periods of heavy or prolonged precipitation.

3.5 Infiltration Testing

Falling-head infiltration testing was completed at the site on September 1 through 3, 2021, in general conformance with the City of Portland 2020 *Stormwater Management Manual* (SMM) using the encased falling-head method outlined in Section 2.3.2 of the manual. The test locations were designated I-1 through I-3 in shallow boreholes at depths of about 4.7 feet to 10 feet below existing site grades. Additional details of the infiltration testing are provided in Appendix A. The average unfactored, field-measured infiltration rates are tabulated below.

INFILTRATION

Table 3-1: INFILTRATION TEST RESULTS

Test No.	Depth of Infiltration Test, feet	Average Field Infiltration Rate, inches/hour	Soil Classification	Fines Content (% Passing No. 200 Sieve)
I-1	4.7	0.25	Clayey SILT, trace fine-grained sand	93
I-2	9.8	< 0.25	SILT, some clay, trace fine-grained sand	92
I-3	10	< 0.25	SILT, trace fine-grained sand and up to trace clay	91

3.6 California Bearing Ratio

The data from the KDCP test probes were used to estimate the California bearing ratio (CBR) value of the in-situ subgrade soils for use in pavement design. The CBR values estimated using the KDCP test probe blow counts are tabulated below.

Table 3-2: RECOMMENDED CBR VALUES

Probe	Boring Location	CBR Value	Soil Classification
KDCP-1	PB-1	5	SILT, some clay, trace fine-grained sand
KDCP-2	PB-2	5	Clayey SILT, up to trace fine-grained sand
KDCP-3	-	6	SILT, some fine-grained sand
KDCP-4	-	5	SILT, some clay, trace fine-grained sand
KDCP-5	PB-3	4	Clayey SILT, trace fine-grained sand

4 CONCLUSIONS AND RECOMMENDATIONS

4.1 General

contradictory statement

Subsurface explorations completed for this investigation indicate the site is mantled with typically soft to medium-stiff silt and clay soils of the Willamette Silt Formation overlying typically stiff to very stiff soils of the Hillsboro Formation to the maximum depth explored. We anticipate the local groundwater level typically occurs at a depth of 10 feet to 15 feet below the ground surface during the normally dry summer months; however, groundwater may approach the ground surface during the wet winter and spring months or periods of prolonged or intense precipitation.

In our opinion, foundation support for new structural loads can be provided by conventional column- and wall-type spread footings established in firm, undisturbed native soil or compacted structural fill. The primary geotechnical considerations associated with the construction of the proposed improvements include the potential presence of fill soils within the footprint of the proposed improvements, the presence of fine-grained soils that are moisture sensitive, and the potential for shallow, perched-groundwater conditions. The following sections of this report provide our conclusions and recommendations for use in the design and construction of the project.

4.2 Seismic Considerations

4.2.1 General

We understand the project will be designed in accordance with the 2019 OSSC, which references ASCE 7-16, for seismic design. We understand the proposed improvements will be considered a special-occupancy structure as defined by Oregon Revised Statute (ORS) 455.447 and will require a site-specific seismic-hazard evaluation. A site-specific seismic-hazard evaluation was completed for the project to fulfill the requirements of amended Section 1803 of the 2019 OSSC for special-occupancy structures. Details of the site-specific seismic-hazard evaluation and the development of the recommended response spectrum are provided in Appendix B.

4.2.2 Mapped Acceleration Parameters

The ASCE 7-16 S_S and S_1 mapped spectral response acceleration parameters for the site located at the approximate latitude and longitude coordinates of 45.2951° N and 122.9727° W are 0.85 g and 0.41 g, respectively, for Site Class B/C, or bedrock conditions.

4.2.3 Site Class

Based on the subsurface conditions disclosed by the explorations, and in accordance with Section 20.4 of ASCE 7-16, the site is classified as Site Class D, or a stiff-soil site, based on an average shear-wave velocity (field-measured shear-wave velocity [V_S]) in the upper 100 feet of the soil profile. The code-based Site Class D conditions are appropriate for design of the structure.

4.2.4 *Site Coefficients*

Due to the S_1 acceleration parameter being greater than or equal to 0.2 g, Section 11.4.8 of ASCE 7-16 requires a ground-motion hazard analysis unless the seismic response coefficient C_s is determined in accordance with Exception 2 of Section 11.4.8 of ASCE 7-16. Assuming the seismic response coefficient, C_s is determined in accordance with Exception 2 of Section 11.4.8 of ASCE 7-16, the site coefficients F_a and F_v were determined from code-tabulated values to be 1.16 and 1.89, respectively, in accordance with Section 11.4 of ASCE 7-16. The site coefficients F_a and F_v were used to develop the Site Class D, Risk-Targeted Maximum Considered Earthquake (MCE_R)-level spectrum in accordance with Section 11.4 of ASCE 7-16.

4.2.5 *Recommended Seismic Design Parameters*

The design-level response spectrum is calculated as two-thirds of the ground-surface MCE_R spectra. The recommended MCE_R- and design-level spectral-response parameters for Site Class D conditions are provided below in Table 4-1.

Table 4-1: RECOMMENDED SEISMIC DESIGN PARAMETERS (2019 OSSC/ASCE 7-16)

Seismic Parameter	Recommended Values*
Site Class	D
MCE_R 0.2-Sec Period Spectral Response Acceleration, S_{MS}	0.99 g
MCE_R 1.0-Sec Period Spectral Response Acceleration, S_{M1}	0.78 g
Design-Level 0.2-Sec Period Spectral Response Acceleration, S_{DS}	0.66 g
Design-Level 1.0-Sec Period Spectral Response Acceleration, S_{D1}	0.52 g

Note: *Exception 2 of Section 11.4.8 should be considered when evaluating base shear calculations in Section 12.8.

4.2.6 *Liquefaction/Cyclic Softening*

The potential for liquefaction and/or cyclic softening at the site was evaluated using the simplified method based on procedures recommended by Idriss and Boulanger (2008) with subsequent revisions (2014). This method uses peak ground acceleration (PGA) to predict the cyclic shear stresses induced by the earthquake. The USGS National Seismic Hazard Mapping Project (NSHMP) was used to determine the contributing earthquake magnitudes representing seismic exposure of the site for the Maximum Considered Earthquake Geometric Mean (MCE_G) hazard level. A crustal event on the Newberg Fault and an event on the Cascadia Subduction Zone (CSZ) were determined to represent sources of seismic shaking.

The results of our evaluation indicate there is a potential that the interbedded lenses of sand below the groundwater surface at the site could experience limited liquefaction and zones of the low-plasticity sandy silt below the groundwater surface at the site could experience limited cyclic softening. Our analysis indicates the potential for up to about 1 inch of seismically induced settlement may occur during the earthquake and after earthquake shaking has ceased. Additional details regarding our liquefaction and/or cyclic softening evaluation are provided in Appendix B. Discussion of seismically induced building-foundation settlement is presented in the Foundation Support section of this report.

4.2.7 *Other Seismic Hazards*

Based on subsurface conditions and site topography, the risk of earthquake-induced slope instability and/or lateral spreading is low. The risk of damage by a tsunami and/or seiche at the site is absent. The USGS considers the Newberg Fault, located approximately 0.1 miles (0.2 kilometers) northeast of the site and the Mount Angel Fault, located about 10.5 miles (16.9 kilometers) southeast of the project site, to be the closest crustal fault sources contributing to the overall seismic hazard at the site. The CSZ is mapped approximately 77 kilometers west of the site (Petersen et al., 2014). Unless occurring on a previously unmapped or unknown fault, the risk of fault rupture at the site is low.

4.3 Earthwork

4.3.1 *General*

The fine-grained soils that mantle the site are moisture sensitive and perched groundwater may approach the ground surface during the wet winter and spring months and periods of heavy or prolonged precipitation. Therefore, it is our opinion earthwork can be completed most economically during the dry summer months, typically extending from June to mid-October. It has been our experience that the moisture content of the upper few feet of fine-grained soils will decrease during extended warm, dry weather. However, the moisture content of the soil below this depth tends to remain relatively unchanged and well above the optimum moisture content for compaction. As a result, the contractor must use construction equipment and procedures that prevent disturbance and softening of the subgrade soils. To minimize disturbance of the moisture-sensitive, fine-grained soils, site grading can be completed using track-mounted hydraulic excavators. The excavation should be finished using a smooth-edged bucket to produce a firm, undisturbed surface. It may also be necessary to construct granular haul roads and work pads concurrently with excavation to minimize subgrade disturbance. If the subgrade is disturbed during construction, soft, disturbed soils should be overexcavated to firm soil and backfilled with structural fill. We recommend the contract documents provide unit costs for overexcavation and structural backfill.

4.3.2 Site Preparation

The ground surface within all building areas, paved areas, walkways or hardscapes, and areas to receive structural fill should be stripped of existing vegetation, surface organics, and loose surface soils. We anticipate stripping up to a depth of about 6 inches to 8 inches will likely be required to extend below the heavily rooted zone; however, deeper grubbing may be required to remove brush and tree roots. All trees, brush, and surficial organic material should be removed from within the limits of the proposed improvements. Existing underground improvements, foundations, and infrastructure should be fully removed. Excavations required to remove existing improvements, brush, and trees should be backfilled with structural fill. Organic strippings should be disposed of off site or stockpiled for use in landscaped areas.

Following stripping or excavation to subgrade level, the exposed subgrade should be evaluated by a qualified geotechnical engineer or engineering geologist. Proof rolling with a loaded dump truck may be part of this evaluation. Any soft areas or areas of unsuitable material disclosed by the evaluation should be overexcavated to firm material and backfilled with structural fill.

4.3.3 Prior Site Development

Due to previous development at the site and surrounding area, there is the potential to encounter fill soils or existing improvements. It should be anticipated that some overexcavation of subgrade may be required. In addition, site improvements within previously developed areas include the risk of encountering undocumented or poorly documented improvements and infrastructure. Although not encountered within the subsurface explorations completed at the site, the possibility does exist to encounter undocumented fill soils or existing underground improvements such as abandoned utilities or agricultural drainage tiles. If encountered, buried structures should be overexcavated and backfilled with structural fill in accordance with Section 4.5 of this report.

4.3.4 Site Grading

Final grading across the site should provide for positive drainage of surface water away from exposed slopes and surfaces to reduce the potential for erosion. Prior to placing pavement base course aggregate, subgrade should be sloped to a minimum 0.5% slope to aid in drainage. Permanent cut-and-fill slopes should not be steeper than 2H:1V (Horizontal to Vertical) and should be protected with vegetation to reduce the risk of surface erosion due to rainfall.

4.3.5 Granular Work Pads

If construction occurs during wet-ground conditions, granular work pads will be required to protect the underlying subgrade and provide a firm working surface for construction activities. In our opinion, a 12- to 18-inch-thick granular work pad should be sufficient to prevent disturbance of the subgrade by lighter construction equipment and limited traffic by dump trucks. Haul roads and other high-density traffic areas, including the use of Gradalls and forklifts, will require a minimum of 18 inches to 24 inches of fragmental rock, up to 6-inch nominal size, to reduce the risk of subgrade deterioration. The use of woven geotextile fabric or geogrid over the subgrade may reduce the need for maintenance during construction. Granular haul roads and work pads can also be constructed by placing a thickened section of the pavement section crushed-rock base (CRB) and subsequently spreading and grading the excess CRB after earthwork is complete.

4.3.6 Cement Amendment

Cement amendment may be an option to stabilize subgrade soils during periods when the soil cannot be suitably moisture conditioned. It has also become common to cement treat subgrade soils for all-weather sports fields. Typically, 5% to 8% cement (by dry weight of soil) mixed to a depth of 12 inches to 14 inches below subgrade is sufficient to provide a stable platform for construction. Additional cement may be necessary for drying if the in-situ moisture content of the subgrade is well above its optimum moisture content for compaction. Cement amendment may allow the contractor to extend the construction season to the typically wet winter to spring months; however, installation of the cement amendment should be accomplished during the drier months. The installation timeline may be extended slightly (i.e., into shoulder season) with the understanding of a likely higher failure rate, but installation should not be conducted during the winter or during prolonged periods of wet weather. The cement-amended soil should be compacted with a sheepsfoot or segmented-pad roller to achieve compaction of about 95% of the maximum dry density as determined by ASTM D558. After the cement-amended area is graded, a smooth-drum roller should be used to produce a smooth, compacted surface. All compaction and grading operations should be completed within four hours of soil mixing and tilling with the cement. Final cement-amended grades should be sloped to a minimum 0.5% slope to aid in drainage and avoid water ponding. Cement-amended soils should be cured for a minimum of five days to increase their strength gain prior to evaluation, being trafficked by any equipment, or placement of the granular base course. After curing, the smooth, compacted surface should be evaluated to determine suitability. Proof rolling with a fully loaded dump truck may be part of this evaluation. Soft areas or areas of insufficient cement should be overexcavated and/or retreated. To support construction equipment, the cement-amended subgrade should be capped with an approximately 6- to 12-inch-thick section of relatively clean, crushed rock that has less than about 5% passing the No. 200 sieve (washed analysis). If the cement amendment option is selected, we recommend additional testing be considered to define the proper cement content of the soil that will achieve a minimum compressive strength of 100 pounds per square inch (psi).

4.4 Excavation

4.4.1 General

We anticipate the maximum height of excavations to establish finished site grades will generally be less than 5 feet and the depth of utility excavations may be on the order of 10 feet to 15 feet. The method of excavation and the design of excavation support are the responsibilities of the contractor and are subject to applicable local, state, and federal regulations, including the current Occupational Safety and Health Administration (OSHA) excavation and trench safety standards. The means, methods, and sequencing of construction operations and site safety are also the responsibilities of the contractor. The information provided below is for the use of our client and should not be interpreted to imply we are assuming responsibility for the contractor's actions or site safety.

4.4.2 Groundwater Management

Depending on the time of year the work is completed, perched groundwater may be encountered in the excavations. Groundwater seepage, running-soil conditions, and unstable excavation sidewalls or excavation subgrades, if encountered during construction, will require dewatering of the excavation and sidewall support. The impact of these conditions can be reduced by completing excavations during the summer months, when perched groundwater levels are lowest and by limiting the depths of the excavations.

We should have a dewatering note to the contractor. Means & Methods based on found field conditions

We anticipate perched groundwater inflow, if encountered, can generally be controlled by pumping from sumps. To facilitate dewatering, it will be necessary to overexcavate the base of the excavation to permit installation of a granular working blanket. We estimate the required thickness of the granular working blanket will be on the order of 1 foot or as required to maintain a stable excavation base. The actual required depth of overexcavation will depend on the conditions exposed in the excavations and the effectiveness of the contractor's dewatering efforts. The thickness of the granular blanket must be evaluated based on field observations during construction. We recommend the use of relatively clean material, such as 2- to 4-inch-minus crushed rock with less than about 5% passing the No 200 sieve (washed analysis), for this purpose. The use of a geotextile fabric over the excavation base will assist in subgrade stability and dewatering.

4.4.3 Temporary Excavations

The inclination of temporary excavation slopes will depend, in part, on the perched-groundwater conditions encountered at the time of construction and the contractor's ability to control these conditions. In this regard, we anticipate temporary excavation slopes can be cut at 1.5H:1V to a maximum depth of 15 feet if groundwater levels are maintained at least 2 feet below the bottom of the excavation. Flatter slopes will be necessary if significant seepage conditions are encountered. Some minor amounts of sloughing, slumping, or running of temporary slopes should be anticipated shortly after groundwater seepage occurs. A blanket of relatively clean, well-graded, crushed rock placed on the slopes may be required to reduce the risk of raveling-soil conditions if temporary excavation slopes encounter perched groundwater. We recommend the use of relatively clean, free-draining material, such as 2- to 4-inch-minus crushed rock, for this purpose. The thickness of the granular blanket should be evaluated based on actual conditions but would likely be in the range of 12 inches to 24 inches.

In our opinion, the short-term stability of temporary slopes will be adequate if surcharge loads due to construction traffic, vehicle parking, material laydown, etc., are maintained at an equal distance to the height of the slope away from the top of the open cut. Additional lateral loading due to surcharge loads can be evaluated using the criteria shown on Figure 3. Other measures that should be implemented to reduce the risk of localized failures of temporary slopes include: 1) using geotextile fabric to protect the exposed cut slopes from surface erosion; 2) providing positive drainage away from the tops and bottoms of the cut slopes; 3) constructing and backfilling walls as soon as practical after completing the excavation; 4) backfilling overexcavated areas as soon as practical after completing the excavation; and 5) periodically monitoring the area around the top of the excavation for evidence of ground cracking. It must be emphasized that following these recommendations will not guarantee sloughing or movement of the temporary cut slopes will not occur; however, the measures should serve to reduce the risk of a major slope failure. It should also be realized that blocks of ground and/or localized slumps may tend to move into the excavation during construction.

4.4.4 Utility Excavations

In our opinion, there are three major considerations associated with the design and construction of new utilities:

1. Provide stable excavation sideslopes or support for trench sidewalls to minimize loss of ground.
2. Provide a safe working environment during construction.
3. Minimize post-construction settlement of the utility and ground surface.

According to current OSHA regulations, the majority of the fine-grained soils encountered in the explorations may be classified as Type C. In our opinion, trenches less than 4 feet deep that do not encounter groundwater may be cut vertically and left unsupported during the normal construction sequence, assuming trenches are excavated and backfilled in the shortest possible sequence. Excavations that encounter groundwater or are more than 4 feet deep should be laterally supported or alternatively provided with sideslopes of 1.5H:1V or flatter to a maximum depth of 15 feet. In our opinion, adequate lateral support may be provided by common methods, such as the use of a trench shield or hydraulic shoring systems. If deeper excavations are required, GRI should be contacted to reevaluate our temporary slope recommendations.

4.5 Structural Fill

4.5.1 General

We anticipate minor amounts of structural fill may be required to achieve finished floor elevation for the structure and finished grades for the associated improvements. In general, structural fills should consist of imported or on-site, organic-free soils and should extend a minimum horizontal distance of 5 feet beyond the edge of new foundations and 1 foot beyond the limits of ancillary improvements, such as the edge of new sidewalks, hardscapes, or pavements.

4.5.2 On-Site Fine-Grained Fill

The use of on-site, fine-grained soils for structural fill material is typically limited to the dry summer months when the moisture content of these soils can be controlled to within about 3% of optimum. However, the natural moisture content of the on-site soils will probably exceed the optimum moisture content throughout the majority of the year; therefore, some aeration and drying will be required to meet the requirements for proper compaction. The required drying can best be accomplished by spreading the material in thin lifts and tilling. Drying rates are dependent on weather factors such as wind, temperature, and relative humidity. Fine-grained soils used as structural fill should have a maximum size of 2 inches and should be placed in 8-inch-thick lifts (loose) and compacted with a segmented-pad or sheepsfoot roller to at least 95% of the maximum dry density as determined by ASTM D698. If fine-grained soils are not compacted at a moisture content within about 3% of optimum, the specified density cannot be achieved, and the fill material will be relatively weak and possibly compressible. Cement amendment of on-site, fine-grained soils may be an option to stabilize structural fill soils during periods when the soil cannot be suitably moisture conditioned.

On-site, fine-grained soils and site strippings free of debris may be used as fill in non-structural landscaped areas where overlying hardscapes such as sidewalks or pavements will not be constructed. These materials should be placed at about 90% of the maximum dry density determined by ASTM D698. The moisture contents of soils placed in landscaped areas are not as critical as the moisture contents of soils placed in structural areas, provided construction equipment can effectively handle the materials. However, it should be understood that fine-grained soils compacted to less than 95% of the maximum dry density determined by ASTM D698 or at a moisture content outside 3% the optimum may result in excessive settlement of fill soils.

4.5.3 Imported Granular Fill

During wet conditions, imported granular material would be most suitable for the construction of the structural fills. Granular material, such as sand, sandy gravel, or crushed rock, with a maximum size of 2 inches and less than 5% passing the No. 200 sieve (washed analysis) would be suitable structural fill material. Granular fill should be placed in lifts and compacted with vibratory equipment to at least 95% of the maximum dry density determined in accordance with ASTM D698. Compaction of granular fill material greater than about 1.5 inches shall be evaluated based on visual observation of compaction equipment and proof rolls. Appropriate lift thicknesses will depend on the type of compaction equipment used. For example, if hand-operated, vibratory-plate equipment is used, lift thicknesses should be limited to about 6 inches to 8 inches. Based on field-density testing, if smooth-drum vibratory rollers are used, lift thicknesses up to about 12 inches may be appropriate, and if backhoe- or excavator-mounted vibratory plates are used, lift thicknesses of up to about 2 feet may be acceptable.

4.5.4 Utility Trench Backfill

All utility trench excavations within building, hardscape, and pavement areas should be backfilled with relatively clean, granular material, such as sand, sandy gravel, or crushed rock, of up to 1½-inch maximum size and having less than about 5% passing the No. 200 sieve (washed analysis). The bottom of the excavation should be thoroughly cleaned to remove loose materials and the utilities should be underlain by a minimum 6-inch thickness of bedding material. The granular backfill material should be compacted to at least 95% of the maximum dry density as determined by ASTM D698 in the upper 5 feet of the trench and at least 92% of this density below a depth of 5 feet. The use of hoe-mounted vibratory-plate compactors is usually the most efficient for this purpose. Flooding or jetting as a means of compacting the trench backfill should not be permitted.

4.6 Foundation Support

4.6.1 General

Based on our understanding of the project, we anticipate the maximum column and wall loads will be less than approximately 200 kips and 3 kips/foot, respectively. In our opinion, the proposed structural loads can be supported on conventional spread and wall footings in accordance with the following design criteria. Excavations for footings will encounter subsurface conditions consisting of silt. Fill soils were encountered in our explorations to a depth of about 4 feet in portions of the site. Due to prior development at the site, fill soils may also be encountered in other portions of the site. If encountered, fill soils or otherwise unsuitable material encountered at foundation subgrade level should be overexcavated and replaced with crushed-rock structural fill compacted in lifts to at least 95% of the maximum dry density as determined by ASTM D698. Overexcavations should extend horizontally beyond footings and settlement-sensitive structures, as indicated on Figure 4. We recommend the contract documents provide unit costs for subgrade overexcavation and structural backfill. Replacing undocumented fill or otherwise unsuitable soils with controlled-density fill or lean-mix concrete are alternatives to backfilling with crushed-rock structural fill.

4.6.2 Footing Subgrade Preparation

The base of all new footings should be established in the native soil that mantles the site or in structural fill. The base of all new footings should be established at a minimum embedment depth of 18 inches below the lowest adjacent finished grade. The footing width should not be less than 24 inches for isolated column footings and 18 inches for wall footings. Final excavations for all foundations should be made with a smooth-edged bucket, and all footing subgrades should be observed by a member of GRI's geotechnical engineering staff. Soft or otherwise unsuitable material encountered at foundation subgrade level should be overexcavated and backfilled with granular structural fill. Local areas of softer subgrade may require deeper overexcavation and should be evaluated by a member of GRI's geotechnical engineering staff. Our experience indicates the subgrade soils are easily disturbed by excavation and construction activities. Due to these considerations, we recommend installing a minimum 3-inch-thick working pad layer of compacted crushed rock in the bottom of all footing excavations. Relatively clean, ¾-inch-minus crushed rock having less than about 5% fines passing the No. 200 sieve (washed analysis) is suitable for this purpose.

4.6.3 Allowable Bearing Pressure

Our allowable bearing pressures are based on the estimated column and wall loads and the results of our subsurface explorations. Footings established in accordance with the above criteria in the native silt soil or compacted structural fill can be designed based on an allowable soil bearing pressure of 3 kips per square foot. This value applies to the total of dead load and/or frequently applied live loads and can be increased by one-third for the total of all loads: dead, live, and wind or seismic.

4.6.4 Static and Dynamic Settlement

We estimate the total static settlement of spread and wall footings designed in accordance with the recommendations presented above will be less than 1 inch for footings supporting column and wall loads of up to 200 kips and 3 kips/foot, respectively. Differential static settlements between adjacent, comparably loaded, and similarly supported footings should be less than half the total settlement. Differential static settlements between footings supported on differing subsurface conditions may approach total settlements.

As discussed in the Seismic Considerations section of this report, our analysis indicates up to about 1 inch of dynamic settlement could occur following a code-based seismic event. Based on the thicknesses of the non-liquefiable soils that mantle the site and the discrete, thin soil lenses subject to liquefaction and/or cyclic softening, we estimate the potential for significant ground manifestation of the seismically induced settlement is generally low. For foundation design purposes, we recommend assuming differential seismic settlement will approach 50% of the calculated total seismic settlement over the length of the building.

Subsection 12.13.9.2 of ASCE 7-16 provides guidance for acceptable limits of seismic differential settlement for different types of structures and different risk categories. In our opinion, based on review of Table 12.13 3 of ASCE 7-16 and our experience with similar Risk Category III structures, up to ½ inch of seismic differential settlement over the building dimension is consistent with current standards of practice for a life safety performance level. However, the structural engineer should determine if the structure can accommodate the estimated total and differential seismic settlements. Tying the foundations together with a network of grade beams, as identified in Subsection 12.13.9.2.1.1 of ASCE 7-16, will reduce the potential adverse effects associated with differential movement.

4.6.5 Horizontal Forces

Horizontal shear forces can be resisted partially or completely by frictional forces developed between the base of the footings and the underlying soil and by soil passive resistance. The total frictional resistance between the footing and the soil is the normal force times the coefficient of friction between the soil and the base of the footing. We recommend ultimate values of 0.35 and 0.55 for the coefficient of friction for footings cast on firm, native soil, and granular crushed rock structural fill material, respectively. The normal force is the sum of the vertical forces (dead load plus real live load). If additional lateral resistance is required, passive earth pressures against embedded footings can be computed based on an equivalent fluid. Footing pressures for horizontal backfill conditions can be computed having a unit weight of 300 pounds per cubic foot (pcf) against native soil or compacted structural fill. These design passive earth pressures would be applicable only if the footing is cast neat against undisturbed native soil or if backfill for the footings is placed as granular structural fill and assumes up to 0.01H inches of lateral movement of the structure will occur to develop this resistance, where H is the depth of embedment to the bottom of the footing. This value also assumes the permanent ground surface in front of the foundation is horizontal, i.e., does not slope downward away from the toe of the footing.

4.7 Subdrainage/Floor Support

To provide a capillary break and reduce the risk of damp floors, slab-on-grade floors established at or near adjacent final site grades should be underlain by a minimum 8 inches of free-draining, clean rock. This material should consist of angular rock such as 1½- to ¾-inch, open-graded crushed rock with less than 2% passing the No. 200 sieve (washed analysis) and should be capped with a 2-inch-thick layer of compacted, ¾-inch-minus crushed rock to improve workability, as indicated on Figure 5. The slab base course section should be placed in one lift and compacted to at least 95% of the maximum dry density (ASTM D698) or until well keyed. In areas where floor coverings will be provided or moisture-sensitive materials stored, it would be appropriate to also install a vapor-retarding membrane. The membrane should be installed as recommended by the manufacturer. In addition, a foundation drain should be installed around the building perimeter to collect water that could potentially infiltrate beneath the foundations. All groundwater collected should be drained by gravity or pumped from sumps into an approved stormwater disposal facility. If the water is pumped, an emergency power supply or battery back-up should be included to prevent flooding due to power loss during storm or high-groundwater events. The perimeter foundation drain should be placed at the base of the footing and embedded within free-draining, clean, angular rock, such as 1½- to ¾-inch crushed rock with less than 2% passing the No. 200 sieve (washed analysis).

For the structures established below final site grades, the floors should be underlain by a subdrainage system in addition to the slab base course section and vapor-retarding membrane, as indicated on Figure 5. A subdrainage system will reduce the buildup of hydrostatic pressures on the floor slab and the risk of groundwater entering through embedded walls and floor slabs. Beneath embedded floor slabs, the center-to-center spacing of the underslab drainpipes should not exceed 20 feet. All groundwater collected should be drained by gravity or pumped from sumps into an approved stormwater disposal facility. If the water is pumped, an emergency power supply or battery back-up should be included to prevent flooding due to power loss during storm or high-groundwater events.

In our opinion, it is appropriate to assume a coefficient of subgrade reaction, k, of 150 pounds per cubic inch to characterize the subgrade support for point loading with 10 inches of compacted crushed rock beneath the floor slab.

4.8 Hardscape Improvements

We anticipate hardscape improvements will include new hard surface playground areas and sidewalks that will be limited to pedestrian loading. Asphalt concrete (AC) hardscape and portland cement concrete (PCC) hardscape sections should be underlain by a minimum 6-inch thickness of crushed rock base course. We recommend the crushed rock be up to about 1½-inch maximum size, have less than about 5% passing the No. 200 sieve (washed analysis), and at least two fractured faces. Prior to placement of the base course, the subgrade should be evaluated by a member of GRI's geotechnical engineering staff. Soft or otherwise unsuitable material should be overexcavated and replaced with compacted structural fill. The base course section should be compacted to at least 95% of the maximum dry density as determined by ASTM D698. Proof rolling with a loaded dump truck may be part of this evaluation.

The hardscape sections provided are not designed to support vehicular traffic or construction traffic loading. If vehicular traffic or wet-weather hardscape construction is considered, it will be necessary to increase the thickness of base course to support the loading and protect the subgrade from disturbance, as discussed in the Earthwork section of this report.

4.9 Retaining Walls

We anticipate portions of the improvements, such as Americans with Disabilities Act access ramps, may be partially embedded and may require embedded walls. For this report, we assumed any site retaining walls would consist of conventional cast-in-place walls supported on spread foundations. Foundation design and subgrade preparation should conform to the recommendations provided above for foundation support.

Design lateral earth pressures for retaining walls depend on the type of construction, i.e., the ability of the wall to yield. Possible conditions are 1) a wall laterally supported at its base and top and therefore unable to yield to the active state; and 2) a retaining wall, such as a typical cantilever or gravity wall, which yields to the active state by tilting about its base. A conventional basement wall and cantilever retaining wall are examples of non-yielding and yielding walls, respectively. To account for seismic loading, the Agusti and Sitar method (2013) was used to develop lateral earth pressures on permanent embedded structures. For completely drained, horizontal backfill, yieldin, and non-yielding walls may be designed based on equivalent fluid unit weights of 35 pcf and 55 pcf, respectively. To account for seismic loading, the earth pressures should be increased by 9 pcf and 19 pcf for yielding and non-yielding walls, respectively. This results in a triangular distribution, with the resultant acting at ⅓H up from the base of the wall, where H is the height of the exposed wall in feet.

The lateral earth pressure design criteria presented above are appropriate if the retaining walls are fully drained. Perched groundwater may occur within the shallow, fine-grained soils and existing utility trenches during periods of prolonged or intense precipitation. Based on these considerations, we recommend the installation of a permanent drainage system behind all of the retaining walls. The drainage system can either consist of a drainage blanket of crushed rock or continuous drainage panels between the retained soil/backfill and the face of the wall. The drainage blanket should have a minimum width of 24 inches and consist of crushed drain rock that contains less than 2% fines content (washed analysis). A nonwoven geotextile fabric should separate the drain rock and wall backfill. A typical drainage system for retaining walls constructed with a drainage blanket is shown on Figure 5. The drainage blanket or drainage panels should extend to the base of the wall, where water should be collected in a perforated pipe and discharged to a suitable outlet, such as a sump or approved storm drain. In addition, the wall design should include positive drainage measures to prevent ponding of surface water behind the top of the wall.

Overcompaction of backfill behind walls should be avoided. Heavy compactors and large pieces of construction equipment should not operate within 5 feet of any embedded wall to avoid the buildup of excessive lateral earth pressures. Compaction close to the walls should be accomplished with hand-operated, vibratory-plate compactors. Overcompaction of backfill could significantly increase lateral earth pressures behind walls. Additional lateral loading to walls due to surcharge loads can be evaluated using the criteria shown on Figure 3.

4.10 Pavement Design

4.10.1 Recommended Design

We anticipate the access roads and parking areas at the site will be subjected to automobile, light, and occasional heavy truck traffic. We anticipate the majority of the site will be paved with AC pavement; however, areas subjected to repeated heavy truck traffic, such as trash-enclosure and service areas, may be paved with PCC pavement. Traffic estimates for the bus loop, access roads, and parking areas are currently unknown. Pavement sections are not designed to support construction traffic loading. Pavements subjected to construction traffic may require repair. Traffic should not be allowed on the new pavement until all lifts have been placed.

Based on our experience with similar projects and subgrade soil conditions, we recommend the following pavement sections provided in Table 4-2.

Table 4-2: RECOMMENDED PAVEMENT SECTIONS

Pavement Type	Traffic Loading	CRB Thickness, inches	Pavement Thickness, inches
AC	Areas Subject to All Traffic (Bus Loops and Access Roads)	14	5
AC	Areas Subject to Primarily Automobile Traffic (Parking Lot Drive Aisles, Occasional Truck Traffic)	12	4
AC	Areas Subject to Primarily Automobile Traffic (Parking Lot Drive Aisles, Occasional Truck Traffic with 12 inches of Cement-Amended Subgrade)	6	4
AC	Areas Subject to Automobile Parking (Parking Areas)	8	3
PCC	Areas Subject to Repeated Heavy Truck Traffic (Trash Enclosure and Service Areas)	6	6

Note: The recommended pavement sections should be considered minimum thicknesses and underlain by a nonwoven geotextile fabric.

PAVEMENT

It should be assumed that some maintenance will be required over the life of the pavement (15 years to 20 years). The recommended pavement sections are based on the assumption that pavement construction will be accomplished during the dry season and after construction of the building has been completed. If wet-weather pavement construction is considered, it will likely be necessary to increase the thickness of CRB course to support construction equipment and protect the subgrade from disturbance as discussed in the Earthwork section of this report. The indicated sections are not intended to support construction traffic such as forklifts, dump trucks, or concrete trucks.

For the above-indicated sections, drainage is an essential aspect of pavement performance. We recommend all paved areas be provided positive drainage to remove surface water and water within the base course; subgrade should be sloped to a minimum 0.5% slope to aid in drainage. This will be particularly important in cut sections or at low points within the paved areas, such as at catch basins. Effective methods to prevent saturation of the base-course materials include providing weepholes in the sidewalls of catch basins, subdrains in conjunction with utility excavations and separate trench-drain systems. To help ensure quality materials and construction practices, we recommend the pavement work conform to current Oregon Department of Transportation (ODOT) standards.

Prior to placing base-course materials, all pavement area subgrade should be proof rolled with a fully loaded dump truck. Any soft areas detected by the proof rolling should be overexcavated to firm ground and backfilled with compacted structural fill.

Provided the pavement section is installed in accordance with the above recommendations, it is our opinion the site-access areas will support infrequent traffic by an emergency vehicle having a gross vehicle weight of up to 80,000 pounds. For the purposes of this evaluation, “infrequent” can be defined as once a month or less.

4.10.2 *Standard Specifications*

Construction materials and procedures should comply with the applicable sections of the current ODOT *Oregon Standard Specifications for Construction* given in Table 4-3.

Table 4-3: ODOT SPECIFICATIONS FOR PAVEMENT CONSTRUCTION

Materials/Activity	Specification
Asphalt Concrete New Construction	Special Provision 00745. Place the AC section using a minimum lift thickness of 2 inches and maximum lift thickness of 3 inches. Lime or latex treatment of aggregate is not required.
Asphalt Binder	Use Performance Grade 64-22 Asphalt Cement in Level 2.
Aggregate Base	Section 00641 (¾ inch – 0 or 1 inch – 0).
Subgrade Geotextile	Sections 00350 and 02320. (Table 02320-4 Geotextile Property Values)

4.11 On-Site Disposal of Stormwater

Three field-infiltration rates were measured in test holes at depths ranging from about 4.7 feet to 10 feet below existing site grades to assist with the design of on-site stormwater disposal. The test method and results are summarized in the Infiltration Testing section of this report and in Appendix A. The unfactored, field-measured infiltration rates for the soils that mantle the site range from less than about 0.25 inches per hour up to about 0.25 inches per hour. The City of Portland 2020 SMM, Section 2.3.2 recommends for the encased falling-head test method, using a minimum factor of safety of 2 to establish the design infiltration rate. Based on the low field-infiltration rates, it is our opinion the near-surface silt soils do not meet the requirements for on-site stormwater disposal without an overflow connected to an approved discharge location.

5 LIMITATIONS

This report has been prepared to aid the owner, architect, and engineer in the planning and preparation of design and associated cost estimates. The scope is limited to the specific project and location described within this report and our description of the project represents our understanding of the significant aspects of the project relevant to the construction of foundations, retaining walls, and pavements. In the event that any changes in the design and location of the project elements as outlined in this report are planned, we should be given the opportunity to review the changes and modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations submitted in this report are based on the data obtained from the explorations made at the locations indicated on Figure 2 and other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between exploration locations. This report does not reflect any variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If during construction, subsurface conditions differ from those encountered in the explorations, we should be advised at once so we can observe and review these conditions and reconsider our recommendations where necessary.

Please contact the undersigned if you have any questions.

Submitted for GRI,

A. Wesley Spang, PhD, PE, GE	Brian Cook, PE
Principal	Project Engineer

This document has been submitted electronically.

6 REFERENCES

Agusti, G. C., and Sitar, N., 2013, Seismic Earth Pressures on Retaining Structures in Cohesive Soils, University of California, Berkley, UCB GT 13-02.

Idriss, I. M., and Boulanger, R. W., 2008, Soil liquefaction during earthquakes, Earthquake Engineering Research Institute, EERI MNO-12.

Idriss, I. M., and Boulanger, R. W., 2014, CPT and SPT based liquefaction triggering procedures, Department of Civil & Environmental Engineering, College of Engineering, University of California at Davis, Report No. UCD/CGM-14/01.

Ma, L., Wells, R. E., Niem, A. R., Niewendorp, C. A., and Madin, I. P., 2009, Preliminary digital geologic compilation map of part of northwestern Oregon, Oregon Department of Geology and Mineral Industries, Open-File Report 09-03.

Oregon Department of Transportation, 2021, Oregon standard specifications for construction.

Oregon Water Resources Department (OWRD), 2019, Well report query, mapping tool, accessed 9/29/2021 from OWRD website: https://apps.wrd.state.or.us/apps/gw/wl_well_report_map/.

Petersen, M. D., Moschetti, M. P., Powers, P. M., Mueller, C. S., Haller, K. M., Frankel, A. D., Zeng, Y., Rezaeian, S., Harmsen, S. C., Boyd, O. S., Field, N., Chen, R., Rukstales, K. S., Nico, L., Wheeler, R. L., Williams, R. A., and Olsen, A. H., 2014, Documentation for the 2014 update of the United States national seismic hazard maps, U.S. Geological Survey, Open-File Report 2014–1091, 243 pages, http://dx.doi.org/10.3133/ofr20141091.

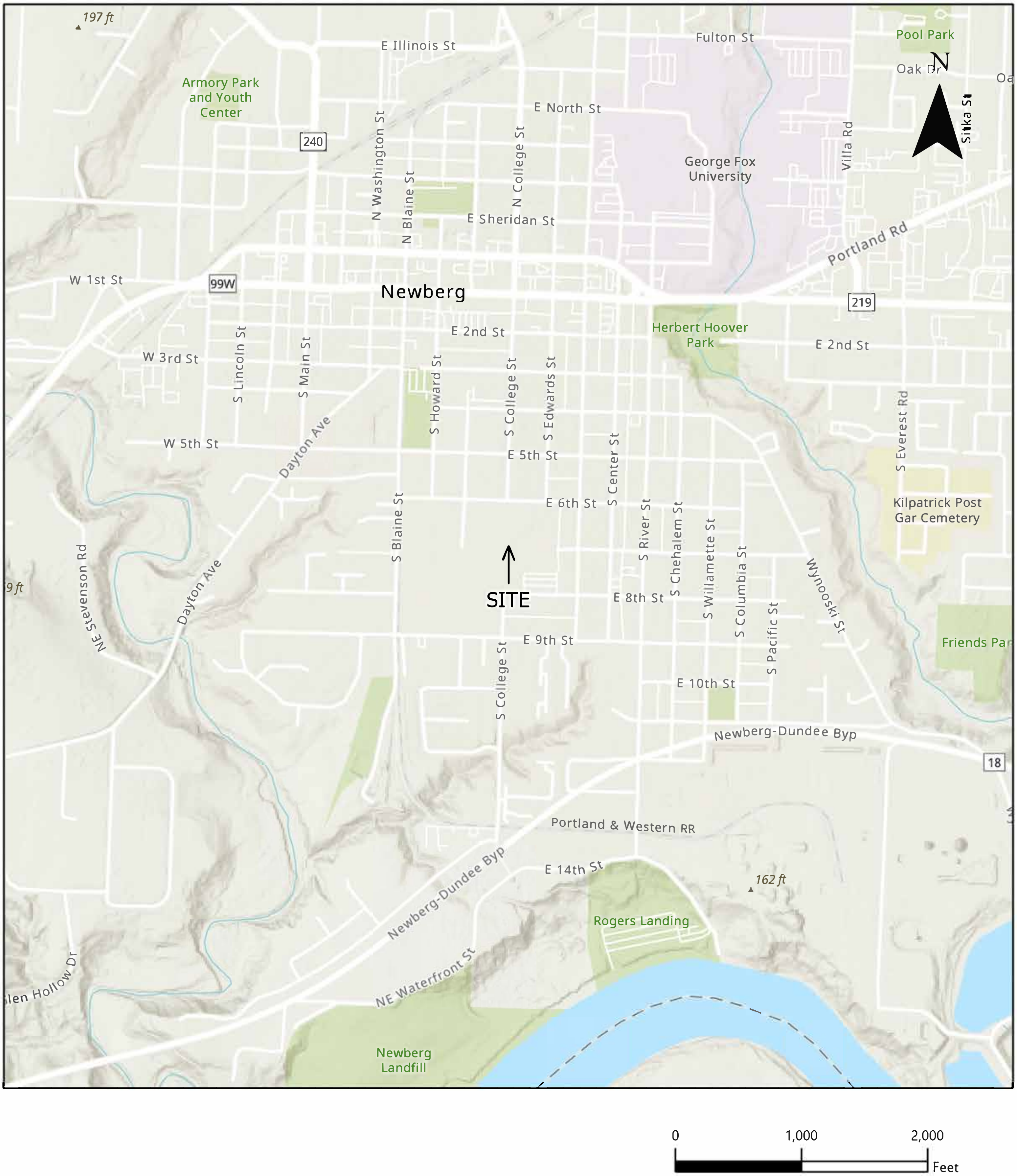
U.S. Geological Survey (USGS), 2020, Quaternary Faults Database, accessed 9/28/2021 from USGS website: https://usgs.maps.arcgis.com/apps/webappviewer.

USGS, ASCE 7-16 Seismic Design Map Web Service, accessed 9/28/2021 from USGS website: https://earthquake.usgs.gov/ws/designmaps/.

USGS, Unified hazard tool, Dynamic: conterminous U.S. 2014 (v4.1.1), accessed 9/28/2021 from USGS website: https://earthquake.usgs.gov/hazards/interactive/.

USGS, 2014 National seismic hazard maps – Source parameters, lookup by latitude, longitude, accessed 9/28/2021 from USGS website: https://earthquake.usgs.gov/cfusion/hazfaults_2014_search/.

Wells, R. E., Haugerud, R., Niem, A., Niem, W., Ma, L., Madin, I., and Evarts, R., 2018, New Geologic Mapping of the Northwestern Willamette Valley, Oregon, and its American Viticultural Areas (AVAs)—A Foundation for Understanding Their Terroir: U.S. Geological Survey Open-File Report 2018-1044, p. 1, doi:10.3133/ofr20181044.



Folder: P:\6501-6600\6519 Edwards ES\6519-A\GIS\

GRI

NEWBERG SCHOOL DISTRICT
EDWARDS ELEMENTARY SCHOOL
2022 ADDITIONS AND IMPROVEMENTS

VICINITY MAP

OCT. 2021 JOB NO. 6519-A FIG. 1

⊕ BORING COMPLETED BY GRI
(SEPTEMBER 1-3, 2021)

◒ BORING AND CONE PENETRATION TEST COMPLETED BY GRI
(SEPTEMBER 2-3, 2021)

⬒ HAND-AUGER BORING COMPLETED BY GRI
(SEPTEMBER 3, 2021)

▲ INFILTRATION TEST COMPLETED BY GRI
(SEPTEMBER 1-3, 2021)

● DYNAMIC CONE PENETRATION TEST COMPLETED BY GRI
(SEPTEMBER 1, 2021)

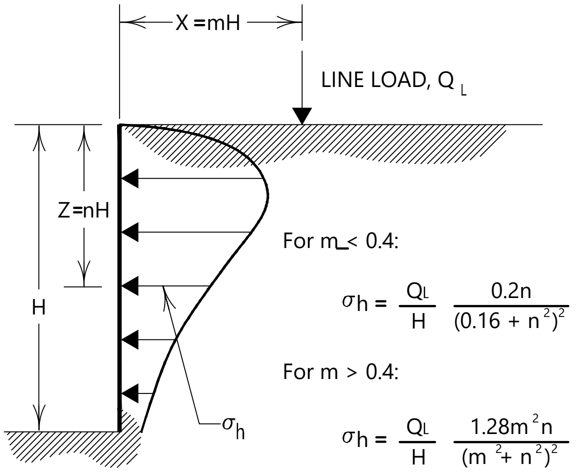
◓ PAVEMENT BORING AND DYNAMIC CONE PENETRATION TEST COMPLETED BY GRI
(SEPTEMBER 2-3, 2021)

0 120 240
Feet

GRI
NEWBERG SCHOOL DISTRICT
EDWARDS ELEMENTARY SCHOOL
2022 ADDITIONS AND IMPROVEMENTS

SITE PLAN

OCT. 2021 JOB NO. 6519-A FIG. 2



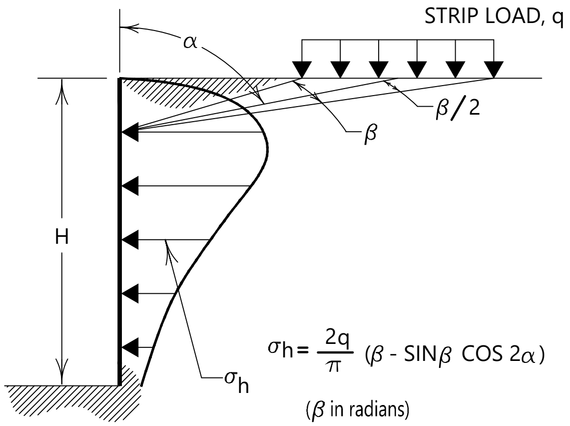
For $m \leq 0.4$:

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

For m > 0.4:

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

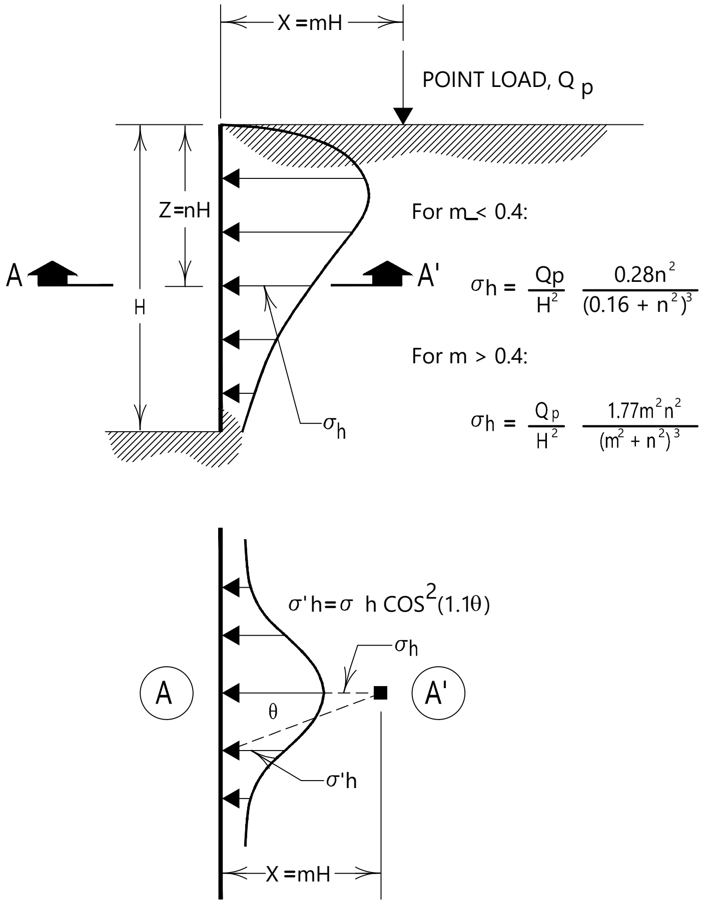
LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{\pi} (\beta - \text{SIN}\beta \ \text{COS} \ 2\alpha)$$

(β in radians)

STRIP LOAD PARALLEL TO WALL



For $m \leq 0.4$:

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

For m > 0.4:

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

DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD

NOTES:

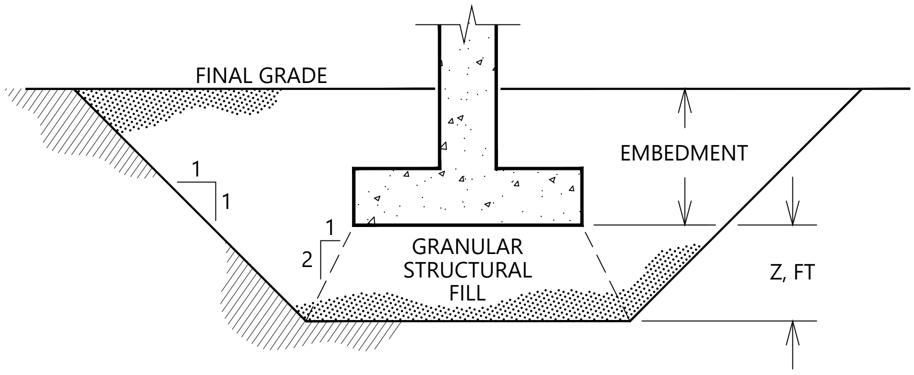
1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.
2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

GRI

NEWBERG SCHOOL DISTRICT
EDWARDS ELEMENTARY SCHOOL
2022 ADDITIONS AND IMPROVEMENTS

SURCHARGE-INDUCED LATERAL PRESSURE

OCT. 2021 JOB NO. 6519-A FIG. 3



LIMITS OF OVEREXCAVATION
NOT TO SCALE

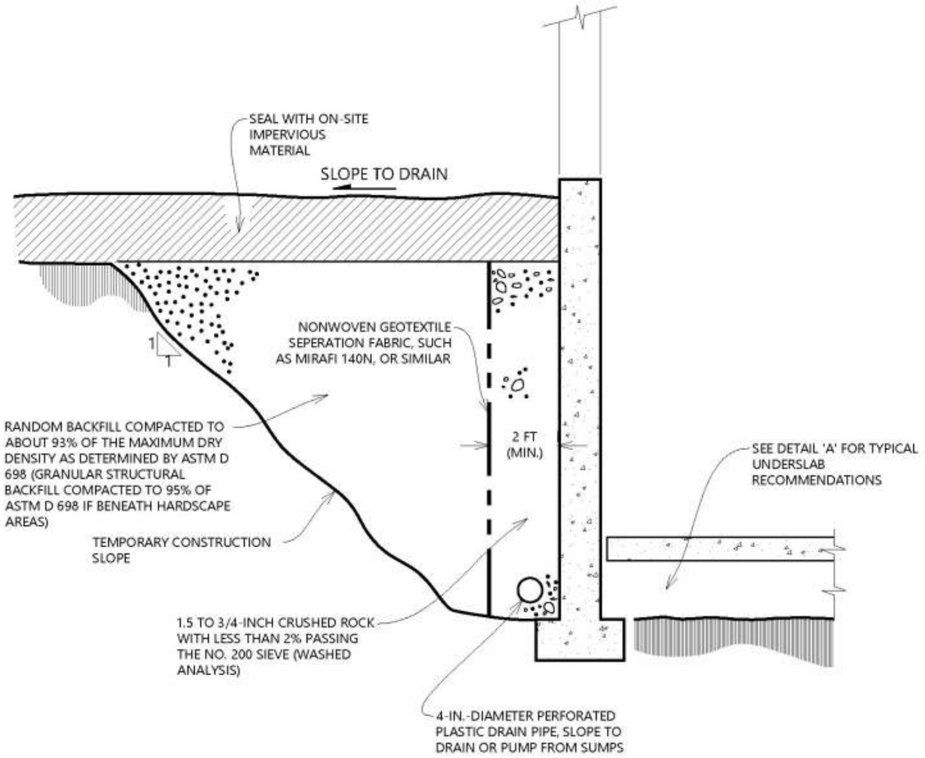
drawings should
reference this

GRI

NEWBERG SCHOOL DISTRICT
EDWARDS ELEMENTARY SCHOOL
2022 ADDITIONS AND IMPROVEMENTS

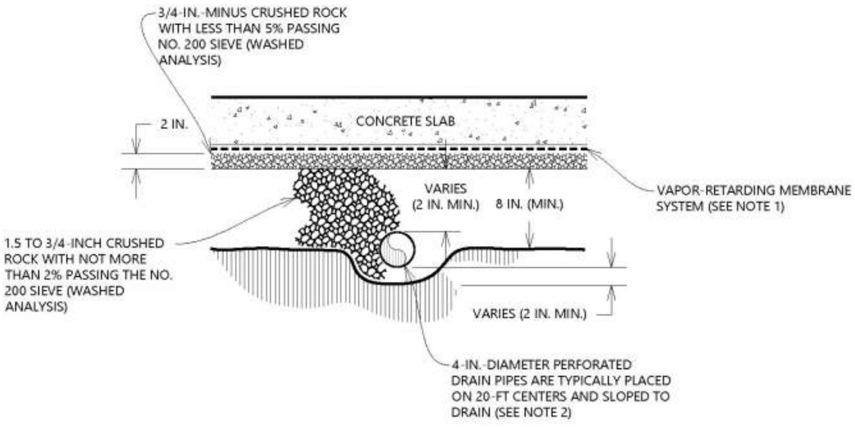
OVEREXCAVATION DETAIL

OCT. 2021 JOB NO. 6519-A FIG. 4



PERIMETER DRAIN

drawings should reference this



DETAIL 'A'
NOT TO SCALE

UNDERSLAB DRAIN

NOTES:

1) A VAPOR-RETARDING MEMBRANE SYSTEM IS RECOMMENDED FOR MOISTURE-SENSITIVE AREAS AND SHOULD BE INSTALLED IN ACCORDANCE WITH MANUFACTURER'S RECOMMENDATIONS.

2) INTERNAL 4-IN.-DIAMETER PERFORATED DRAIN PIPES ARE TYPICALLY NOT NECESSARY IN THOSE AREAS WHERE THE FINISH FLOOR WILL BE ABOVE EXISTING SITE GRADES.

GRI

NEWBERG SCHOOL DISTRICT
EDWARDS ELEMENTARY SCHOOL
2022 ADDITIONS AND IMPROVEMENTS

TYPICAL SUBDRAINAGE DETAIL

OCT. 2021 JOB NO. 6519-A FIG. 5

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

A.1 FIELD EXPLORATIONS

Subsurface materials and conditions at the site were investigated on September 1, through September 3, 2021, with four machine-drilled geotechnical borings, designated B-1 through B-4; three machine-drilled pavement borings, designated PB-1 through PB-3; three machine-drilled infiltration test borings, designated I-1 through I-3; two hand-augered borings, designated HA-1 and HA-2; one cone penetration test (CPT) probe, designated CPT-1; five Kessler dynamic cone penetration tests (KDCPs), designated KDCP-1 through KDCP-5; and falling-head infiltration testing in borings I-1 through I-3. The approximate locations of the explorations are shown on Figure 2. The terms and symbols used to describe the soil encountered in the explorations are defined in Tables 2A and 3A and on the attached legend. The field-exploration work was coordinated and documented by an experienced member of GRI's geotechnical engineering or geology team, who maintained a log of the materials and conditions disclosed during the course of work.

A.1.1 Drilled Borings

The machine-drilled borings designated B-1 through B-4 were completed using mud-rotary, open-hole drilling techniques, and the machine-drilled borings designated PB-1 through PB-3 and I-1 through I-3 were completed using hollow-stem auger, open-hole drilling techniques. The machine-drilled borings were advanced using a track-mounted Geoprobe drill rig provided and operated by Western States Soil Conservation, Inc., of Hubbard, Oregon. The machine-drilled borings were advanced to depths ranging from about 6.5 feet to 41.5 feet below existing site grades.

Disturbed and undisturbed soil samples were generally obtained from the machine-drilled borings at 2.5-foot intervals of depth in the upper 15 feet and at 5-foot intervals below this depth. Disturbed soil samples were obtained using a 2-inch-outside-diameter standard split-spoon sampler. Standard penetration tests (SPTs) were conducted by driving the samplers into the soil a distance of 18 inches using a 140-pound hammer dropped 30 inches. The number of blows required to drive the standard split-spoon sampler the last 12 inches is known as the Standard Penetration Resistance, or SPT N-value. The SPT N-values provide a measure of relative density of granular soils and the relative consistency of cohesive soils. Select soil samples obtained from the borings were placed in airtight jars and returned to our laboratory for further classification and testing. In addition, relatively undisturbed samples were collected by pushing a 3-inch-outside-diameter Shelby tube into the undisturbed soil a maximum distance of 24 inches using the hydraulic ram of the drill rig. The soil exposed in the end of the Shelby tube was examined and classified in the field. After classification, the tubes were sealed with rubber caps and returned to our laboratory for further examination and testing.

Logs of the geotechnical borings, B-1 through B-4, are provided on Figures 1A through 4A; logs of the pavement borings, PB-1 through PB-3, are provided on Figures 5A through 7A; and logs of the infiltration test borings, I-1 through I-3, are provided on Figures 8A through 11A. Each log presents a descriptive summary of the various types of materials encountered in the boring and notes the depth at which the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples are indicated. Farther to the right, SPT N-values are shown graphically, along with the natural moisture contents, Atterberg limits, and percentages passing the No. 200 sieve where applicable. The terms and symbols used to describe the materials encountered in the borings are defined in Table 2A and in the attached legend.

A.1.2 Hand-Augered Borings

Two hand-augered borings, designated HA-1 and HA-2, were advanced by GRI to a depth of approximately 5.3 feet below the ground surface.

Disturbed soil samples were generally obtained from the hand-augered borings at 2-foot intervals of depth or as subsurface conditions changed. Select soil samples were classified in the field. After classification, the samples were sealed in jars and returned to our laboratory for further examination and classification.

Logs of the hand-augered borings are provided on Figures 11A and 12A. Each log presents a descriptive summary of the various types of materials encountered and notes the depth at which the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples are indicated. Farther to the right, the natural moisture contents, Atterberg limits, and percentages passing the No. 200 sieve are shown where applicable. The terms and symbols used to describe the materials encountered in the excavations are defined in Table 2A and in the attached legend.

A.1.3 Cone Penetration Test

One CPT probe, designated CPT-1, was advanced to a depth of about 64 feet below existing site grade using a track-mounted Geoprobe rig provided and operated by Oregon Geotechnical Explorations, Inc., of Keizer, Oregon. During a CPT, a steel cone is forced vertically into the soil at a constant rate of penetration. The force required to cause penetration at a constant rate can be related to the bearing capacity of the soil immediately surrounding the point of the penetrometer cone. This force is measured and recorded every 2 inches. In addition to the cone measurements, measurements are obtained of the magnitude of force required to force a friction sleeve attached above the cone through the soil. The force required to move the friction sleeve can be related to the undrained shear strength of fine-grained soils. The dimensionless ratio of sleeve friction to point-bearing capacity provides an indicator of the type of soil penetrated. The cone penetration resistance and sleeve friction can be used to evaluate the relative consistency of cohesionless and cohesive soils, respectively. In addition, a piezometer fitted between the cone and the sleeve measures changes in water pressure as the probe is advanced and can also be used to estimate the groundwater depth. The probe is also operated using an accelerometer fitted to the probe, which allows measurement of the arrival time of shear waves from impulses generated at the ground surface and calculation of shear-wave velocities for the surrounding soil profile.

A log of the CPT probe is provided on Figure 13A, which presents a graphical summary of the tip resistance, local (sleeve) friction, friction ratio, pore pressure, and soil behavior type index. The terms used to describe the soils encountered in the probe are defined in Table 3A. Shear-wave velocity measurements were recorded for the CPT probe and are shown on Figure 14A.

A.1.4 KDCP Probes

Five KDCP probes, designated KDCP-1 through KDCP-5, were advanced to a depth about 3 feet below the ground surface using a Kessler DCP manufactured by KSE Testing Equipment. The DCP tests were completed in accordance with ASTM International (ASTM) D6951 by driving a ⅝-inch-diameter steel rod with a cone tip into the soil using a 17.6-pound sliding hammer dropped at a fixed height of 22.6 inches. The number of blows required to drive the probe approximately 5 centimeters (2 inches) was recorded to depths ranging from 910 millimeters to 941 millimeters (35.8 inches to 37 inches). The DCP blow counts were used to estimate a California bearing ratio (CBR) value for the in-situ subgrade. Logs of the KDCP test probes are provided on Figures 15A through 19A.

A.1.5 Infiltration testing

Falling-head infiltration testing was completed at the site on September 1 through 3, 2021, in general conformance with the City of Portland 2020 *Stormwater Management Manual* (SMM) using the encased falling-head method outlined in Section 2.3.2 of the manual. The test locations were designated I-1 through I-3 in shallow boreholes at depths of about 4.7 feet to 10 feet below existing site grades. The boreholes were drilled to the selected depths using a track-mounted Geoprobe drill rig and a 6-inch-inside-diameter hollow-stem auger. The auger was seated firmly into the base of the borehole and filled with water to a height of approximately 1 foot above the base of the hole. After soaking overnight, infiltration testing was conducted by reestablishing the water level in the auger to the target height and recording the drop in water level over one hour or until the water completely drained, whichever occurred first. Where necessary, the infiltration test was repeated until consecutive tests showed little or no change in infiltration rate. The average unfactored, field-measured infiltration rates are tabulated below.

Table 1A: INFILTRATION TEST RESULTS

Test No.	Depth of Infiltration Test, feet	Average Field Infiltration Rate, inches/hour	Soil Classification	Fines Content (% Passing No. 200 Sieve)
I-1	4.7	0.25	Clayey SILT, trace fine-grained sand	93
I-2	9.8	< 0.25	SILT, some clay, trace fine-grained sand	92
I-3	10	< 0.25	SILT, trace fine-grained sand and up to trace clay	91

After the infiltration testing was completed, disturbed samples of the material were collected and examined in the field, and selected portions were saved in airtight jars for further examination and physical testing in our laboratory. The City of Portland 2020 SMM, Section 2.3.2 recommends encased falling head test methods using a minimum factor of safety of 2 to establish the design infiltration rate.

A.2 LABORATORY TESTING

A.2.1 General

The samples obtained from the borings were examined in our laboratory, where the physical characteristics of the samples were noted, and the field classifications modified where necessary. At the time of classification, the natural moisture content of each sample was determined. Additional testing included one-dimensional consolidation, Atterberg-limits determination, and grain-size analysis. A summary of the laboratory test results has been provided in Table 4A. The following sections describe the testing program in more detail.

A.2.2 Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM D2216. The results are summarized on Figures 1A through 12A, where applicable, and in Table 4A.

A.2.3 Grain-Size Analysis

A.2.3.1 Washed-Sieve Method

To assist in the classification of the soils, samples of known dry weight were washed over a No. 200 sieve. The material retained on the sieve was oven-dried and weighed. The percentage of material passing the No. 200 sieve was then calculated. The results are summarized on Figures 1A through 12A, where applicable, and in Table 4A.

A.2.4 Atterberg Limits

Atterberg-limits determinations were performed on samples obtained from the borings in conformance with ASTM D4318. The results of the tests are shown graphically on Figures 1A through 12A, where applicable; the Plasticity Chart, Figure 20A; and in Table 4A.

A.2.5 Torvane Shear Strength

The approximate undrained shear strength of the fine-grained soils was determined using the Torvane shear device. The Torvane is a handheld apparatus with vanes that are inserted into the soil. The torque required to fail the soil in shear around the vanes is measured using a calibrated spring. The results of the Torvane shear-strength testing are shown on Figures 1A through 4A, where applicable.

A.2.6 Undisturbed Unit Weight

The unit weight, or density, of undisturbed soil samples, was determined in the laboratory in substantial conformance with ASTM D2937. The results are summarized on Figures 1A through 4A, where applicable, and in Table 4A.

A.2.7 One-Dimensional Consolidation

One-dimensional consolidation testing was performed in accordance with ASTM D2435 on relatively undisturbed soil samples obtained from boring B-1 at depths of about 14.5 feet and 26.5 feet, boring B-3 at a depth of about 8.5 feet, and boring B-4 at a depth of about 38.5 feet. The test provides data on the compressibility of underlying fine-grained soils. Test results are summarized on Figures 21A through 24A in the form of a curve showing effective stress versus percent strain. The initial dry unit weights and moisture contents of the samples are also shown on the figures.

Table 2A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values), blows/ft
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values), blows/ft	Torvane or Undrained Shear Strength, tsf
Very Soft	0 - 2	less than 0.125
Soft	2 - 4	0.125 - 0.25
Medium Stiff	4 - 8	0.25 - 0.50
Stiff	8 - 15	0.50 - 1.0
Very Stiff	15 - 30	1.0 - 2.0
Hard	over 30	over 2.0

Grain-Size Classification

Boulders:
- >12 in.

Cobbles:
- 3-12 in.

Gravel:
- ¼ - ¾ in. (fine)
- ¾ - 3 in. (coarse)

Sand:
- No. 200 - No. 40 sieve (fine)
- No. 40 - No. 10 sieve (medium)
- No. 10 - No. 4 sieve (coarse)

Silt/Clay:
- Pass No. 200 sieve

Modifier for Subclassification

Adjective	Primary Constituent SAND or GRAVEL	Primary Constituent SILT or CLAY
	Percentage of Other Material (By Weight)	
trace:	5 - 15 (sand, gravel)	5 - 15 (sand, gravel)
some:	15 - 30 (sand, gravel)	15 - 30 (sand, gravel)
sandy, gravelly:	30 - 50 (sand, gravel)	30 - 50 (sand, gravel)
trace:	<5 (silt, clay)	*Relationship of clay and silt determined by plasticity index test*
some:	5 - 12 (silt, clay)	
silty, clayey:	12 - 50 (silt, clay)	

Table 3A

CONE PENETRATION TEST (CPT) CORRELATIONS

Cohesive Soils

Cone Tip Resistance, tsf	Consistency
<5	Very Soft
5 to 15	Soft to Medium Stiff
15 to 30	Stiff
30 to 60	Very Stiff
>60	Hard

Cohesionless Soils

Cone Tip Resistance, tsf	Relative Density
<20	Very Loose
20 to 40	Loose
40 to 120	Medium
120 to 200	Dense
>200	Very Dense

Reference

Kulhawy, F. H., and Mayne, P. W., 1990, Manual on Estimating Soil Properties for Foundation Design, Electric Power Research Institute, EL-6800.

Table 4A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	**Sample**	**Depth, ft**	**Elevation, ft**	**Moisture Content, %**	**Dry Unit Weight, pcf**	**Liquid Limit, %**	**Plasticity Index, %**	**Fines Content, %**	**Soil Type**
B-1	S-1	2.5	--	32	--	--	--	--	Clayey SILT
	S-2	5.0	--	37	--	--	--	--	SILT
	S-3	7.5	--	40	--	--	--	88	SILT
	S-4	10.0	--	43	--	--	--	--	SILT
	S-5	13.5	--	38	86	--	--	--	SILT
	S-5	14.5	--	35	--	--	--	96	SILT
	S-6	15.0	--	38	--	--	--	78	Sandy SILT
	S-7	20.0	--	36	--	--	--	--	SILT
	S-8	25.5	--	36	86	--	--	--	SILT
	S-8	26.5	--	36	--	--	--	100	SILT
	S-9	27.0	--	31	--	--	--	--	SILT
	S-10	30.0	--	40	--	--	--	--	SILT
	S-11	35.0	--	33	--	--	--	84	Clayey SILT
	S-12	40.0	--	26	--	--	--	--	CLAY
B-2	S-1	2.5	--	25	--	--	--	--	Clayey SILT
	S-2	5.0	--	36	--	--	--	--	SILT
	S-3	7.5	--	40	--	--	--	94	SILT
	S-4	10.0	--	39	--	--	--	--	SILT
B-3	S-1	2.5	--	34	--	--	--	--	Clayey SILT
	S-2	5.0	--	37	--	--	--	--	SILT
	S-3	8.5	--	39	--	--	--	93	SILT
	S-3	9.0	--	40	82	--	--	--	SILT
	S-4	10.0	--	41	--	--	--	95	SILT
	S-5	12.5	--	38	--	--	--	--	SILT
	S-6	15.5	--	39	83	--	--	--	SILT
	S-7	17.0	--	36	--	--	--	--	Sandy SILT
	S-8	20.0	--	33	--	--	--	92	SILT
	S-9	25.0	--	32	--	--	--	--	SILT
	S-10	30.0	--	35	--	--	--	--	SILT
	S-11	33.5	--	35	--	--	--	98	SILT
	S-11	34.0	--	34	90	--	--	--	SILT
	S-12	35.0	--	27	--	--	--	--	Silty CLAY
B-4	S-1	2.5	--	35	--	--	--	92	Silty CLAY
	S-3	5.0	--	40	--	--	--	--	Silty CLAY
	S-4	7.5	--	38	--	--	--	84	SILT
	S-5	10.0	--	42	--	--	--	--	SILT
	S-6	14.0	--	40	81	--	--	--	SILT
	S-7	14.5	--	41	--	--	--	83	SILT
	S-8	20.0	--	36	--	--	--	--	SILT
	S-9	25.0	--	38	--	--	--	--	SILT

Table 4A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-4	S-10	29.5	--	30	94	--	--	--	SILT
	S-11	30.0	--	32	--	--	--	--	SILT
	S-12	35.0	--	36	--	--	--	92	SILT
	S-13	38.5	--	26	--	--	--	60	Sandy CLAY
	S-13	39.0	--	26	102	--	--	--	Sandy CLAY
	S-14	40.0	--	28	--	--	--	41	Clayey SAND
HA-1	S-1	2.0	--	12	--	--	--	--	SILT
	S-2	4.0	--	33	--	--	--	--	Clayey SILT
	S-3	5.0	--	28	--	--	--	--	Clayey SILT
HA-2	S-1	2.0	--	6	--	--	--	--	Silty GRAVEL
	S-2	3.0	--	26	--	--	--	--	Silty CLAY
	S-3	4.0	--	29	--	--	--	--	Silty CLAY
	S-4	5.0	--	38	--	--	--	92	SILT
I-1	S-1	2.5	--	23	--	--	--	93	Clayey SILT
	S-2	5.0	--	37	--	--	--	93	Clayey SILT
I-2	S-1	2.5	--	27	--	--	--	--	SILT
	S-2	5.0	--	32	--	--	--	--	SILT
	S-3	7.5	--	40	--	--	--	--	SILT
	S-4	10.0	--	40	--	--	--	92	SILT
I-3	S-1	2.5	--	36	--	--	--	--	Silty CLAY
	S-2	5.0	--	37	--	--	--	88	SILT
	S-3	7.5	--	41	--	--	--	--	SILT
	S-4	10.0	--	41	--	--	--	91	SILT
PB-1	S-1	2.5	--	19	--	--	--	--	SILT
	S-2	5.0	--	40	--	--	--	91	SILT
	S-3	7.5	--	39	--	--	--	90	SILT
	S-4	10.0	--	41	--	--	--	--	SILT
	S-5	12.5	--	41	--	--	--	90	Sandy SILT
PB-2	S-1	2.5	--	41	--	--	--	97	Clayey SILT
	S-2	5.0	--	35	--	--	--	--	SILT
	S-3	7.5	--	40	--	--	--	--	SILT
	S-4	10.0	--	41	--	42	17	--	Silty CLAY
	S-5	12.5	--	38	--	--	--	94	SILT
PB-3	S-1	2.5	--	35	--	--	--	--	Clayey SILT
	S-2	5.0	--	37	--	--	--	--	SILT
	S-3	7.5	--	40	--	--	--	--	SILT
	S-4	10.0	--	41	--	--	--	93	SILT
	S-5	12.5	--	39	--	--	--	--	SILT
	S-6	15.0	--	37	--	--	--	92	SILT

BORING AND TEST PIT LOG LEGEND

SOIL SYMBOLS

Symbol	Typical Description
⁘	LANDSCAPE MATERIALS
▩	FILL
○	GRAVEL; clean to some silt, clay, and sand
○	Sandy GRAVEL; clean to some silt and clay
○	Silty GRAVEL; up to some clay and sand
○	Clayey GRAVEL; up to some silt and sand
░	SAND; clean to some silt, clay, and gravel
░	Gravelly SAND; clean to some silt and clay
░	Silty SAND; up to some clay and gravel
░	Clayey SAND; up to some silt and gravel
▥	SILT; up to some clay, sand, and gravel
▥	Gravelly SILT; up to some clay and sand
▥	Sandy SILT; up to some clay and gravel
▥	Clayey SILT; up to some sand and gravel
▨	CLAY; up to some silt, sand, and gravel
▨	Gravelly CLAY; up to some silt and sand
▨	Sandy CLAY; up to some silt and gravel
▨	Silty CLAY; up to some sand and gravel
≋	PEAT

BEDROCK SYMBOLS

Symbol	Typical Description
✚	BASALT
▤	MUDSTONE
▤	SILTSTONE
▤	SANDSTONE

SURFACE MATERIAL SYMBOLS

Symbol	Typical Description
■	Asphalt concrete PAVEMENT
▩	Portland cement concrete PAVEMENT
○	Crushed rock BASE COURSE

SAMPLER SYMBOLS

Symbol	Sampler Description
▮	2.0 in. O.D. split-spoon sampler and Standard Penetration Test with recovery (ASTM D1586)
▮	Shelby tube sampler with recovery (ASTM D1587)
▮	3.0 in. O.D. split-spoon sampler with recovery (ASTM D3550)
☒	Grab Sample
▯	Rock core sample interval
▩	Sonic core sample interval
▒	Push probe sample interval

INSTALLATION SYMBOLS

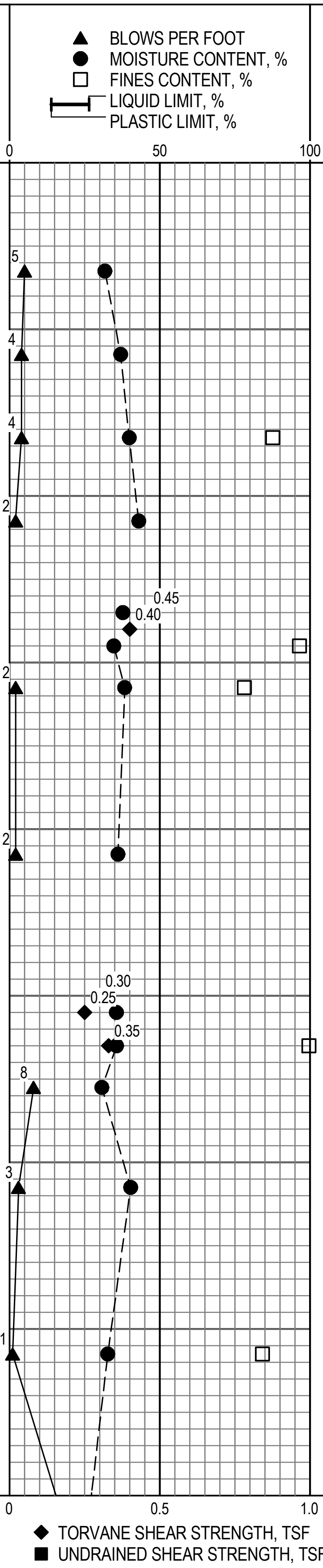
Symbol	Symbol Description
◨	Flush-mount monument set in concrete
◨	Concrete, well casing shown where applicable
▨	Bentonite seal, well casing shown if applicable
⁘	Filter pack, machine-slotted well casing shown where applicable
▒	Grout, vibrating-wire transducer cable shown where applicable
P	Vibrating-wire pressure transducer
▯	1-in.-diameter solid PVC
⋮	1-in.-diameter hand-slotted PVC
▒	Grout, inclinometer casing shown where applicable

FIELD MEASUREMENTS

Symbol	Typical Description
▽	Groundwater level during drilling and date measured
▼	Groundwater level after drilling and date measured
▨	Rock/sonic core or push probe recovery (%)
▨	Rock quality designation (RQD, %)

Surface Elevation: Not Available

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	Clayey SILT, trace fine-grained sand, brown to dark brown, soft to medium stiff, 5-in.-thick heavily rooted zone at ground surface (Willamette Silt)					
				S-1	3 2 3	
5	---trace to some clay, brown mottled rust below 5 feet			S-2	1 2 2	
				S-3	1 2 2	
10				S-4	0 1 1	
				S-5		Dry Density = 86 pcf
15	---sandy, brown to gray, fine-grained sand below 15 feet			S-6	1 1 1	
20	---up to trace fine-grained sand, dark gray below 20 feet			S-7	1 1 1	
25				S-8		Dry Density = 86 pcf
	---medium stiff to stiff below 27 feet			S-9	2 4 4	
30	---very soft to soft below 30 feet			S-10	3 1 2	
35	---clayey, some fine-grained sand below 35 feet			S-11	0 0 1	
40						



(CONTINUED NEXT PAGE)

GRI BORING LOG (LAT/LONG) GRI DATA TEMPLATE.GDT 10/5/21

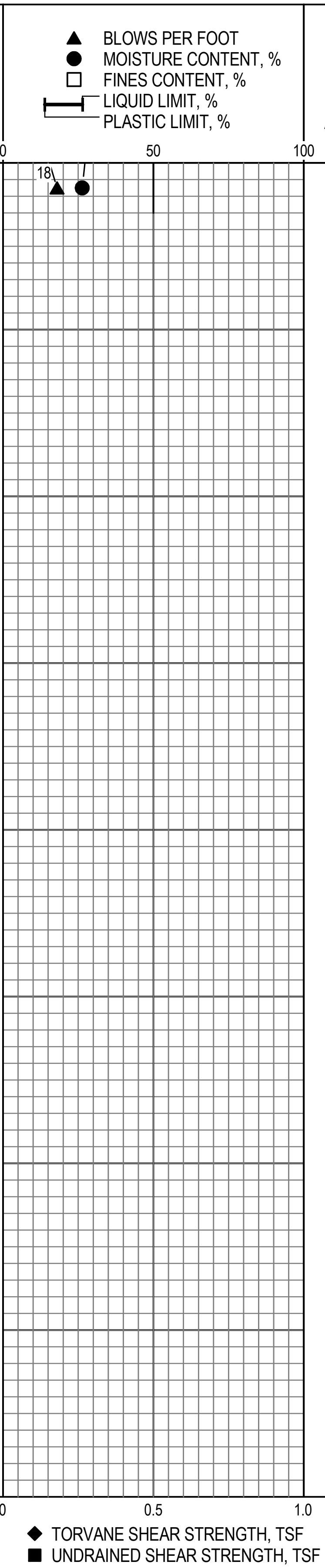
Logged By: A. Horst
Drilled by: Western States Soil Conservation, Inc.
Date Started: 9/1/21
Coordinates: 45.2954° N 122.9728° W (WGS 84)
Drilling Method: Mud Rotary
Equipment: 7720-DT Track-Mounted Geoprobe Rig
Hole Diameter: 4 in.
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio: 0.86
Note: See Legend for Explanation of Symbols

GRI

BORING B-1

OCT. 2021 JOB NO. 6519-A FIG. 1A

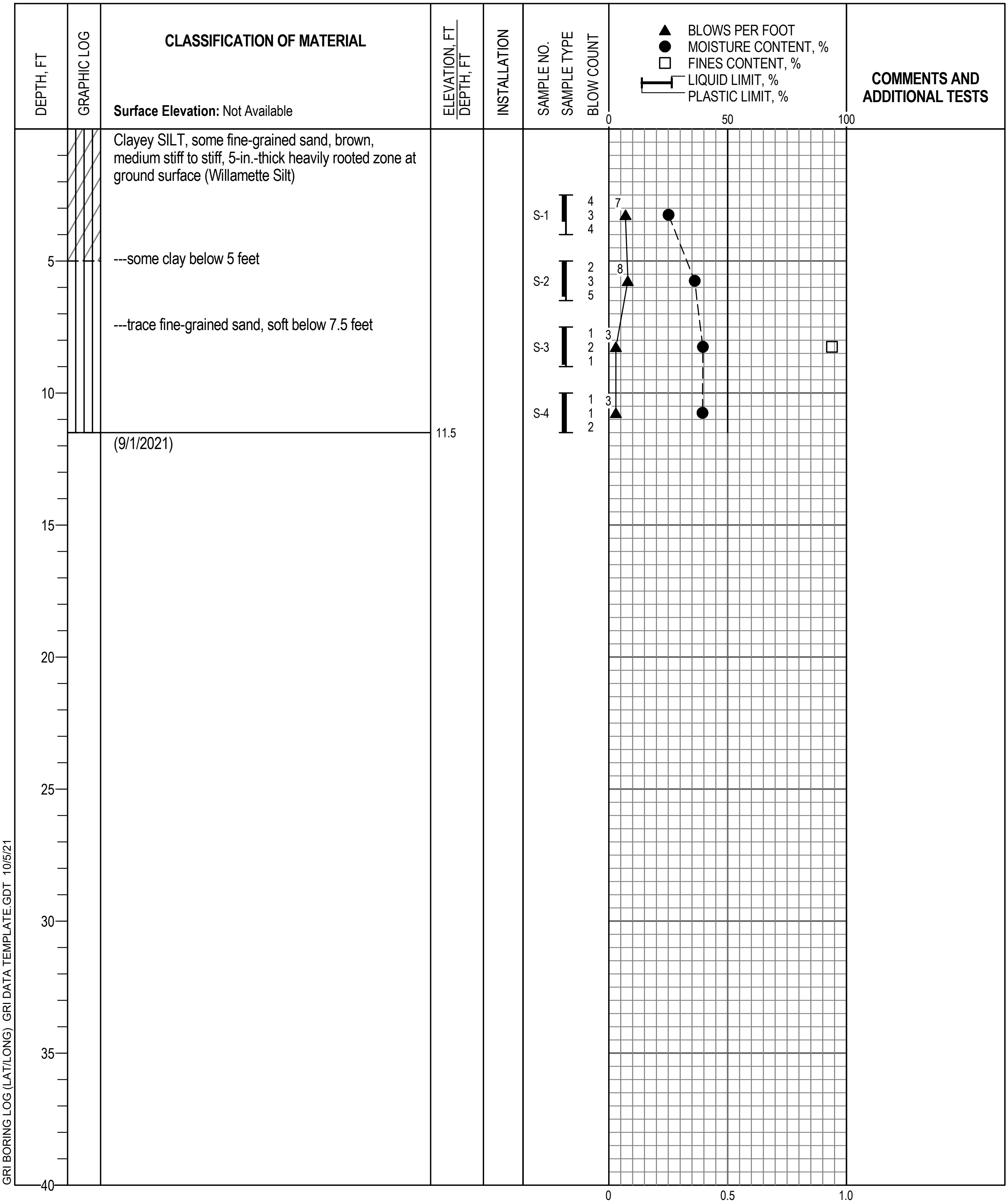
DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** Not Available	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	▨	CLAY, some silt and fine-grained sand, gray to dark gray, very stiff (Hillsboro Formation)			S-12	I	4 7 11	
		(9/1/2021)	41.5					
45								
50								
55								
60								
65								
70								
75								
80								



GRI

BORING B-1

OCT. 2021 JOB NO. 6519-A FIG. 1A



Surface Elevation: Not Available

Clayey SILT, some fine-grained sand, brown, medium stiff to stiff, 5-in.-thick heavily rooted zone at ground surface (Willamette Silt)

---some clay below 5 feet

---trace fine-grained sand, soft below 7.5 feet

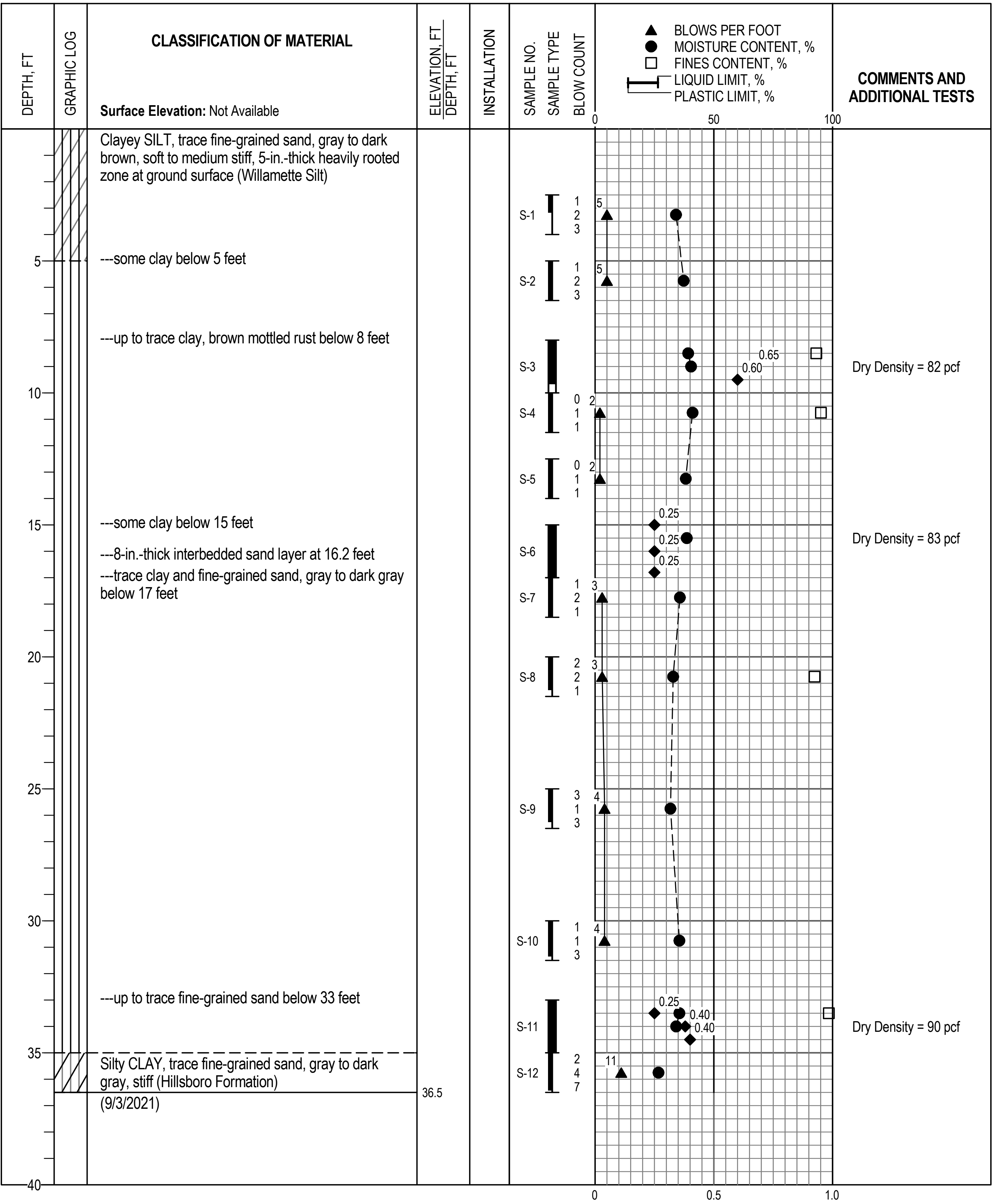
(9/1/2021)

◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

Logged By: A. Horst
Drilled by: Western States Soil Conservation, Inc.
Date Started: 9/1/21
Coordinates: 45.2952° N 122.9728° W (WGS 84)
Drilling Method: Mud Rotary
Equipment: 7720-DT Track-Mounted Geoprobe Rig
Hole Diameter: 4 in.
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio: 0.86
Note: See Legend for Explanation of Symbols

GRI

BORING B-2



DEPTH, FT	CLASSIFICATION OF MATERIAL Surface Elevation: Not Available	ELEVATION, FT / DEPTH, FT
0	Clayey SILT, trace fine-grained sand, gray to dark brown, soft to medium stiff, 5-in.-thick heavily rooted zone at ground surface (Willamette Silt)	
5	---some clay below 5 feet	
8	---up to trace clay, brown mottled rust below 8 feet	
15	---some clay below 15 feet	
16.2	---8-in.-thick interbedded sand layer at 16.2 feet	
17	---trace clay and fine-grained sand, gray to dark gray below 17 feet	
33	---up to trace fine-grained sand below 33 feet	
35	Silty CLAY, trace fine-grained sand, gray to dark gray, stiff (Hillsboro Formation)	
36.5	(9/3/2021)	36.5

SAMPLE NO.	BLOW COUNT	BLOWS PER FOOT	TORVANE SHEAR STRENGTH, TSF	COMMENTS AND ADDITIONAL TESTS
S-1	1 2 3	5		
S-2	1 2 3	5		
S-3			0.65 0.60	Dry Density = 82 pcf
S-4	0 1 1	2		
S-5	0 1 1	2		
S-6			0.25 0.25 0.25	Dry Density = 83 pcf
S-7	1 2 1	3		
S-8	2 2 1	3		
S-9	3 1 3	4		
S-10	1 1 3	4		
S-11			0.25 0.40 0.40	Dry Density = 90 pcf
S-12	2 4 7	11		

GRI BORING LOG (LAT/LONG) GRI DATA TEMPLATE.GDT 10/5/21

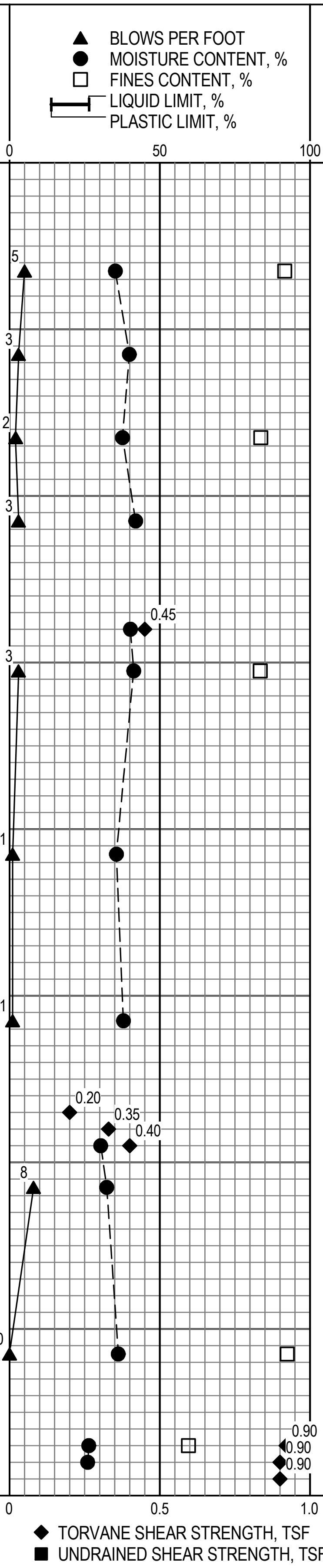
Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 9/3/21	**Coordinates:** 45.2943° N 122.973° W (WGS 84)
Drilling Method: Mud Rotary **Equipment:** 7720-DT Track-Mounted Geoprobe Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.86
Note: See Legend for Explanation of Symbols	

GRI

BORING B-3

OCT. 2021 JOB NO. 6519-A FIG. 3A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: Not Available						
		Silty CLAY, trace fine-grained sand, brown mottled rust, medium stiff, 5-in.-thick heavily rooted zone at ground surface (Willamette Silt)			S-1		1 2 3	
5					S-3		0 2 1	
		SILT, some clay and fine-grained sand dark brown mottled rust, soft to medium stiff (Willamette Silt)	7.5		S-4		1 1 1	
10					S-5		0 1 2	
		---6-in.-thick interbedded sand layer at 13.5 feet			S-6			Dry Density = 81 pcf
15		---some fine- to coarse-grained sand, brown mottled gray, very soft to soft below 14 feet			S-7		0 1 2	
20		---trace clay and fine-grained sand, gray to dark gray, very soft below 20 feet			S-8		0 0 1	
25					S-9		0 0 1	
		---soft to medium stiff below 28 feet			S-10			Dry Density = 94 pcf
30		---some clay, stiff to very stiff below 30 feet			S-11		1 3 5	
35		---very soft below 35 feet			S-12		0 0 0	
		Sandy CLAY, some silt and fine-grained sand, gray, stiff (Hillsboro Formation)			S-13			Dry Density = 102 pcf
40								



(CONTINUED NEXT PAGE)

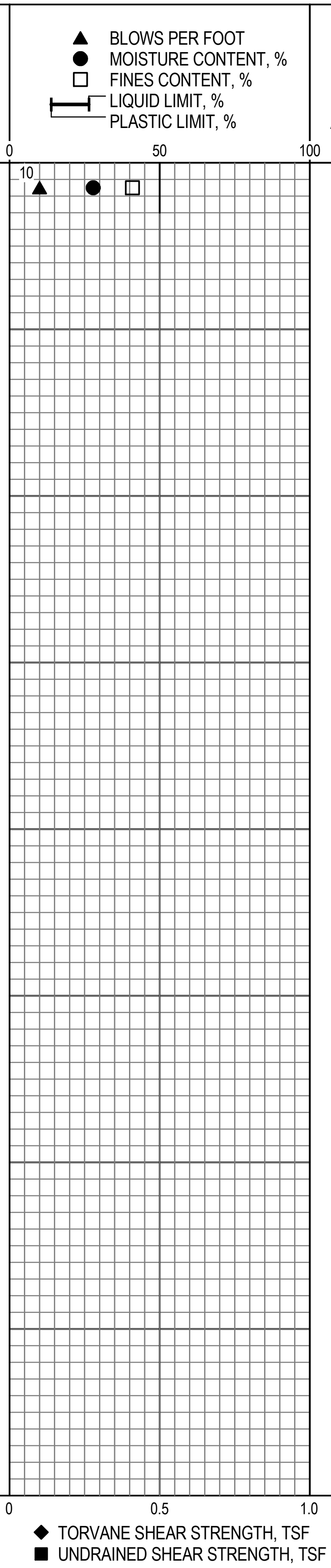
Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 9/2/21	**Coordinates:** 45.2945° N 122.9725° W (WGS 84)
Drilling Method: Mud Rotary **Equipment:** 7720-DT Track-Mounted Geoprobe Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.86
Note: See Legend for Explanation of Symbols	

GRI

BORING B-4

OCT. 2021 JOB NO. 6519-A FIG. 4A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** Not Available	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Clayey SAND, trace silt, gray mottled yellow-brown, fine grained sand, very stiff (Hillsboro Formation)	40.0 41.5		S-14		3 4 6	
		(9/3/2021)						
45								
50								
55								
60								
65								
70								
75								
80								



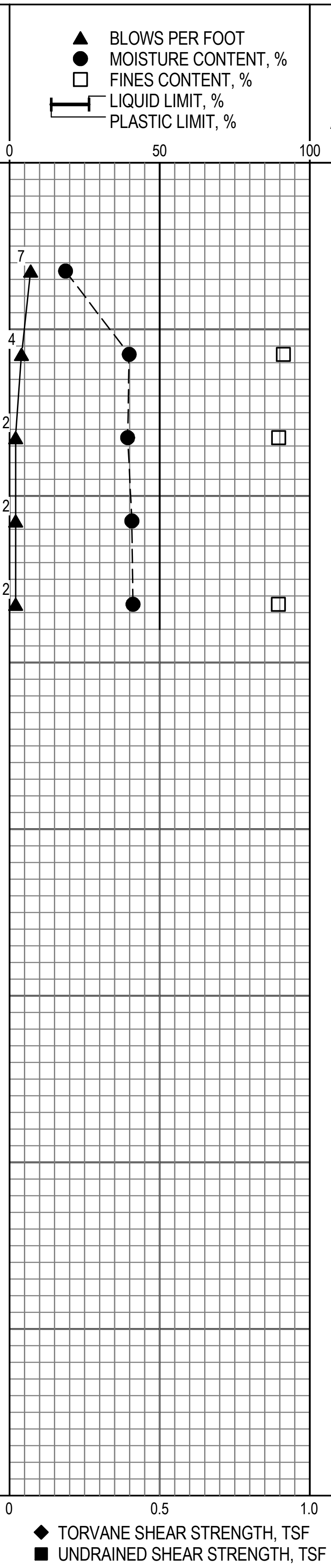
GRI BORING LOG (LAT/LONG) GRI DATA TEMPLATE.GDT 10/5/21

GRI

BORING B-4

OCT. 2021 JOB NO. 6519-A FIG. 4A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL Surface Elevation: Not Available	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		SILT, some clay, trace fine-grained sand, gray to dark brown, soft to medium stiff, contains fine roots, 5-in.thick heavily rooted zone at ground surface (Willamette Silt)			S-1	I	3 3 4	
5		---brown to gray below 5 feet			S-2	I	1 2 2	
		---trace fine- to medium-grained sand below 7.5 feet			S-3	I	1 1 1	
10		---trace clay, brown below 10 feet			S-4	I	0 1 1	
		---sandy, dark brown, soft, fine- to medium-grained below 12.5 feet			S-5	I	0 1 1	
		(9/3/2021) Groundwater not encountered	14.0					
15								
20								
25								
30								
35								
40								



Logged By: A. Horst
Drilled by: Western States Soil Conservation, Inc.
Date Started: 9/3/21
Coordinates: 45.2957° N 122.9727° W (WGS 84)
Drilling Method: Hollow-Stem Auger
Equipment: 7720-DT Track-Mounted Geoprobe Rig
Hole Diameter: 8 in.
Note: See Legend for Explanation of Symbols
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio: 0.86

GRI

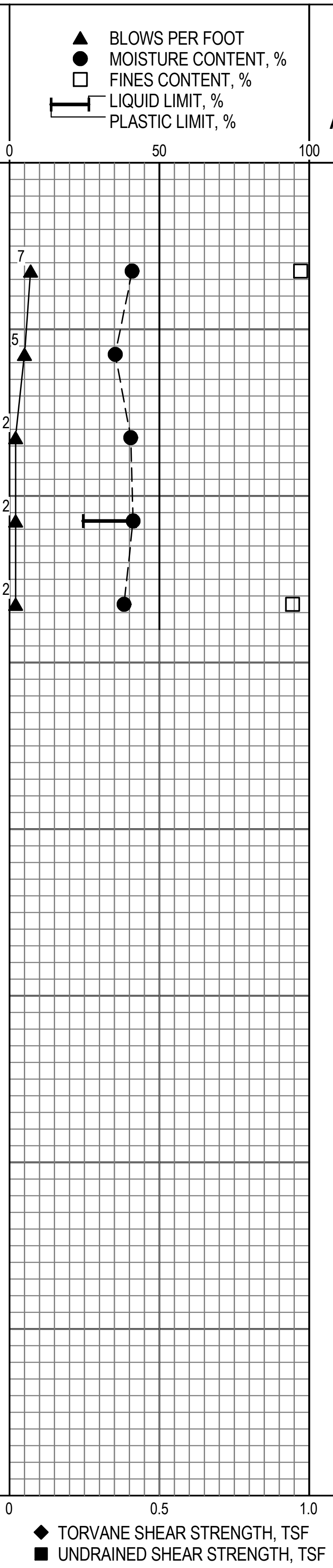
BORING PB-1

OCT. 2021 JOB NO. 6519-A FIG. 5A

Surface Elevation: Not Available

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT
0	Clayey SILT, up to trace fine-grained sand, gray, soft to medium stiff, 5-in.thick heavily rooted zone at ground surface (Willamette Silt)	
5	---some clay, gray to dark brown below 5 feet	
	---brown below 7.5 feet	
10	Silty CLAY, up to trace fine-grained sand, brown, soft (Willamette Silt)	10.0
	SILT, some clay, trace fine-grained sand, dark brown, soft (Willamette Silt)	12.5
	(9/3/2021) Groundwater not encountered	14.0

SAMPLE NO.	BLOW COUNT	BLOWS PER FOOT
S-1	2 4 3	7
S-2	2 2 3	5
S-3	1 1 1	2
S-4	0 1 1	2
S-5	0 1 1	2



COMMENTS AND ADDITIONAL TESTS

Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 9/3/21	**Coordinates:** 45.2957° N 122.9722° W (WGS 84)
Drilling Method: Hollow-Stem Auger **Equipment:** 7720-DT Track-Mounted Geoprobe Rig **Hole Diameter:** 8 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.86
Note: See Legend for Explanation of Symbols	

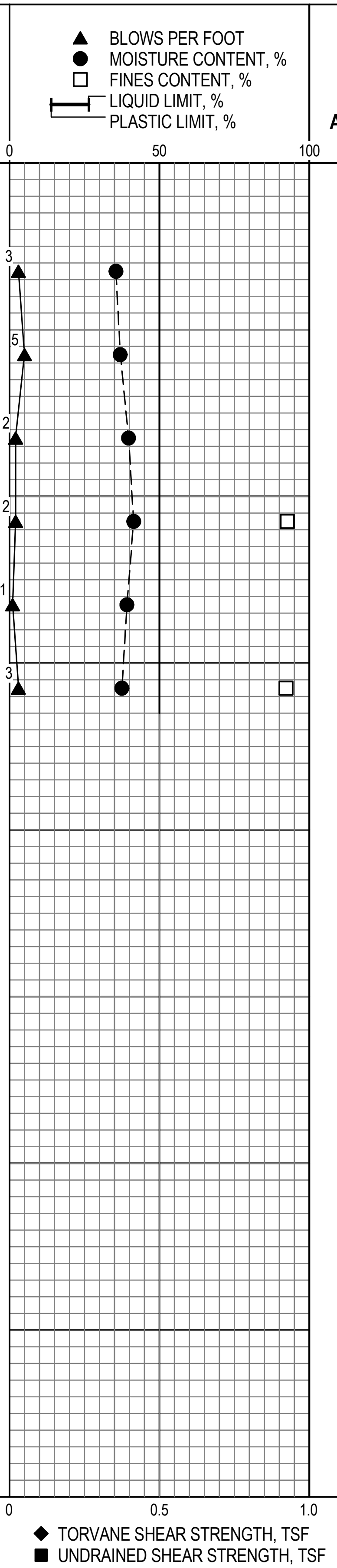
GRI

BORING PB-2

OCT. 2021 JOB NO. 6519-A FIG. 6A

Surface Elevation: Not Available

DEPTH, FT	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
0	Clayey SILT, trace fine-grained sand, brown-gray, soft to medium stiff, 4-in.-thick heavily rooted zone at ground surface (Willamette Silt)						
				S-1	I	0 1 2	
5	---some clay, brown below 5 feet			S-2	I	1 2 3	
				S-3	I	0 1 1	
10	---clayey below 10 feet			S-4	I	0 1 1	
	---trace fine-grained sand below 12.5 feet			S-5	I	0 1 0	
15	---trace fine- to medium-grained sand below 15 feet			S-6	I	0 1 2	
	(9/2/2021) Groundwater not encountered	16.5					
20							
25							
30							
35							
40							



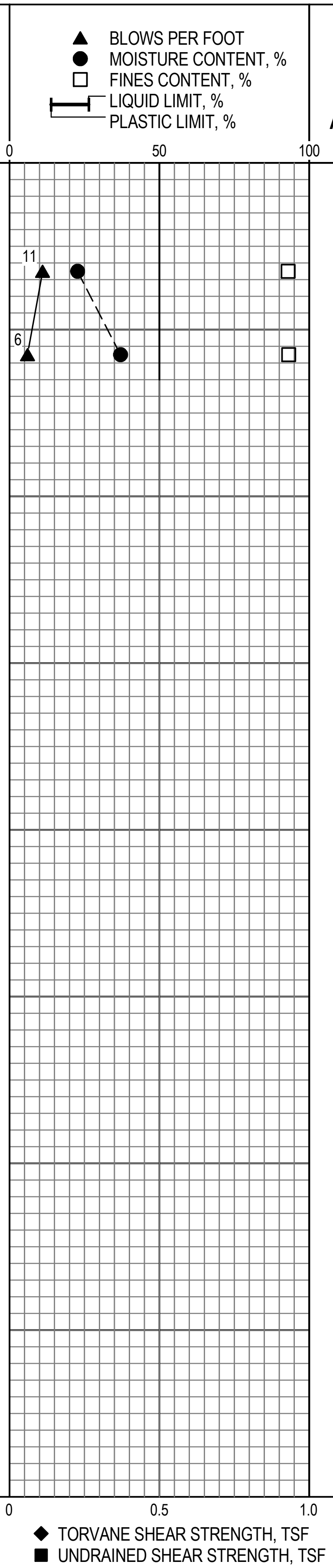
Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 9/2/21	**Coordinates:** 45.2941° N 122.9724° W (WGS 84)
Drilling Method: Hollow-Stem Auger **Equipment:** 7720-DT Track-Mounted Geoprobe Rig **Hole Diameter:** 8 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.86
Note: See Legend for Explanation of Symbols	

GRI

BORING PB-3

OCT. 2021 JOB NO. 6519-A FIG. 7A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL Surface Elevation: Not Available	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
0–6.5	▨	Clayey SILT, trace fine-grained sand, gray to dark brown, stiff, 5-in.-thick heavily rooted zone at ground surface (Willamette Silt)			S-1	I	3 5 6	
5		---gray to light brown, medium stiff below 5 feet	6.5		S-2	I	2 2 4	Falling head infiltration testing performed at about 5 ft Sample S-2 collected after infiltration testing
		(9/3/2021) Groundwater not encountered						



Depth scale: 5, 10, 15, 20, 25, 30, 35, 40

Logged By: A. Horst | **Drilled by:** Western States Soil Conservation, Inc.

Date Started: 9/1/21 | **Coordinates:** 45.2954° N 122.9731° W (WGS 84)

Drilling Method: Hollow-Stem Auger
Equipment: 7720-DT Track-Mounted Geoprobe Rig
Hole Diameter: 10 in.

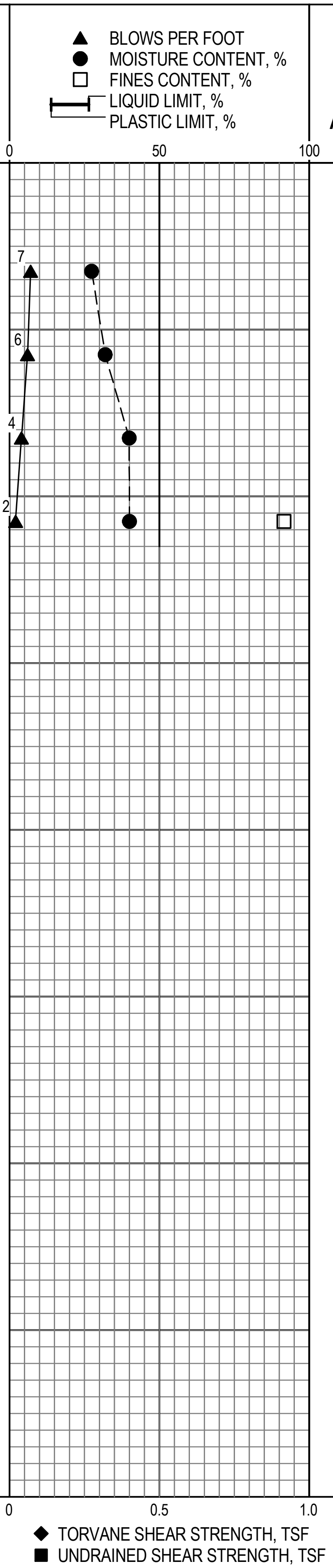
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio: 0.86

Note: See Legend for Explanation of Symbols

GRI

BORING I-1

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** Not Available	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
0		SILT, some clay, trace fine-grained sand, gray to dark brown, medium stiff, 5-in.-thick heavily rooted zone at ground surface (Willamette Silt)						
3					S-1	I	3 3 4	
5.5					S-2	I	2 3 3	
7.5		---trace clay, some fine-grained sand, soft to medium stiff below 7.5 feet						
8					S-3	I	1 2 2	
10		---some clay, trace fine-grained sand below 10 feet						Falling head infiltration testing performed at about 10 ft Sample S-4 collected after infiltration testing
10.5					S-4	I	0 1 1	
11.5		(9/3/2021) Groundwater not encountered	11.5					



GRI BORING LOG (LAT/LONG) GRI DATA TEMPLATE.GDT 10/5/21

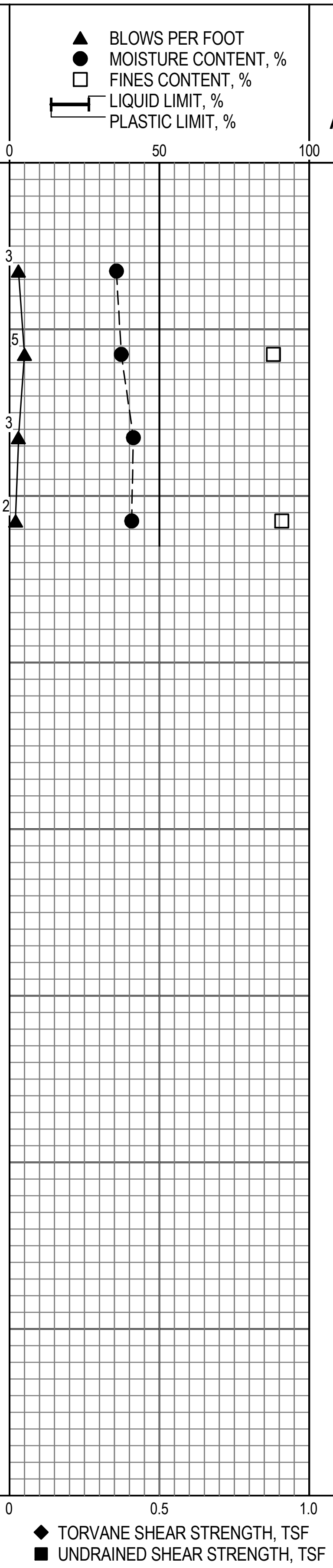
Logged By: A. Horst	**Drilled by:** Western States Soil Conservation, Inc.	
Date Started: 9/1/21	**Coordinates:** 45.2944° N 122.9733° W (WGS 84)	
Drilling Method: Hollow-Stem Auger **Equipment:** 7720-DT Track-Mounted Geoprobe Rig **Hole Diameter:** 10 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:** 0.86	
Note: See Legend for Explanation of Symbols		

GRI

BORING I-2

OCT. 2021 JOB NO. 6519-A FIG. 9A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** Not Available	ELEVATION, FT / DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Silty CLAY, trace fine-grained sand, dark brown, soft, 5-in.-thick heavily rooted zone at ground surface (Willamette Silt)			S-1	I	0 1 2	
5		SILT, some clay, trace fine-graned sand, brown, medium stiff (Willamette Silt) ---soft below 7.5 ft	5.0		S-2	I	1 2 3	
					S-3	I	0 1 2	
10		---trace clay, very soft to soft below 10 feet			S-4	I	0 1 1	Falling head infiltration testing performed at about 10 ft Sample S-4 collected after infiltration testing
		(9/3/2021) Groundwater not encountered	11.5					
15								
20								
25								
30								
35								
40								

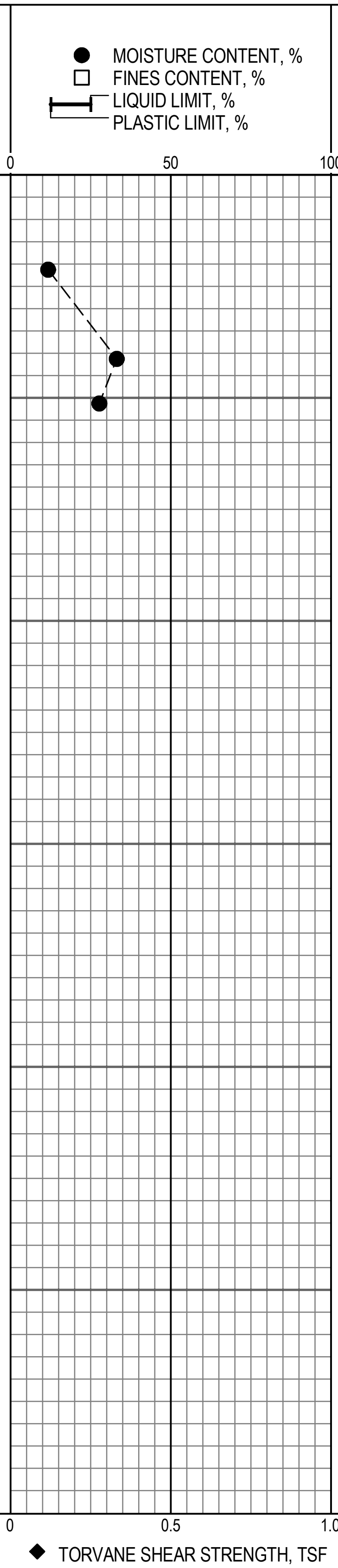


Logged By: A. Horst
Drilled by: Western States Soil Conservation, Inc.
Date Started: 9/1/21
Coordinates: 45.2943° N 122.9724° W (WGS 84)
Drilling Method: Hollow-Stem Auger
Equipment: 7720-DT Track-Mounted Geoprobe Rig
Hole Diameter: 10 in.
Hammer Type: Auto Hammer
Weight: 140 lb
Drop: 30 in.
Energy Ratio: 0.86
Note: See Legend for Explanation of Symbols

GRI

BORING I-3

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	ELEVATION, FT / DEPTH, FT	SAMPLE NO.	SAMPLE TYPE	COMMENTS AND ADDITIONAL TESTS
		Surface Elevation: Not Available				
		SILT, some fine-grained sand, trace clay, dark brown, contains fine roots, 4-in.-thick heavily rooted zone at ground surface (Willamette Silt)				
				S-1	☒	
		---clayey, trace fine-grained sand, roots absent below 4 feet		S-2	☒	
5				S-3	☒	
		(9/3/2021) Groundwater not encountered	5.3			
10						
15						
20						
25						
30						



Logged By: A. Horst

Excavated by: GRI

Equipment: Hand Auger

Date Started: 9/3/21

GPS Coordinates: 45.2954° N 122.9731° W (WGS 84)

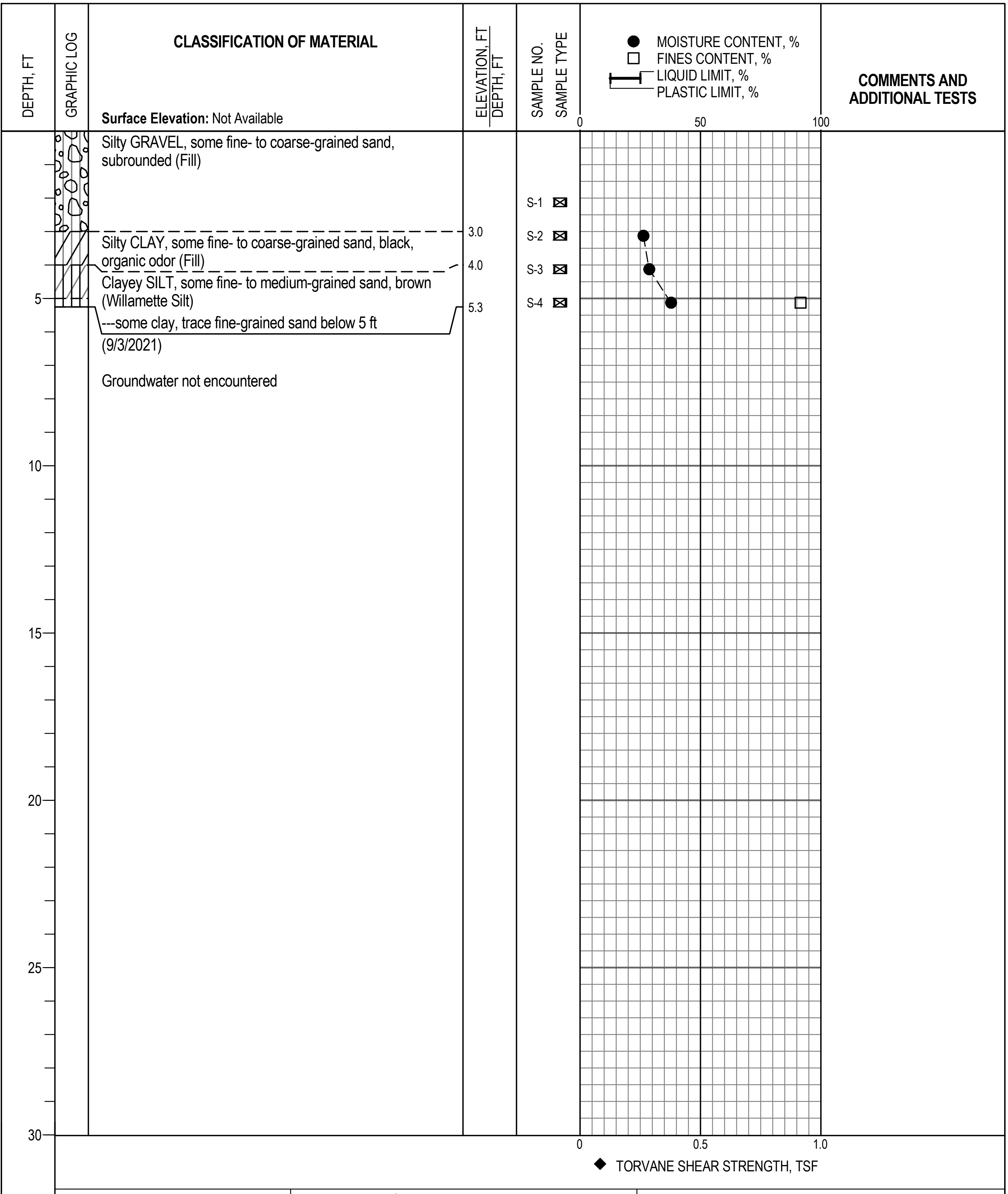
Note: See Legend for Explanation of Symbols

GRI HA LOG (GPS) GRI DATA TEMPLATE.GDT 10/5/21

GRI

BORING HA-1

OCT. 2021 JOB NO. 6519-A FIG. 11A



CLASSIFICATION OF MATERIAL

COMMENTS AND ADDITIONAL TESTS

Surface Elevation: Not Available

Silty GRAVEL, some fine- to coarse-grained sand, subrounded (Fill)

Silty CLAY, some fine- to coarse-grained sand, black, organic odor (Fill)

Clayey SILT, some fine- to medium-grained sand, brown (Willamette Silt)

---some clay, trace fine-grained sand below 5 ft

(9/3/2021)

Groundwater not encountered

Logged By: A. Horst

Excavated by: GRI

Equipment: Hand Auger

Date Started: 9/3/21

GPS Coordinates: 45.295° N 122.9732° W (WGS 84)

Note: See Legend for Explanation of Symbols

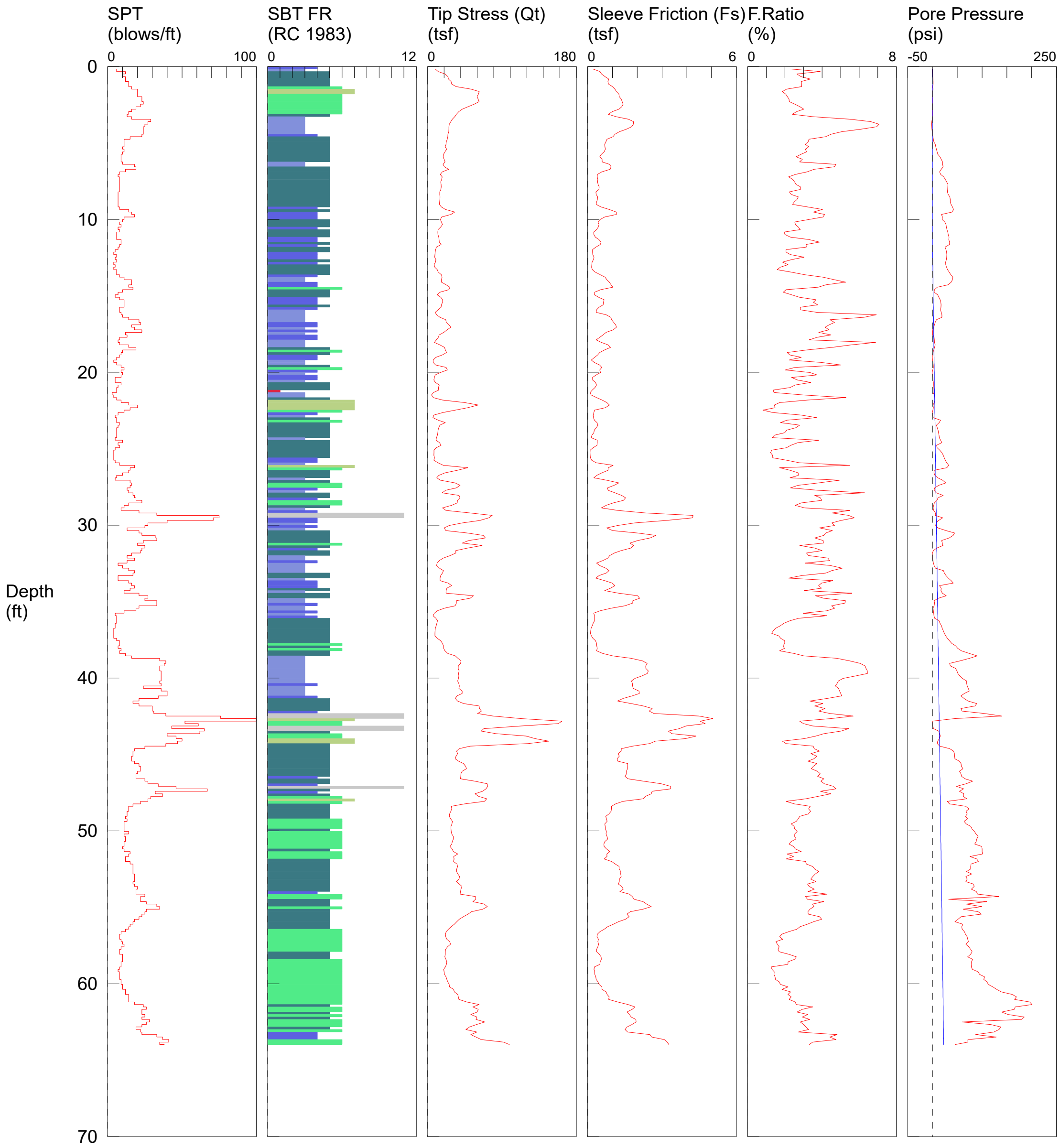
GRI HA LOG (GPS) GRI DATA TEMPLATE.GDT 10/5/21

GRI

BORING HA-2

OCT. 2021 JOB NO. 6519-A FIG. 12A

OPERATOR: OGE DMM
CONE ID: DDG1296
HOLE NUMBER: CPT-1
TEST DATE: 9/3/2021 8:51:01 AM
TOTAL DEPTH: 63.976 ft



🟥 1 sensitive fine grained	🟦 4 silty clay to clay	🟩 7 silty sand to sandy silt	🟧 10 gravelly sand to sand
🟥 2 organic material	🟦 5 clayey silt to silty clay	🟨 8 sand to silty sand	⬜ 11 very stiff fine grained (*)
🟦 3 clay	🟩 6 sandy silt to clayey silt	🟫 9 sand	⬛ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

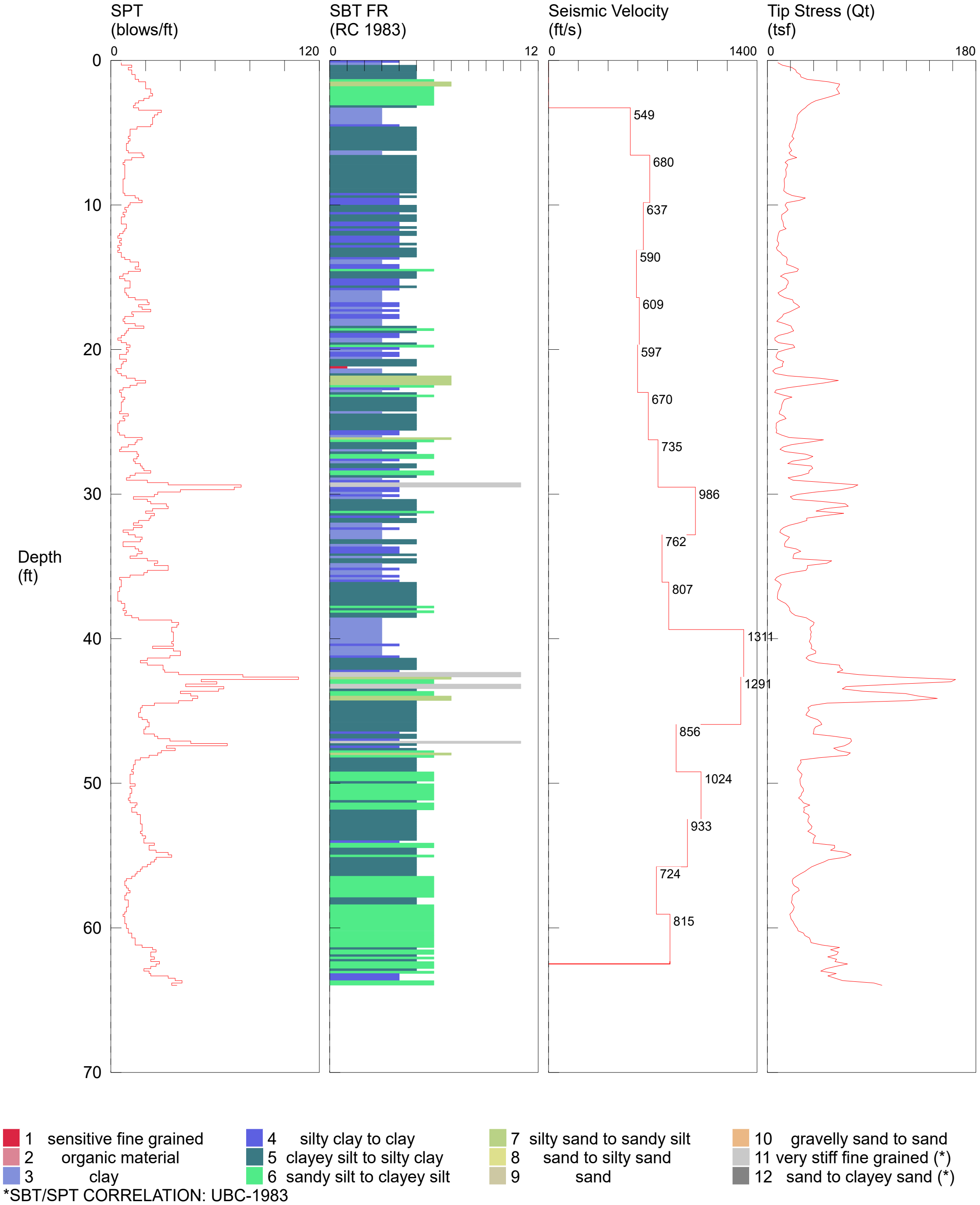
Observed By: B. Cook	**Advanced By:** Oregon Geotechnical Explorations, Inc.
Date Started: 09/03/21	**Ground Surface Elevation:** Not Available
Coordinates: Not Available	

GRI

CONE PENETRATION TEST CPT-1

OCT. 2021 JOB NO. 6519-A FIG. 13A

OPERATOR: OGE DMM
CONE ID: DDG1296
HOLE NUMBER: CPT-1
TEST DATE: 9/3/2021 8:51:01 AM
TOTAL DEPTH: 63.976 ft



*SBT/SPT CORRELATION: UBC-1983

Observed By: B. Cook	**Advanced By:** Oregon Geotechnical Explorations, Inc.
Date Started: 09/03/21	**Ground Surface Elevation:** Not Available
Coordinates: Not Available	

GRI

CONE PENETRATION TEST CPT-1

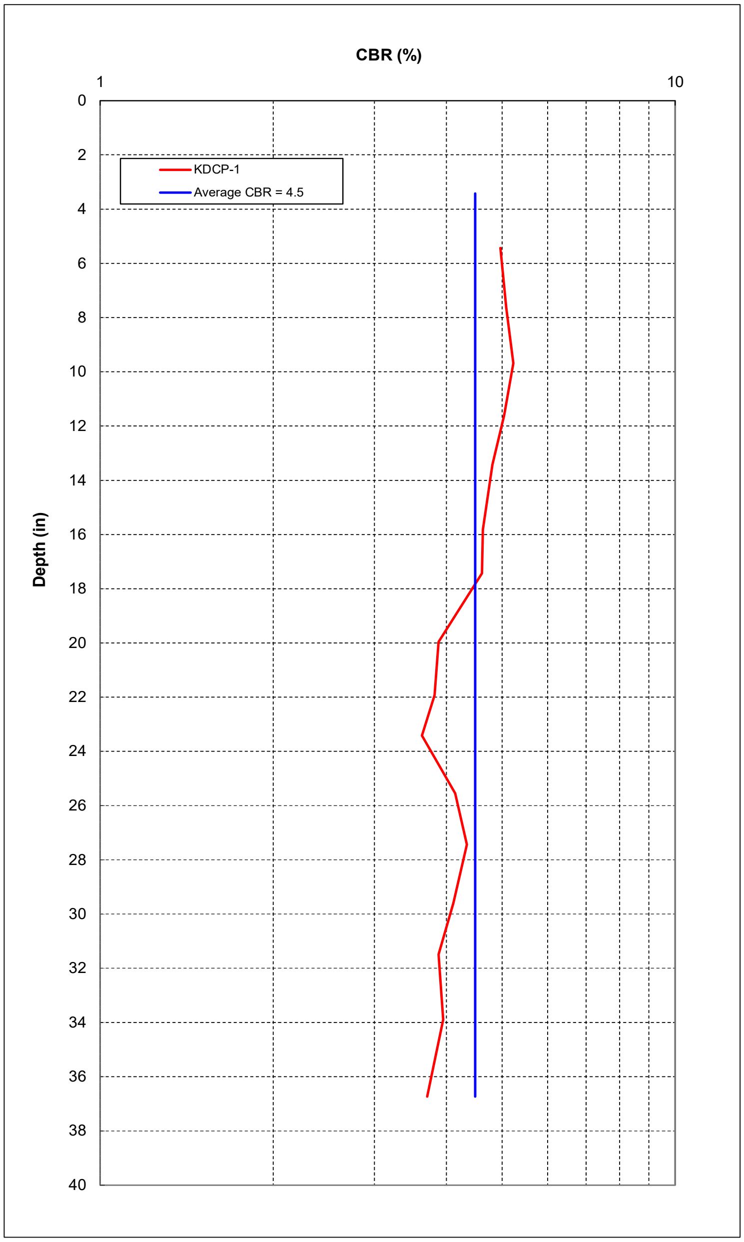
(SEISMIC VELOCITY PROFILE)

OCT. 2021 JOB NO. 6519-A FIG. 14A

GRI	**DYNAMIC CONE PENETROMETER LOG**	**Edwards Elementary School 2022 Addition and Improvements Newberg, OR**
PROJECT NUMBER: 6519-A	DRAWN: GDM	TESTING DATE: 9/2/2021

Boring	PB-1	KDCP-1	Surface Type	Grass
			Hammer	17.6 lb

Depth, in	Depth, mm	Cumulative Blows
3.4	87	4
5.4	138	10
7.6	194	17
9.7	246	24
11.6	295	30
13.4	341	35
15.8	402	41
17.4	443	45
20.0	507	49
21.9	557	52
23.4	595	54
25.6	649	58
27.4	697	62
29.6	752	66
31.5	800	69
33.9	861	73
36.7	933	77



GRI

DYNAMIC CONE PENETROMETER

OCT. 2021 JOB NO. 6519-A FIG. 15A

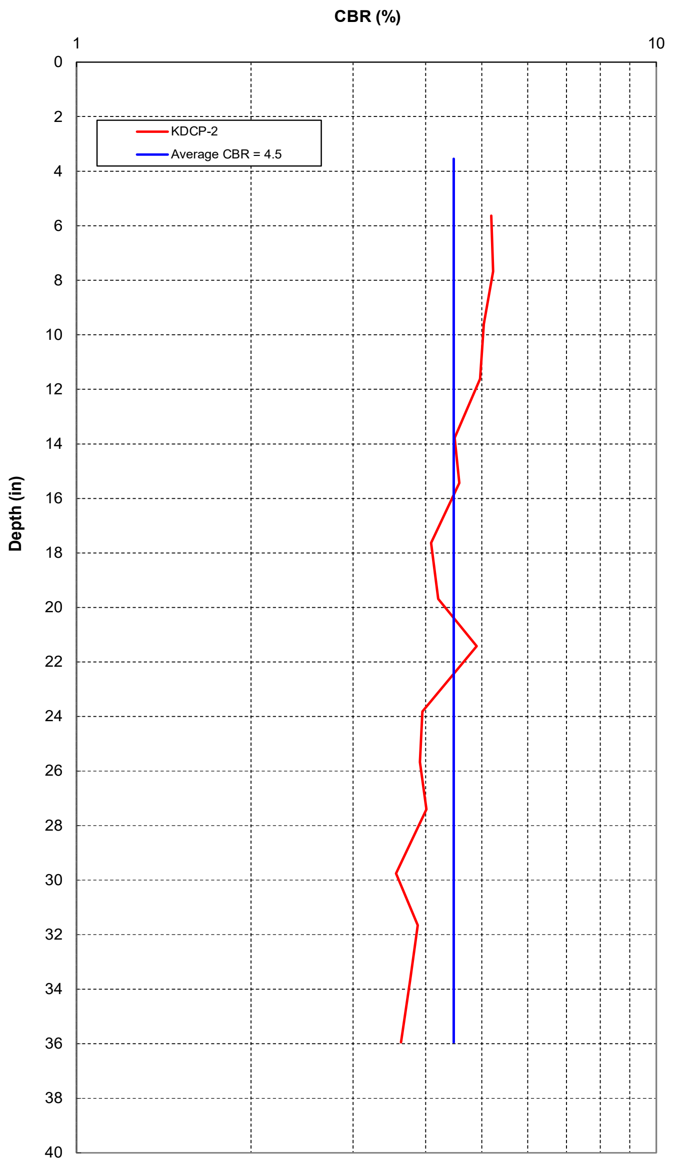
DYNAMIC CONE
PENETROMETER
LOG

Edwards Elementary School
2022 Addition and
Improvements
Newberg, OR

PROJECT NUMBER:	6519-A	DRAWN:	GDM	TESTING DATE:	9/2/2021

Boring	PB-2	KDCP-2	Surface Type	Grass
			Hammer	17.6 lb

Depth, in	Depth, mm	Cumulative Blows
3.5	90	4
5.6	143	11
7.7	195	18
9.6	244	24
11.6	295	30
13.8	350	35
15.4	392	39
17.6	448	43
19.7	500	47
21.4	544	52
23.8	605	56
25.7	652	59
27.4	696	62
29.8	756	65
31.7	804	68
33.7	856	71
35.9	913	74



DYNAMIC CONE PENETROMETER

OCT. 2021 JOB NO. 6519-A FIG. 16A

GRI

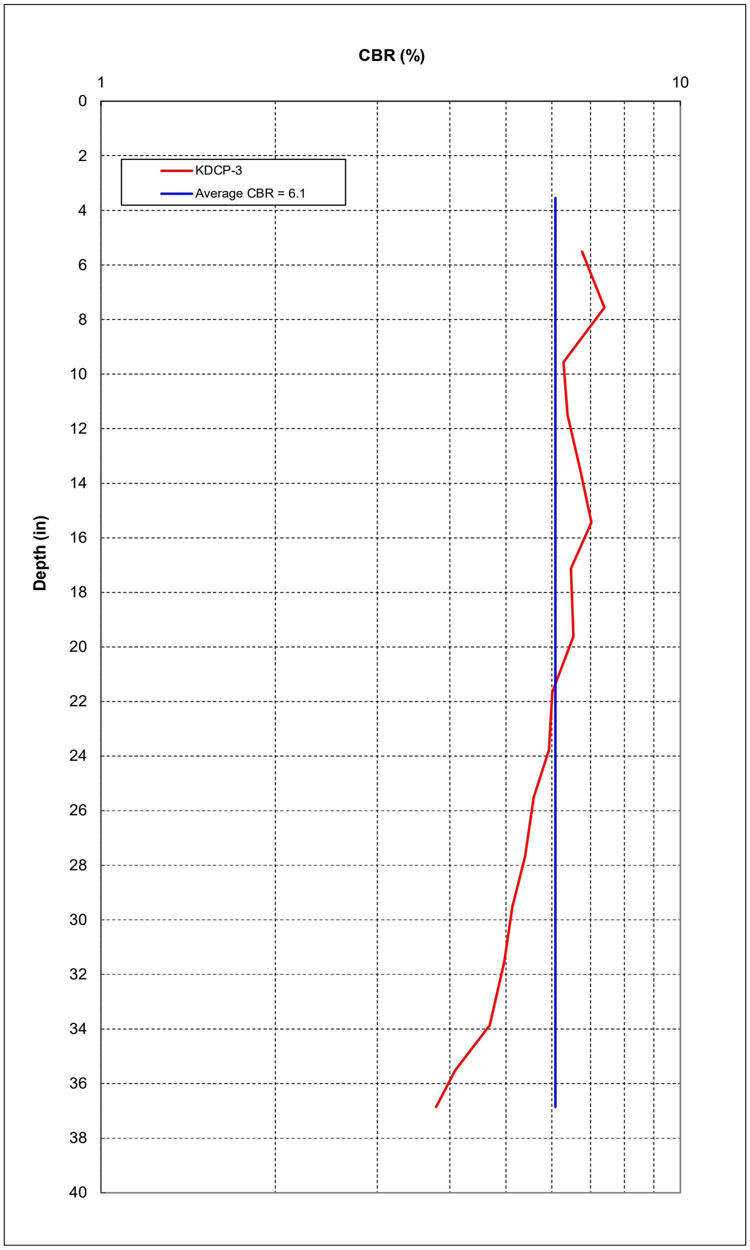
DYNAMIC CONE PENETROMETER LOG

Edwards Elementary School 2022 Addition and Improvements Newberg, OR

PROJECT NUMBER: 6519-A DRAWN: GDM TESTING DATE: 9/1/2021

Boring	KDCP-3	Surface Type	Grass
		Hammer	17.6 lb

Depth, in	Depth, mm	Cumulative Blows
3.5	90	6
5.5	140	19
7.6	192	36
9.6	243	47
11.5	292	58
13.5	343	71
15.4	392	85
17.1	435	95
19.6	498	110
21.7	550	120
23.8	604	130
25.5	648	137
27.7	703	145
29.5	750	151
31.5	801	157
33.9	860	163
35.5	902	166
36.9	936	168



GRI

DYNAMIC CONE PENETROMETER

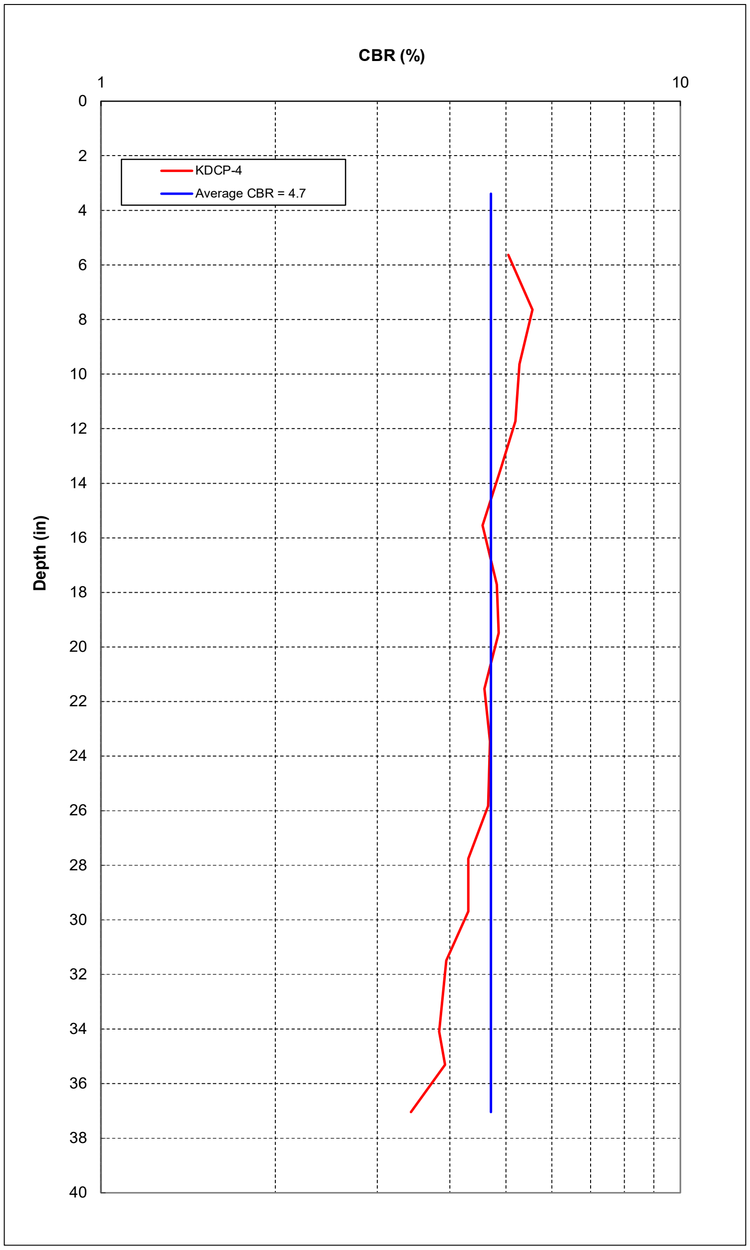
DYNAMIC CONE PENETROMETER LOG

Edwards Elementary School 2022 Addition and Improvements Newberg, OR

PROJECT NUMBER:	6519-A	DRAWN:	GDM	TESTING DATE:	9/1/2021

Boring	KDCP-4	Surface Type	Grass
		Hammer	17.6 lb

Depth, in	Depth, mm	Cumulative Blows
3.4	86	3
5.6	143	10
7.6	194	18
9.6	245	25
11.7	298	32
13.5	342	37
15.6	395	42
17.7	450	48
19.5	495	53
21.5	547	58
23.5	596	63
25.8	656	69
27.8	705	73
29.7	754	77
31.5	800	80
34.1	866	84
35.3	897	86
37.0	941	88

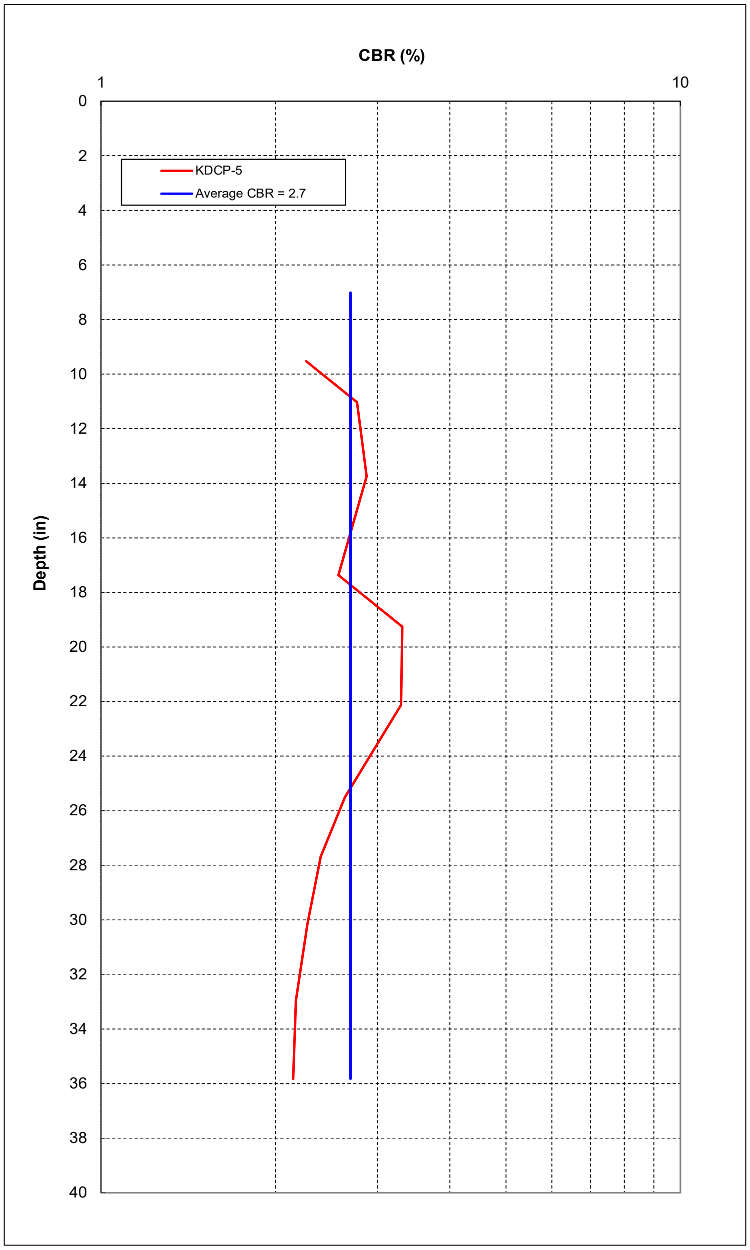


DYNAMIC CONE PENETROMETER

DYNAMIC CONE PENETROMETER LOG		**Edwards Elementary School 2022 Addition and Improvements Newberg, OR**
PROJECT NUMBER: 6519-A	DRAWN: GDM	TESTING DATE: 9/2/2021

Boring	PB-3	KDCP-5	Surface Type	Grass
			Hammer	17.6 lb

Depth, in	Depth, mm	Cumulative Blows
7.0	178	1
9.5	242	2
11.0	280	3
13.7	349	5
17.4	441	7
19.3	489	9
22.1	562	12
25.5	648	14
27.7	703	15
30.2	766	16
33.0	837	17
35.8	910	18

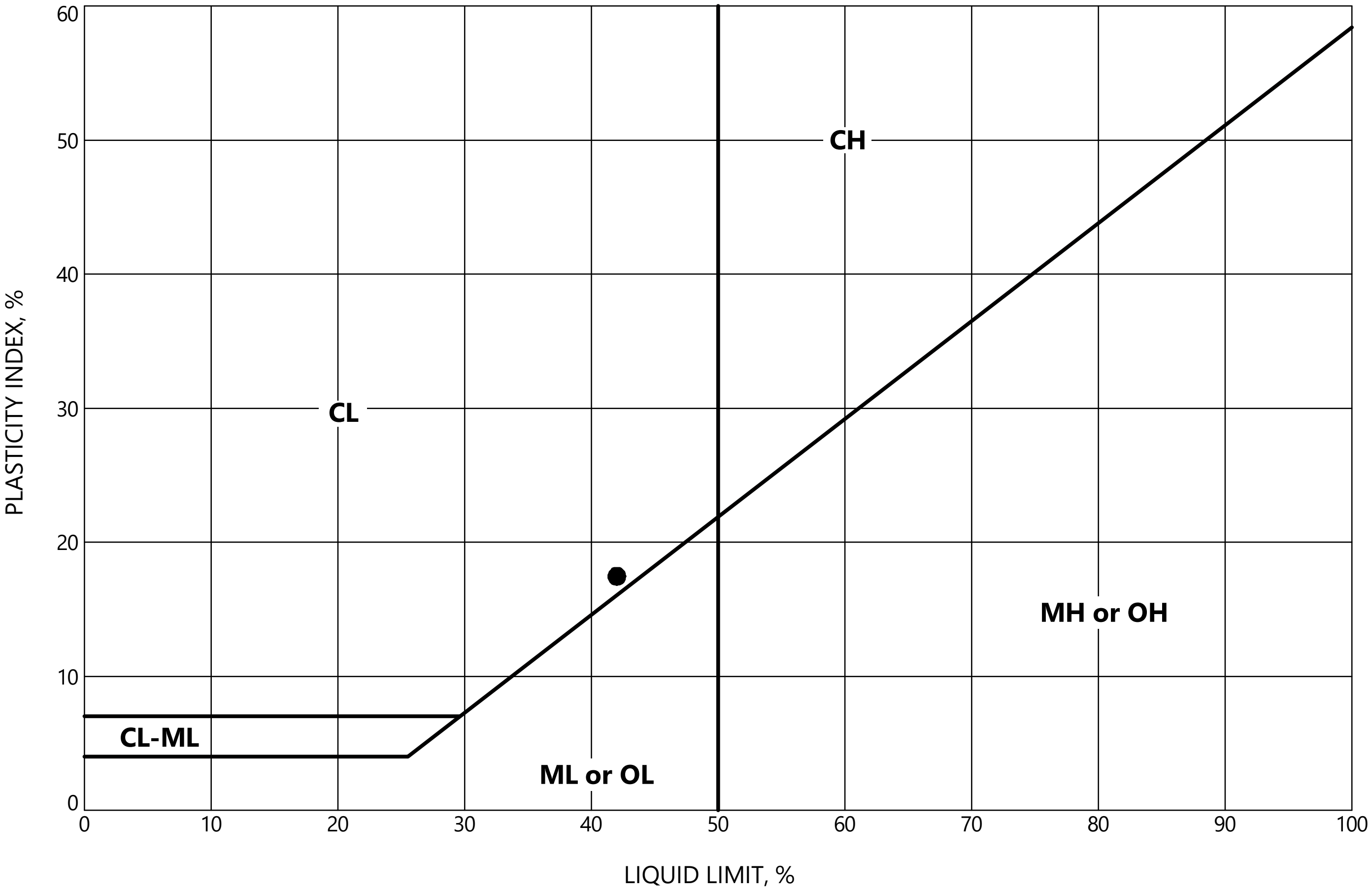


DYNAMIC CONE PENETROMETER

OCT. 2021 JOB NO. 6519-A FIG. 19A

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
ML	INORGANIC CLAYEY SILTS TO VERY FINE SANDS OF SLIGHT PLASTICITY
CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY

GROUP SYMBOL	UNIFIED SOIL CLASSIFICATION FINE-GRAINED SOIL GROUPS
OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
MH	INORGANIC SILTS AND CLAYEY SILT
CH	INORGANIC CLAYS OF HIGH PLASTICITY



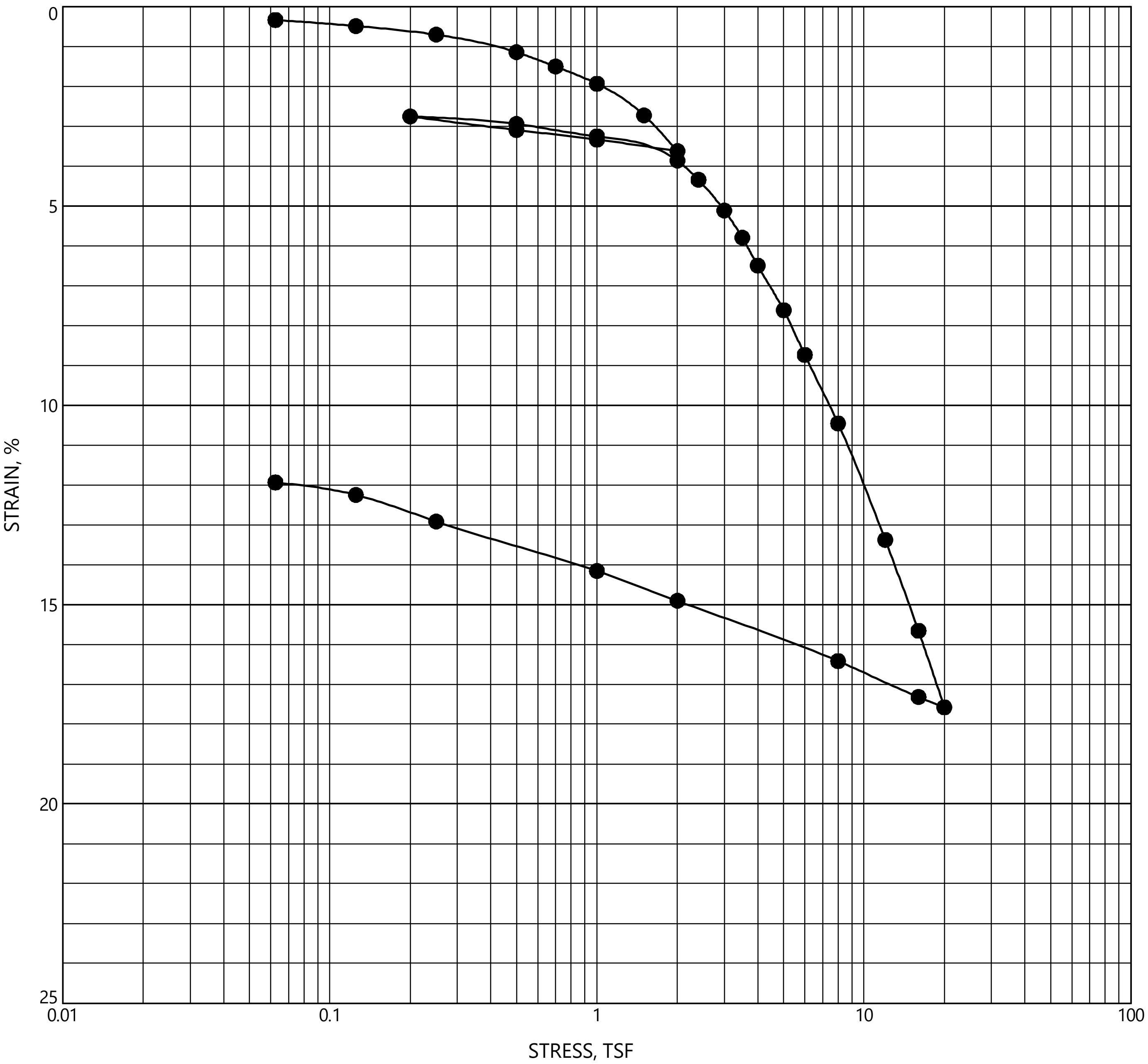
	Location	Sample	Depth, ft	Classification	LL	PL	PI	MC, %
●	PB-2	S-4	10.0	Silty CLAY, up to trace fine-grained sand, dark brown (Willamette Silt)	42	25	17	41

ATTERBERG-PLASTICITY 1 PER PAGE GRI DATA TEMPLATE.GDT 10/4/21

GRI

PLASTICITY CHART

OCT. 2021 JOB NO. 6519-A FIG. 20A

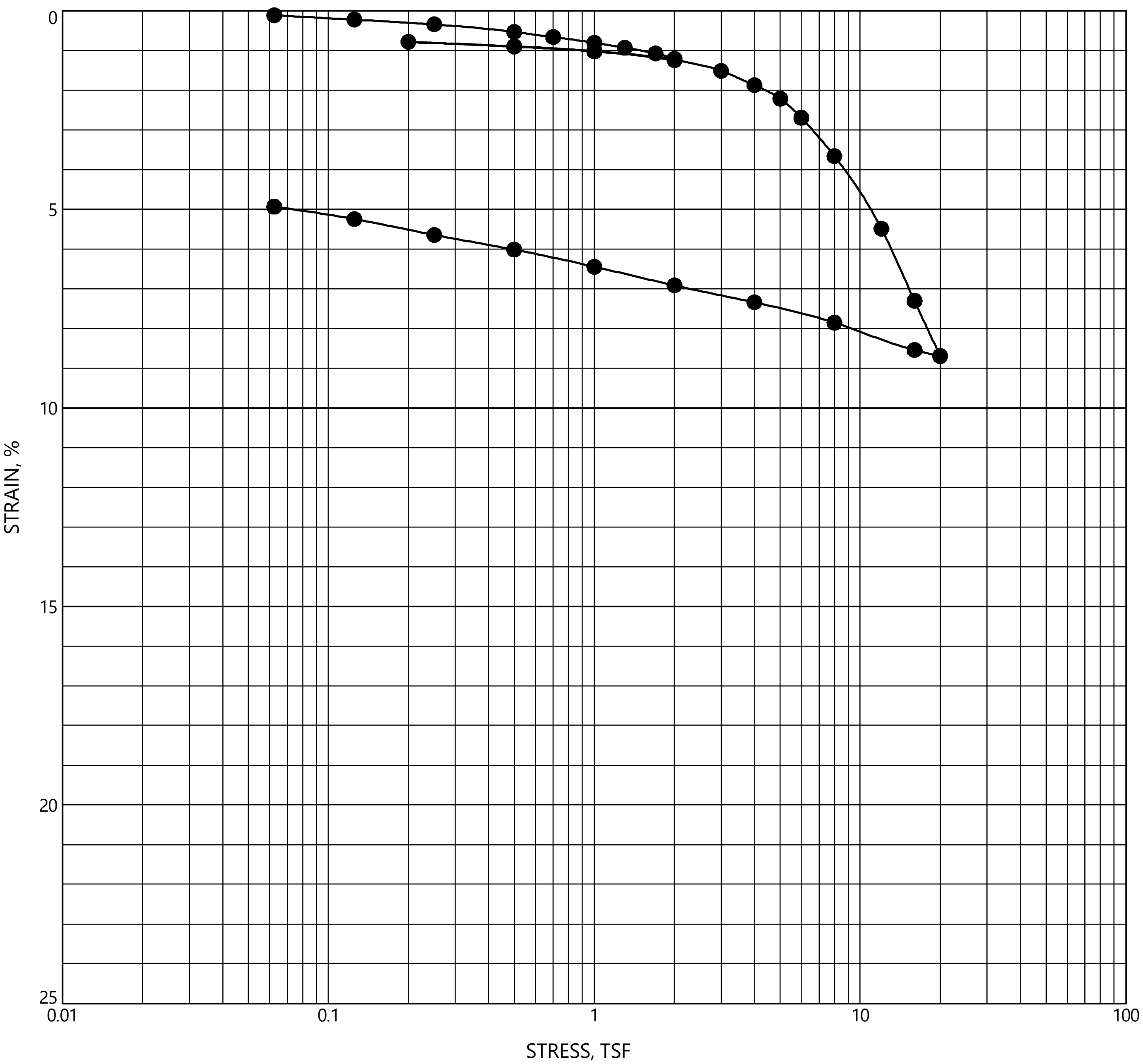


	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-1	S-5	14.5	SILT, some clay to clayey, some fine- to coarse-grained sand, brown mottled gray (Willamette Silt)	87	36

GRI

CONSOLIDATION TEST

OCT. 2021 JOB NO. 6519-A FIG. 21A



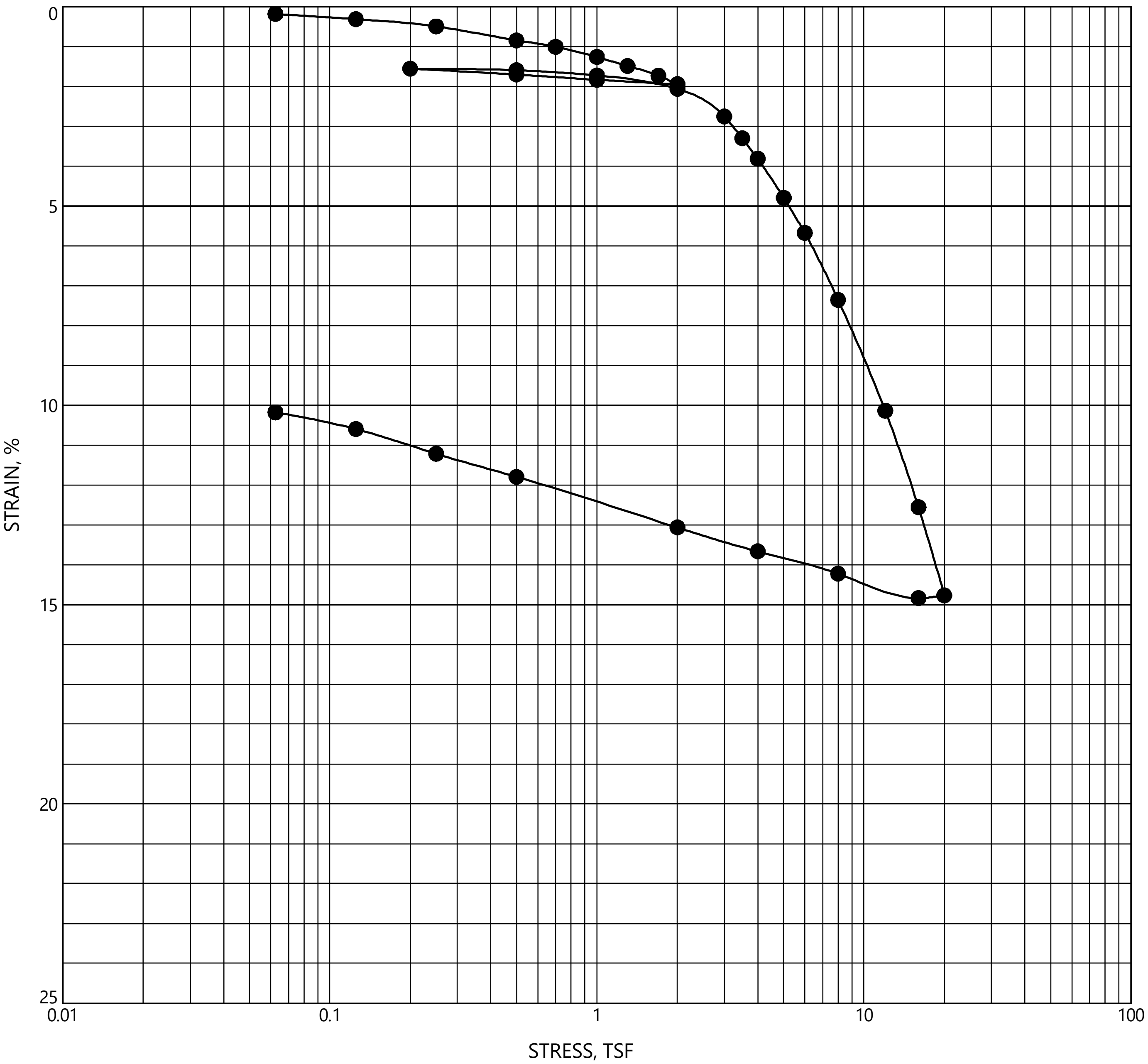
					Initial	
	Location	Sample	Depth, ft	Classification	γ_d, pcf	MC, %
●	B-1	S-8	26.5	SILT, trace to some clay, up to trace fine-grained sand, dark gray (Willamette Silt)	88	34

GRI

CONSOLIDATION TEST

OCT. 2021 JOB NO. 6519-A FIG. 22A

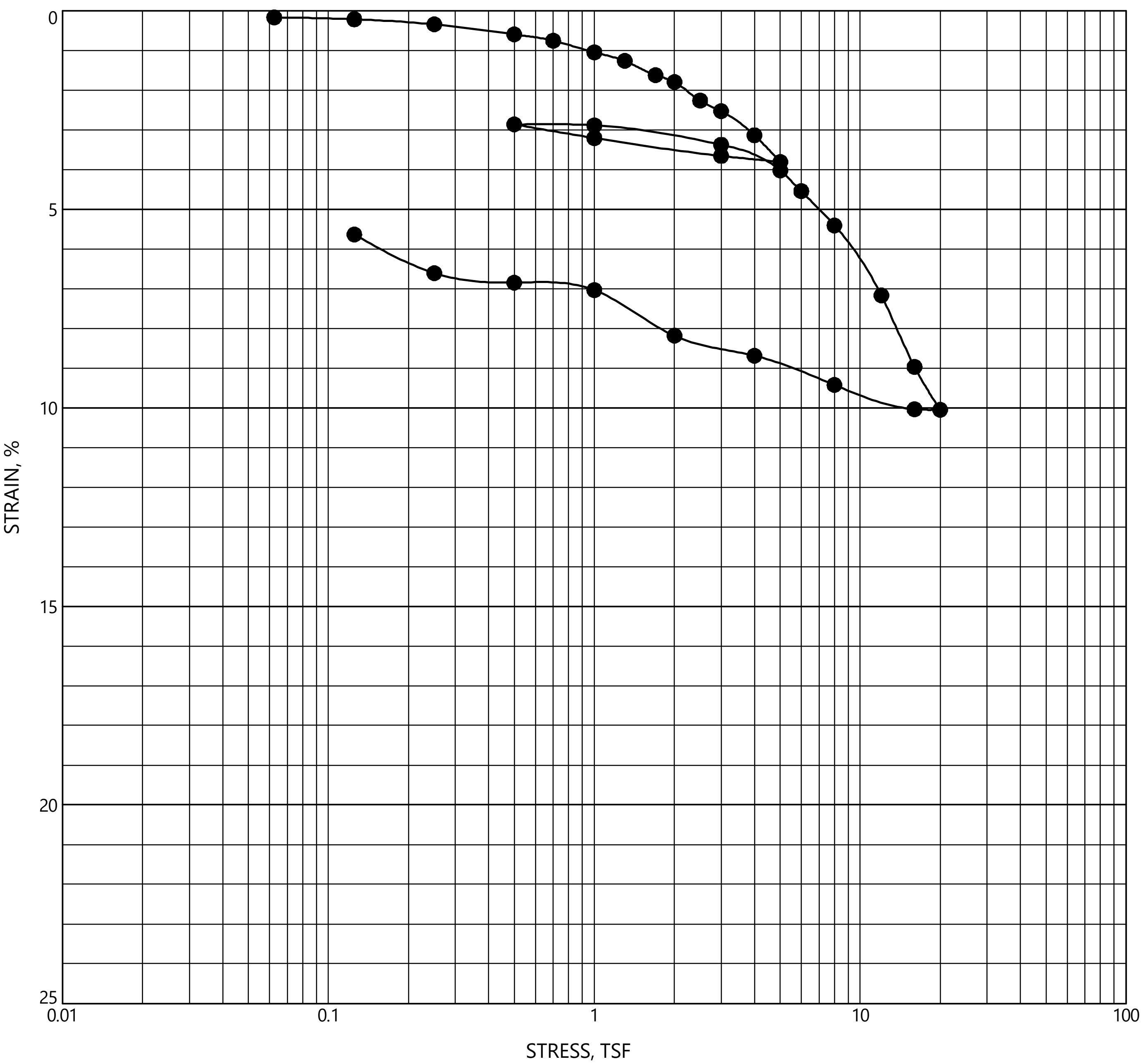
CONSOL STRAIN GRI-0 TO 25-1 PER PAGE GRI DATA TEMPLATE.GDT 10/4/21



	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-3	S-3	8.5	SILT, trace fine-grained sand, up to trace clay, green-gray to brown mottled rust (Willamette Silt)	81	42

GRI

CONSOLIDATION TEST



	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-4	S-13	38.5	Sandy CLAY, some silt and fine-grained sand, gray (Hillsboro Formation)	100	28

CONSOL STRAIN GRI-0 TO 25-1 PER PAGE GRI DATA TEMPLATE.GDT 10/4/21

GRI

CONSOLIDATION TEST

DRAFT

APPENDIX B

Site-Specific Seismic-Hazard Evaluation

APPENDIX B

SITE-SPECIFIC SEISMIC-HAZARD EVALUATION

B.1 INTRODUCTION

GRI completed a site-specific seismic-hazard evaluation for the proposed project improvements at Edwards Elementary School located in Newberg, Oregon. The purpose of the evaluation was to review the potential seismic hazards associated with regional and local seismicity. We understand the proposed improvements will be considered a special-occupancy structure as defined by Oregon Revised Statute (ORS) 455.447. The site-specific seismic-hazard evaluation is intended to fulfill the requirements of amended Section 1803 of the 2019 Oregon Structural Specialty Code (OSSC) for special-occupancy structures, which references 2016 American Society of Civil Engineers (ASCE) 7-16 document, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 7-16), for seismic design. Our site-specific seismic-hazard evaluation was based on the potential for regional and local seismic activity, as described in the existing scientific literature, and the subsurface conditions at the site, as disclosed by the geotechnical explorations completed for the project. Specifically, our work included the following tasks:

1. A review of available literature, including published papers, maps, open-file reports, seismic histories and catalogs, and other sources of information regarding the tectonic setting, regional and local geology, and historical seismic activity that might have a significant effect on the site.

2. Compilation and evaluation of subsurface data collected at and in the vicinity of the site, including classification and laboratory analyses of soil samples. This information was used to prepare a generalized subsurface profile for the site.

3. Identification of potential seismic sources appropriate for the site, and characterization of those sources in terms of magnitude, distance, and acceleration response spectra.

4. Engineering analyses based on the generalized subsurface profile and potential seismic sources resulting in conclusions and recommendations concerning:

    a. Specific seismic events and characteristic earthquakes that might have a significant effect on the project site.

    b. The potential for seismic energy amplification and liquefaction or soil-strength loss at the site.

c. Site-specific acceleration response spectra for design of structures at the site.

This appendix describes the work accomplished and summarizes our conclusions and recommendations.

B.2 GEOLOGIC SETTING

B.2.1 General

On a regional scale, the site is located in the northern end of the Willamette River Valley of the Puget-Willamette lowland trough of the Cascadia convergent tectonic system (Blakely et al., 2000). The lowland areas consist of broad, north-south-trending basins in the underlying geologic structure between the Coast Range to the west and the Cascade Mountains to the east. The lowland trough is characterized by alluvial plains with areas of buttes and terraces. The site is located approximately 77 kilometers inland from the rupture zone of the Cascadia Subduction Zone (CSZ), an active convergent-plate boundary along which remnants of the Farallon Plate (the Gorda, Juan de Fuca, and Explorer plates) are being subducted beneath the western edge of the North American continent. The subduction zone is a broad, eastward-dipping zone of contact between the upper portion of the subducting slabs and the overriding North American Plate, as shown on Figure 1B.

On a local scale, the site is located in the Tualatin Basin, a large, southeast-trending structural basin bounded by high-angle, northwest-trending, right-lateral, strike-slip faults considered to be seismogenic. The geologic units in the area are shown on the Regional Geologic Map, Figure 2B. The distribution of nearby Quaternary faults is shown on the Local Fault Map, Figure 3B. Information regarding the continuity and potential activity of these faults is lacking due largely to the scale at which geologic mapping in the area has been conducted and the presence of thick, relatively young, basin-filling sediments that obscure underlying structural features. Active faults may be present within the basin, but clear stratigraphic and/or geophysical evidence regarding their location and extent is not presently available. Additional discussion regarding crustal faults is provided in the Local Crustal Event section below.

Because of the proximity of the site to the CSZ and its location within the Tualatin Basin, three distinctly different seismic sources contribute to the potential for damaging earthquake motions at the site. Two of these sources are associated with deep-seated tectonic activity related to the CSZ; the third is associated with movement on relatively shallow faults within and adjacent to the Portland Basin.

B.2.2 Subsurface and Geologic Conditions

Published geologic mapping indicates the site is mantled with Missoula flood deposits, locally referred to in the project area as the Willamette Silt Formation. In general, Willamette Silt is composed of beds and lenses of clay, silt, and sand. Stratification within this formation commonly consists of 4- to 6-inch-thick beds, although in some areas, the clay, silt, and sand are massive, and the bedding is indistinct or nonexistent (Wells et al., 2018). The Hillsboro Formation, which typically consists of stiff to very stiff, brown to gray clay, commonly underlies the Willamette Silt at depths of about 40 feet to 60 feet in this area (Ma et al., 2009).

B.3 SEISMICITY

B.3.1 General

The available information indicates the potential seismic sources that may affect the site can be grouped into three independent categories: *subduction-zone events* related to sudden slip between the upper surface of the Juan de Fuca Plate and the lower surface of the North American Plate, *subcrustal events* related to deformation and volume changes within the subducted mass of the Juan de Fuca Plate, and *local crustal events* associated with movement on shallow, local faults within and adjacent to the Portland Basin. Each of these sources is considered capable of producing damaging earthquakes in the Pacific Northwest. Based on our review of currently available information, we developed generalized design earthquakes for each of these categories. The design earthquakes are characterized by three important properties: size, location relative to the subject site, and the peak horizontal bedrock accelerations produced by the event. In this study, earthquake size is generally expressed by moment magnitude (M_W); location is expressed as the closest distance to the fault rupture, measured in kilometers; and peak horizontal bedrock accelerations are expressed in units of gravity (1 g = 32.2 feet/second^2 = 981 centimeters/second^2).

B.3.2 Cascadia Subduction Zone (CSZ) Event

Written Japanese tsunami records suggest a great CSZ earthquake occurred in January 1700 (Atwater et al., 2015). Geological studies suggest great megathrust earthquakes have occurred repeatedly in the past 7,000 years (Atwater et al., 1995; Clague, 1997; Goldfinger et al., 2003; and Kelsey et al., 2005), and geodetic studies (Hyndman and Wang, 1995; and Savage et al., 2000) indicate rate of strain accumulation consistent with the assumption that the CSZ is locked beneath offshore northern California, Oregon, Washington, and southern British Columbia (Fluck et al., 1997; and Wang et al., 2001). Numerous geological and geophysical studies suggest the CSZ may be segmented (Hughes and Carr, 1980; Weaver and Michaelson, 1985; Guffanti and Weaver, 1988; Goldfinger, 1994; Kelsey and Bockheim, 1994; Mitchell et al., 1994; Personius, 1995; Nelson and Personius, 1996; and Witter, 1999), but the most recent studies suggest that for the last great earthquake in 1700, most of the subduction zone ruptured in a single M_W 9 earthquake (Satake et al., 1996; Atwater and Hemphill-Haley, 1997; and Clague et al., 2000). Published estimates of the probable maximum size of subduction-zone events range from M_W 8.3 to >M_W 9.

Numerous detailed studies of coastal subsidence, tsunamis, and turbidites yield a wide range of recurrence intervals, but the most complete records (>4,000 years) indicate intervals of about 350 years to 600 years between great earthquakes on the CSZ (Adams, 1990; Atwater and Hemphill-Haley, 1997; Witter, 1999; Clague et al., 2000; Kelsey et al., 2002; Kelsey et al., 2005; and Witter et al., 2003). Tsunami inundation in buried marshes along the Washington and Oregon coast and stratigraphic evidence from the Cascadia margin support these recurrence intervals (Kelsey et al., 2005; and Goldfinger et al., 2003). Goldfinger et al. (2003, 2012, and 2016) evaluated turbidite evidence for 20 earthquakes that ruptured the entire CSZ over the past 10,000 years and about 20 M_W 8 earthquakes that only ruptured along the southern portion of the CSZ and developed a model for recurrence of CSZ M_W 8 to M_W 9 earthquakes.

The U.S. Geological Survey (USGS) probabilistic analysis assumes four potential locations (three alternative down-dip edge options and one up-dip edge option) for the eastern edge of the earthquake rupture zone for the CSZ, as shown on Figure 4B. As discussed in Petersen et al. (2014), the 2014 USGS mapping effort represents the 2014 CSZ source model with the full CSZ ruptures with moment magnitudes from M_W 8.6 to M_W 9.3 supplemented by partial ruptures with smaller magnitudes from M_W 8.0 to M_W 9.1. The partial ruptures were accounted for using a segmented model and an unsegmented model. The magnitude-frequency distribution showing the contributions to the earthquake rates from each of the models and how the rates vary along the fault is presented on Figure 5B. In general, the earthquake rates along the CSZ are dominated by the full-characteristic ruptures, with one event in 526 years (M_W 8.6 to M_W 9.3 earthquakes likely occur more often than the smaller, segmented ruptures). Therefore, in our opinion, the CSZ event should be represented by an earthquake of M_W 9.0 at a focal depth of 30 kilometers and rupture distance of about 77 kilometers.

B.3.3 Subcrustal Event

There is no historical earthquake record of significant (i.e., >M_W 6.0) subcrustal, intraslab earthquakes in Oregon. Although both the Puget Sound and northern California regions have experienced many of these earthquakes in historical times, Wong (2005) hypothesizes that due to subduction-zone geometry, geophysical conditions, and local geology, Oregon may not be subject to intraslab earthquakes. In the Puget Sound area, these moderate to large earthquakes are deep (40 kilometers to 60 kilometers) and more than 200 kilometers from the deformation front of the subduction zone. Offshore along the northern California coast, the earthquakes are shallower (up to 40 kilometers) and located along the deformation front. Estimates of the probable size, location, and frequency of subcrustal events in Oregon are generally based on comparisons of the CSZ with active convergent-plate margins in other parts of the world and the historical seismic record for the region surrounding Puget Sound, where significant events known to have occurred within the subducting Juan de Fuca Plate have been recorded. The 1949, 1965, and 2001 documented subcrustal earthquakes in the Puget Sound area correspond to M_W 7.1, M_W 6.5, and M_W 6.8, respectively. Published estimates of the probable maximum size of these events range from M_W 7.0 to M_W 7.5. Published information regarding the location and geometry of the subducting zone indicates a focal depth of 50 kilometers is probable (Weaver and Shedlock, 1989). In our opinion, it is appropriate to represent the subcrustal event by a design earthquake of M_W 7.0 at a focal depth of 50 kilometers and rupture distance of 60 kilometers.

B.3.4 Local Crustal Event

Sudden crustal movements along relatively shallow, local faults in the Portland area, although rare, have been responsible for local crustal earthquakes. The precise relationship between specific earthquakes and individual faults is not well understood since few of the faults in the area are expressed at the ground surface and the foci of the observed earthquakes have not been located with precision. The history of local seismic activity is commonly used as a basis for determining the size and frequency to be expected of local crustal events. Although the historical record of local earthquakes is relatively short (the earliest reported seismic event in the area occurred in 1920), it can serve as a guide for estimating the potential for seismic activity in the area.

Based on fault mapping conducted by the USGS (2014 National Seismic Hazard Maps [NSHMs]), there are three faults within about 25 kilometers of the site the USGS identifies as contributing to the crustal seismic hazard: the Newberg Fault, located approximately 0.1 miles (0.2 kilometers) northeast of the site; the Mount Angel Fault, located about 10.5 miles (16.9 kilometers) southeast of the project site; and the Gales Creek Fault Zone, located about 15.6 miles (25 kilometers) from the project site. Based on our review of the faults that contribute to the overall seismicity of the site, the Newberg Fault is the closest dominant crustal fault identified as a hazard to the site, with a magnitude of M_W 6.7.

B.4 CODE BACKGROUND AND DESIGN RESPONSE SPECTRUM

B.4.1 General

We understand the project will be designed in accordance with the 2019 OSSC, which references ASCE 7-16, for seismic design. A site-specific seismic-hazard evaluation was completed for the project to fulfill the requirements of amended Section 1803 of the 2019 OSSC for special-occupancy structures.

B.4.2 Code Background

The ASCE 7-16 seismic-hazard levels are based on a Risk-Targeted Maximum Considered Earthquake (MCE_R) with the intent of including the probability of structural collapse. Based on generalized building fragility curves, seismic design of a structure using the probabilistic MCE_R represents a targeted risk level of 1% in 50 years probability of collapse in the direction of maximum horizontal response. In general, these risk-targeted ground motions are developed by applying adjustment factors of directivity and risk coefficients to the 2% probability of exceedance in 50 years (2,475-year return-period hazard level) ground motions developed from the recently updated 2014 USGS probabilistic seismic-hazard maps. The risk-targeted probabilistic values are also subject to a deterministic check, which is computed from the models of earthquake sources and ground-motion propagation that form the basis of the 2014 USGS NSHMs. ASCE 7-16 defines the site-specific deterministic MCE_R ground motions in terms of 84th-percentile, 5%-damped response spectral acceleration in the direction of maximum horizontal response. The MCE_R ground motions are taken as the lesser of the probabilistic and deterministic spectral accelerations.

The ASCE methodology uses two bedrock spectral response mapped acceleration parameters, S_S and S_1, corresponding to periods around 0.2 second and 1.0 second to develop the MCE_R response spectrum. To establish the ground-surface MCE_R spectrum, these mapped bedrock spectral parameters are adjusted for site class using the short- and long-period site coefficients, F_a and F_v, in accordance with Section 11.4.3 of ASCE 7-16, which includes new seismic site coefficients to adjust the mapped values for soil properties.

B.4.3 Mapped Acceleration Parameters

ASCE 7-16 uses two bedrock spectral response acceleration parameters, S_S and S_1, corresponding to periods of about 0.2 second and 1.0 second to develop the MCE_R response spectrum for Site Class B/C, or bedrock conditions. The ASCE 7-16 S_S and S_1 mapped spectral response acceleration parameters for the site located at the approximate latitude and longitude coordinates of 45.2951° N and 122.9727° W are 0.85 g and 0.41 g, respectively, for Site Class B/C, or bedrock conditions.

B.4.4 Site Class

Based on the subsurface conditions disclosed by the explorations and in accordance with Section 20.4 of ASCE 7-16, the site is classified as Site Class D, or a stiff-soil site, based on an average shear-wave velocity (field-measured shear-wave velocity [V_S]) in the upper 100 feet of the soil profile. However, our analysis identified a potential risk of seismically induced settlement at the site. In accordance with Section 20.3.1 of ASCE 7-16, sites with soils vulnerable to failure or collapse under seismic loading should be classified as Site Class F, which requires a site-specific, site-response analysis unless the structure has a fundamental period of vibration less than or equal to 0.5 second. The design response spectrum for sites with structures having a fundamental period of less than or equal to 0.5 second can be derived using the non-liquefied subsurface profile and code-tabulated site coefficients. We anticipate the new structure will have a fundamental period of less than

0.5 second; therefore, the code-based Site Class D conditions are appropriate for the design of the structure.

B.4.5 Site Coefficients

Due to the S_1 acceleration parameter being greater than or equal to 0.2 g, Section 11.4.8 of ASCE 7-16 requires a ground-motion hazard analysis unless the seismic response coefficient C_s is determined in accordance with Exception 2 of Section 11.4.8 of ASCE 7-16. Assuming the seismic response coefficient C_s is determined in accordance with Exception 2 of Section 11.4.8 of ASCE 7-16, the site coefficients F_a and F_v were determined from code-tabulated values to be 1.16 and 1.89, respectively, in accordance with Section 11.4 of ASCE 7-16. The site coefficients F_a and F_v were used to develop the Site Class D, MCE_R-level spectrum in accordance with Section 11.4 of ASCE 7-16.

B.4.6 Recommended Seismic Design Parameters

The design-level response spectrum is calculated as two thirds of the ground-surface MCE_R spectrum. The recommended MCE_R- and design-level spectral-response parameters for Site Class D conditions are provided below in Table 2B.

Table 2B: RECOMMENDED SEISMIC DESIGN PARAMETERS (2019 OSSC/ASCE 7-16)

Seismic Parameter	Recommended Values*
Site Class	D
MCE_R 0.2-Sec Period Spectral Response Acceleration, S_{MS}	0.99 g
MCE_R 1.0-Sec Period Spectral Response Acceleration, S_{M1}	0.78 g
Design-Level 0.2-Sec Period Spectral Response Acceleration, S_{DS}	0.66 g
Design-Level 1.0-Sec Period Spectral Response Acceleration, S_{D1}	0.52 g

Note: *Exception 2 of Section 11.4.8 should be considered when evaluating base shear calculations in Section 12.8.

B.4.7 Liquefaction/Cyclic Softening

Liquefaction is the process by which loose, saturated granular materials, such as clean sand and, to a somewhat lesser degree, non-plastic and low-plasticity silts, temporarily lose stiffness and strength during and immediately after a seismic event. This degradation in soil properties may be substantial and abrupt, particularly in loose sands. Liquefaction occurs as seismic shear stresses propagate through saturated soil and distort the soil structure, causing loosely packed groups of particles to contract or collapse. If drainage is impeded and cannot occur quickly, the collapsing soil structure causes the pore-water pressure to increase between the soil grains. If the pore-water pressure becomes sufficiently large, the intergranular stresses become small, and the granular layer temporarily behaves as a viscous liquid rather than a solid. After liquefaction is triggered, there is an increased risk of settlement, loss of bearing capacity, lateral spreading, and/or slope instability, particularly along waterfront areas. Liquefaction-induced settlement occurs as the elevated pore-water pressures dissipate and the soil consolidates after the earthquake.

"Cyclic softening" is a term that describes a relatively gradual and progressive increase in shear strain with load cycles and is more common within fine-grained soils. Excess pore pressures may increase due to cyclic loading but will generally not approach the total overburden stress. Shear strains accumulate with additional loading cycles, but an abrupt or sudden decrease in shear stiffness is not typically expected. Settlement due to post-seismic consolidation can occur, particularly in lower-plasticity silts. Large shear strains can develop, and strength loss related to soil sensitivity may be a concern.

The potential for liquefaction and/or cyclic softening is typically estimated using a simplified method that compares the cyclic shear stresses induced by the earthquake (demand) to the cyclic shear strength of the soil available to resist these stresses (resistance). Estimates of seismically induced stresses are based on earthquake magnitude (M_W) and PGA. The cyclic resistance of soils is dependent on several factors, including the number of loading cycles, relative density, confining stress, plasticity, natural water content, stress history, age, depositional environment (fabric), and composition. The cyclic resistance of soils is evaluated using in-situ testing in conjunction with laboratory index testing but may also include monotonic and cyclic laboratory strength tests. For sand-like soils, the cyclic resistance is typically evaluated using SPT N-values or CPT tip-resistance values normalized for overburden pressures and corrected for factors that influence cyclic resistance, such as fines content. For clay-like soils, the cyclic resistance is typically evaluated using estimates of the undrained shear strength, overconsolidation ratio, and sensitivity or directly from cyclic laboratory tests.

The potential for liquefaction and/or cyclic softening at the site was evaluated using the simplified method based on procedures recommended by Idriss and Boulanger (2008) with subsequent revisions (2014). This method utilizes the PGA to predict the cyclic shear stresses induced by the earthquake. The USGS National Seismic Hazard Mapping Project (NSHMP) was used to determine the contributing earthquake magnitudes that represent the seismic exposure of the site for the Maximum Considered Earthquake Geometric Mean

(MCE_G) hazard level. A crustal event on the Gales Creek Fault and an event on the CSZ were determined to represent the sources of seismic shaking.

For our evaluation, we considered an M_W 6.7 crustal earthquake at a distance of about 2.2 kilometers and M_W 9.0 CSZ earthquake at a distance of about 77 kilometers with code-level PGAs (PGA_M) of 0.48 g and 0.39 g, respectively. We assumed a groundwater depth of about 10 feet below the ground surface, which corresponds to the highest assumed year-round sustained groundwater level at the site. The results of our evaluation indicate there is a potential that the interbedded lenses of sand below the groundwater surface at the site could experience limited liquefaction, and zones of the low-plasticity silt below the groundwater surface at the site could experience limited cyclic softening. Our analysis indicates the potential for up to about 1 inch of seismically induced settlement that may occur during the earthquake and after earthquake shaking has ceased.

B.4.8 Other Seismic Hazards

Based on subsurface conditions and site topography, the risk of earthquake-induced slope instability and/or lateral spreading is low. The risk of damage by tsunami and/or seiche at the site is absent. The USGS considers the Newberg Fault, located approximately 0.1 miles (0.2 kilometers) northeast of the site, and the Mount Angel Fault, located about 10.5 miles (16.9 kilometers) southeast of the project site, to be the closest crustal fault sources contributing to the overall seismic hazard at the site. The CSZ is mapped approximately 77 kilometers west of the site (Petersen et al., 2014). Unless occurring on a previously unmapped or unknown fault, the risk of fault rupture at the site is low.

B.5 CONCLUSIONS

Based on our review of the ASCE 7-16 design methodology, we recommend the project site at the approximate latitude and longitude coordinates of 45.2951° N and 122.9727° W be designed using the mapped spectral acceleration parameters of S_S and S_1 of 0.85 g and 0.41 g, respectively. We recommend using the Site Class D design spectrum and tabulated code values for design of the proposed improvements.

B.6 REFERENCES

Adams, J., 1990, Paleoseismicity of the Cascadia subduction zone: Evidence from turbidites off the Oregon-Washington margin: Tectonics, v. 9, no. 4, pp. 569-583.

American Society of Civil Engineers, 2017, Minimum design loads and associated criteria for buildings and other structures, ASCE 7-16.

Atwater, B. F., Nelson, A. R., Clague, J. J., Carver, G. A., Yamaguchi, D. K., Bobrowsky, P. T., Bourgeois, J., Darienzo, M. E., Grant, W. C., Hemphill-Haley, E., Kelsey, H. M., Jacoby, G. C., Nishenko, S. P., Palmer, S. P., Peterson, C. D., and Reinhart, M. A., 1995, Summary of coastal geologic evidence for past great earthquakes at the Cascadia subduction zone: Earthquake Spectra, v. 11, no. 1, pp. 1-18.

Atwater, B. F., and Hemphill-Haley, E., 1997, Recurrence intervals for great earthquakes of the past 3,500 years at northeastern Willapa Bay, Washington: USGS, Professional Paper 1576, 108 p.

Atwater, B. F., Musumi-Rokkaku, S., Satake, K., Tsuji, Y., Ueda, K., and Yamaguchi, D. K., 2015, The orphan tsunami of 1700—Japanese clues to a parent earthquake in North America, 2nd ed., USGS, Professional Paper 1707, 135 p.

Blakely, R. J., Wells, R. E., Tolan, T. L., Beeson, M. H., Trehu, A. M., and Liberty, L. M., 2000, New aeromagnetic data reveal large strike-slip (?) faults in the northern Willamette Valley, Oregon: Geological Society of America Bulletin 112, no. 8, pp. 1225-1233.

Clague, J. J., 1997, Evidence for large earthquakes at the Cascadia subduction zone: Reviews of Geophysics, v. 35, no. 4, pp. 439-460.

Clague, J. J., Atwater, B. F., Wang, K., Wang, Y., and Wong, I., 2000, Penrose conference report--Great Cascadia earthquake tricentennial: GSA Today, v. 10, no. 11, pp. 14-15.

Fluck, P., Hyndman, R. D., and Wang, K., 1997, Three-dimensional dislocation model for great earthquakes of the Cascadia subduction zone: Journal of Geophysical Research, v. 102, no. B9, pp. 20,539-20,550.

Goldfinger, C., 1994, Active deformation of the Cascadia Forearc--Implications for great earthquake potential in Oregon and Washington, Oregon State University, unpublished Ph.D. dissertation, 246 p.

Goldfinger, C., Nelson, C. H., and Johnson, J. E., 2003, Holocene earthquake records from the Cascadia subduction zone and northern San Andreas fault based on precise dating of offshore turbidites: Annual Review of Earth and Planetary Sciences 31, pp. 555-577.

Goldfinger, C., Nelson, C. H., Morey, A. E., Johnson, J. R., Patton, J., Karabanov, E., Gutierrez-Pastor, J., Eriksson, A. T., Gracia, E., Dunhill, G., Enkin, R. J., Dallimore, A., and Vallier, T., 2012, Turbidite event history—Methods and implications for Holocene paleoseismicity of the Cascadia subduction zone: USGS, Professional Paper 1661–F, 170 p., 64 figures, available at http://pubs.usgs.gov/pp/pp1661.

Goldfinger, C., Galer, S., Beeson, J., Hamilton, T., Black, B., Romsos, C., Patton, J., Nelson, C. H., Hausmann, R., and Morey, A., 2016, The importance of site selection, sediment supply, and hydrodynamics: A case study of submarine paleoseismology on the northern Cascadia margin, Washington USA: Marine Geology in Press, DOI: 10.1016/j.margeo.2016.06.008.

Guffanti, M., and Weaver, C. S., 1988, Distribution of late Cenozoic volcanic vents in the Cascade Range--Volcanic arc segmentation and regional tectonic considerations: Journal of Geophysical Research, v. 93, no. B6, pp. 6,513-6,529.

Hughes, J. M., and Carr, M. J., 1980, Segmentation of the Cascade volcanic chain: Geology, v. 8, pp. 15-17.

Hyndman, R. D., and Wang, K., 1995, The rupture zone of Cascadia great earthquakes from current deformation and the thermal regime: Journal of Geophysical Research, v. 100, no. B11, pp. 22,133-22,154.

Idriss, I. M., and Boulanger, R. W., 2008, Soil liquefaction during earthquakes, Earthquake Engineering Research Institute, EERI MNO-12.

Idriss, I. M., and Boulanger, R. W., 2014, CPT and SPT based liquefaction triggering procedures, Department of Civil & Environmental Engineering, College of Engineering, University of California at Davis, Report No. UCD/CGM-14/01.

International Code Council, Inc., 2018, International building code, IBC-2018.

Kelsey, H. M., and Bockheim, J. G., 1994, Coastal landscape evolution as a function of eustasy and surface uplift rate, Cascadia margin, southern Oregon: GSA Bulletin, v. 106, pp. 840-854.

Kelsey, H. M., Witter, R. C., and Hemphill-Haley, E., 2002, Pl-boundary earthquakes and tsunamis of the past 5,500 years, Sixes River estuary, southern Oregon: GSA Bulletin, v. 114, no. 3, pp. 298-314.

Kelsey, H. M., Nelson, A. R., Hemphill-Haley, E., and Witter, R. C., 2005, Tsunami history of an Oregon coastal lake reveals a 4600 yr. record of great earthquakes on the Cascadia subduction zone: GSA Bulletin, v. 117, pp. 1009-1032.

Ma, L., Wells, R. E., Niem, A. R., Niewendorp, C. A., and Madin, I. P., 2009, Preliminary digital geologic compilation map of part of northwestern Oregon, Oregon Department of Geology and Mineral Industries, Open-File Report 09-03.

Mitchell, C. E., Vincent, P., Weldon, R.J. III, and Richards, M. A., 1994, Present-day vertical deformation of the Cascadia margin, Pacific Northwest, United States: Journal of Geophysical Research, v. 99, no. B6, pp. 12,257-12,277.

Nelson, A. R., and Personius, S. F., 1996, Great-earthquake potential in Oregon and Washington--An overview of recent coastal geologic studies and their bearing on segmentation of Holocene ruptures, central Cascadia subduction zone, in Rogers, A.M., Walsh, T. J., Kockelman, W. J., and Priest, G. R., eds., Assessing earthquake hazards and reducing risk in the Pacific Northwest: USGS, Professional Paper 1560, v. 1, pp. 91-114.

Personius, S. F., 1995, Late Quaternary stream incision and uplift in the forearc of the Cascadia subduction zone, western Oregon: Journal of Geophysical Research, v. 100, no. B10, pp. 20,193-20,210.

Petersen, M. D., Moschetti, M. P., Powers, P. M., Mueller, C. S., Haller, K. M., Frankel, A. D., Zeng, Y., Rezaeian, S., Harmsen, S. C., Boyd, O. S., Field, N., Chen, R., Rukstales, K. S., Nico, L., Wheeler, R. L., Williams, R. A., and Olsen, A. H., 2014, Documentation for the 2014 update of the United States national seismic hazard maps: USGS, Open-File Report 2014–1091, 243 p., obtained at: http://dx.doi.org/10.3133/ofr20141091.

Satake, K., Shimazaki, K., Tsuji, Y., and Ueda, K., 1996, Time and size of a giant earthquake in Cascadia inferred from Japanese tsunami records of January 1700: Nature, v.379, pp. 246-249.

Savage, J. C., Svarc, J. L., Prescott, W. H., and Murray, M. H., 2000, Deformation across the forearc of the Cascadia subduction zone at Cape Blanco, Oregon: Journal of Geophysical Research, v. 105, no. B2, p. 3,095-3,102.

State of Oregon, 2019, Oregon structural specialty code (OSSC).

USGS, 2020, Quaternary faults database, accessed 9/28/2021 from USGS website: https://usgs.maps.arcgis.com/apps/webappviewer

USGS, ASCE 7-16 Seismic Design Map Web Service, accessed 9/28/2021 from USGS website: https://earthquake.usgs.gov/ws/designmaps/.

USGS, Unified hazard tool, Dynamic: conterminous U.S. 2014 (v4.1.1), accessed 9/28/2021 from USGS website: https://earthquake.usgs.gov/hazards/interactive/.

USGS, 2014 National seismic hazard maps – Source parameters, lookup by latitude, longitude, accessed 9/28/2021 from USGS website: https://earthquake.usgs.gov/cfusion/hazfaults_2014_search/.

Wang, Y., He, J., Dragert, H., and James, T. S., 2001, Three-dimensional viscoelastic interseismic deformation model for the Cascadia subduction zone: Earth, Planets and Space, v. 53, pp. 295-306.

Weaver, C. S., and Michaelson, C. A., 1985, Seismicity and volcanism in the Pacific Northwest--Evidence for the segmentation of the Juan de Fuca Pl.: Geophysical Research Letters, v. 12, no. 4, pp. 215-218.

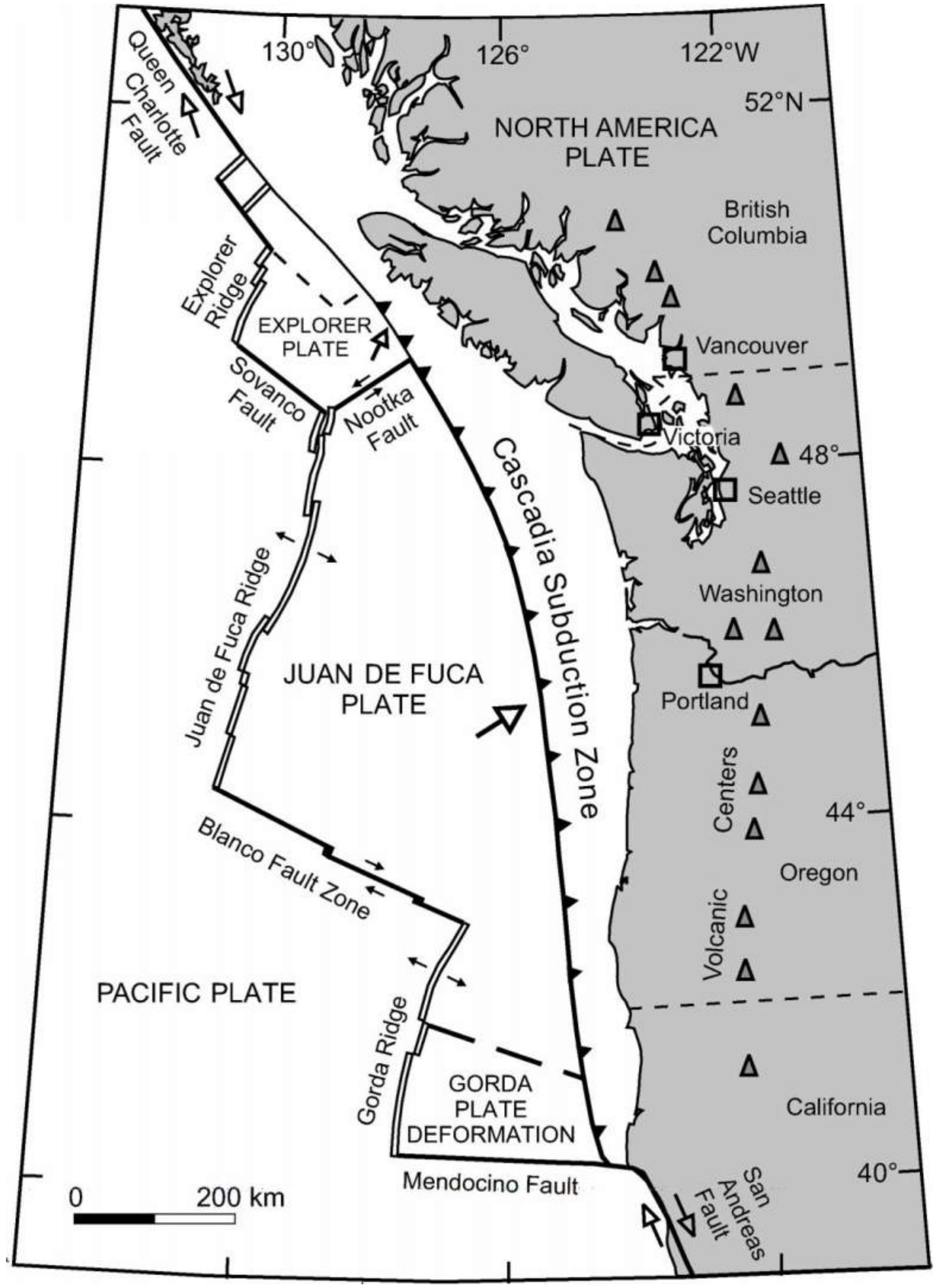
Weaver, C. S., and Shedlock, K. M., 1989, Potential subduction, probable intraplate and known crustal earthquake source areas in the Cascadia Subduction Zone, U.S. Geological Survey, Open-File Report 89-465, pp. 11-26.

Wells, R. E., Haugerud, R., Niem, A., Niem, W., Ma, L., Madin, I., and Evarts, R., 2018, New Geologic Mapping of the Northwestern Willamette Valley, Oregon, and its American Viticultural Areas (AVAs)—A Foundation for Understanding Their Terroir: U.S. Geological Survey Open-File Report 2018-1044, p. 1, doi:10.3133/ofr20181044.

Witter, R. C., 1999, Late Holocene paleoseismicity, tsunamis and relative sea-level changes along the south-central Cascadia subduction zone, southern Oregon: University of Oregon, unpublished PhD dissertation, 178 p.

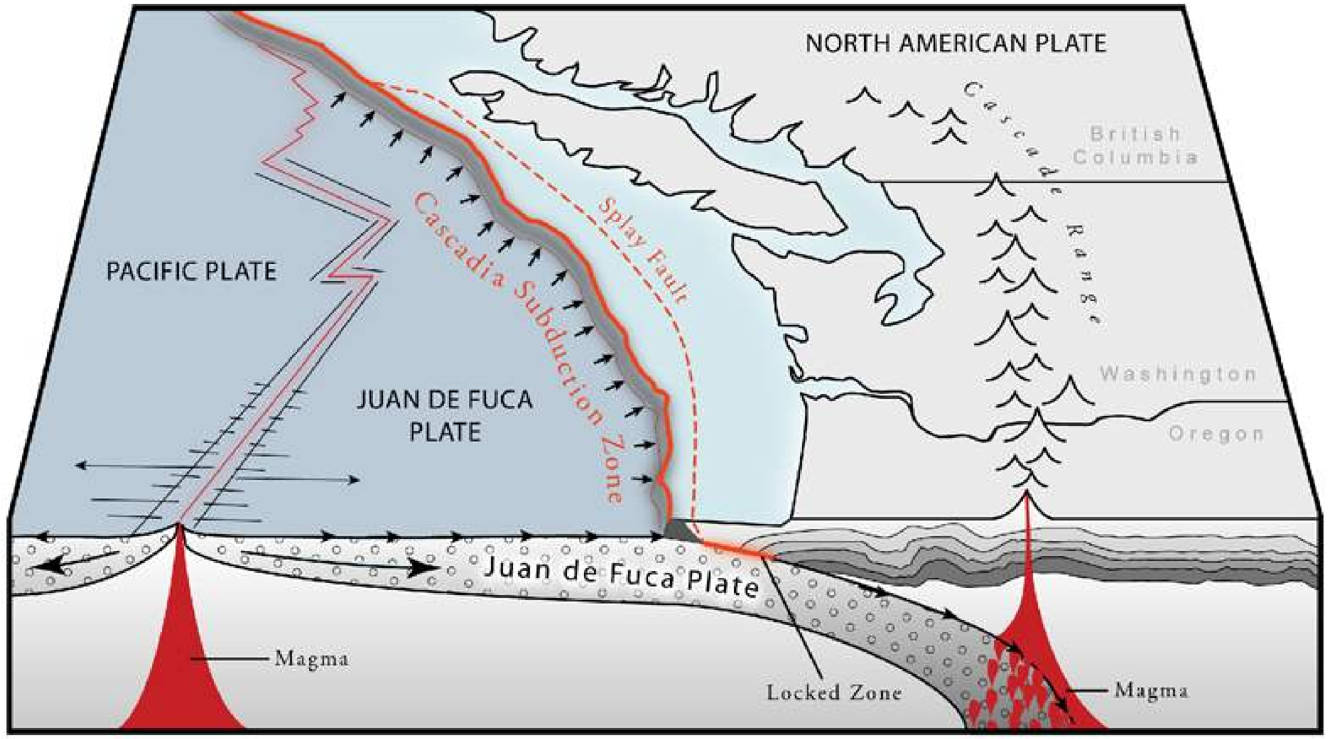
Witter, R. C., Kelsey, H. M., and Hemphill-Haley, E., 2003, Great Cascadia earthquakes and tsunamis of the past 6,700 years, Coquille River estuary, southern coastal Oregon: GSA Bulletin 115, pp. 1,289-1,306.

Wong, I., 2005, Low potential for large intraslab earthquakes in the central Cascadia subduction zone: Bulletin of the Seismological Society of America, v. 95, no. 5.



A) TECTONIC MAP OF PACIFIC NORTHWEST, SHOWING ORIENTATION AND EXTENT OF CASCADIA SUBDUCTION ZONE (MODIFIED FROM DRAGERT AND OTHERS, 1994)

Cascadia Subduction Zone Setting

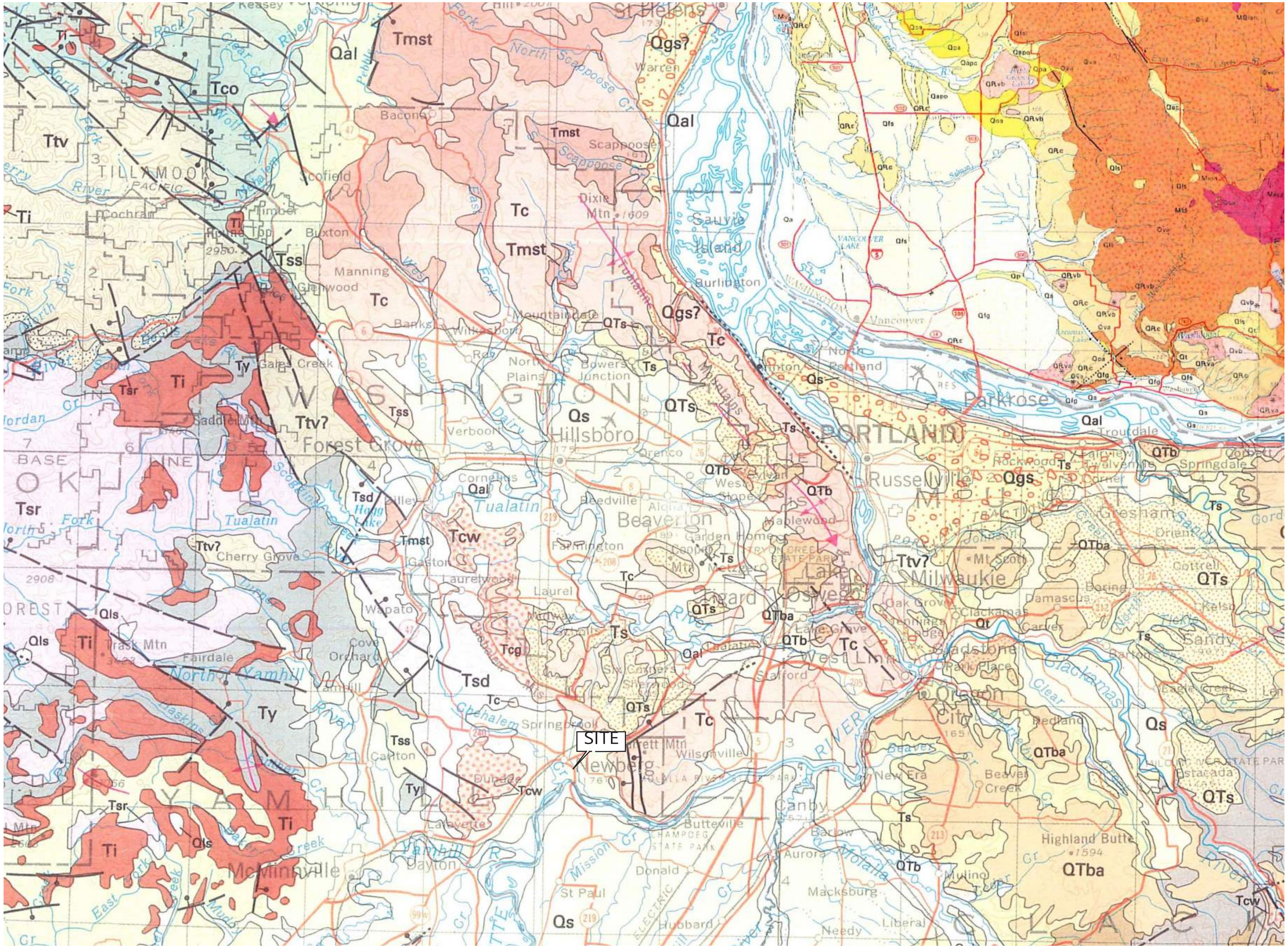


CASCADIA SUBDUCTION ZONE SETTING, TSUNAMI INUNDATION MAPS, OREGON DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRY, 2013

GRI

TECTONIC SETTING
SUMMARY

OCT. 2021 JOB NO. 6519-A FIG. 1B



Contact — Approximately located

Fault — Dashed where inferred; dotted where concealed; queried where doubtful; ball and bar on downthrown side

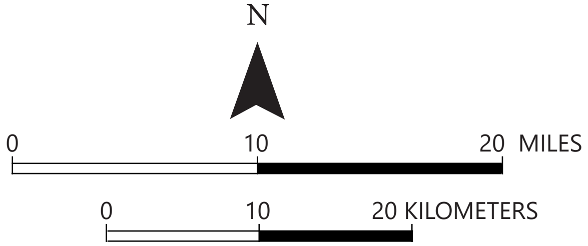
Thrust fault — Dashed where inferred; dotted where concealed; queried where doubtful; sawteeth on upper plate

7 Strike and dip of bed

FROM:

WALSH, T.J., KOROSEC, M.A., PHILLIPS, W.M., LOGAN, R.L., AND SCHASSE, H.W., 1987, GEOLOGIC MAP OF WASHINGTON-SOUTHWEST QUADRANT; 1:250,000: WASHINGTON DIVISION OF GEOLOGY AND EARTH RESOURCES, 6M-34

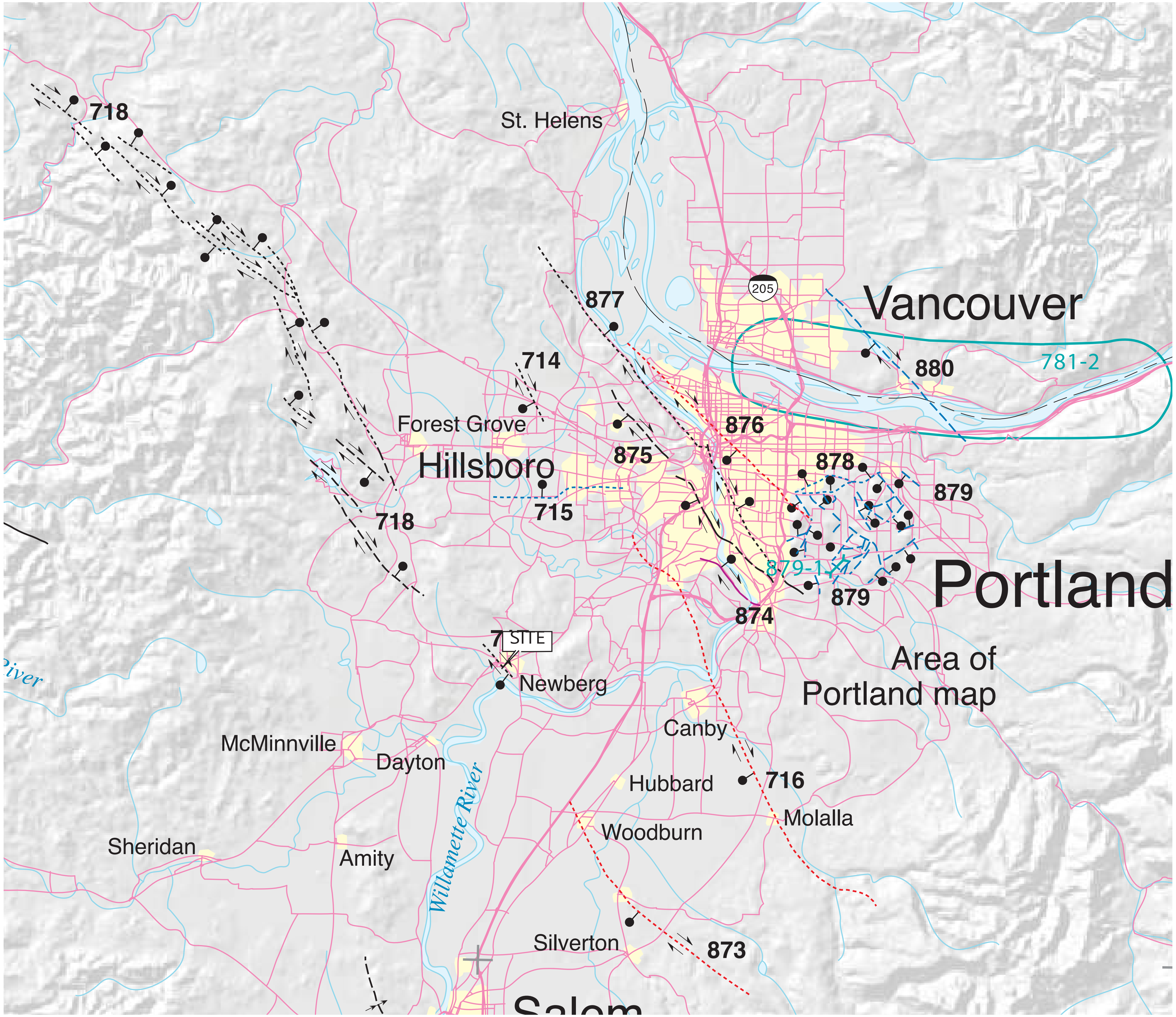
WALKER, G.W., AND MACLEOD, N.S., 1991, GEOLOGIC MAP OF OREGON: U.S. GEOLOGICAL SURVEY



GRI

REGIONAL GEOLOGIC MAP

OCT. 2021 JOB NO. 6519-A FIG. 2B



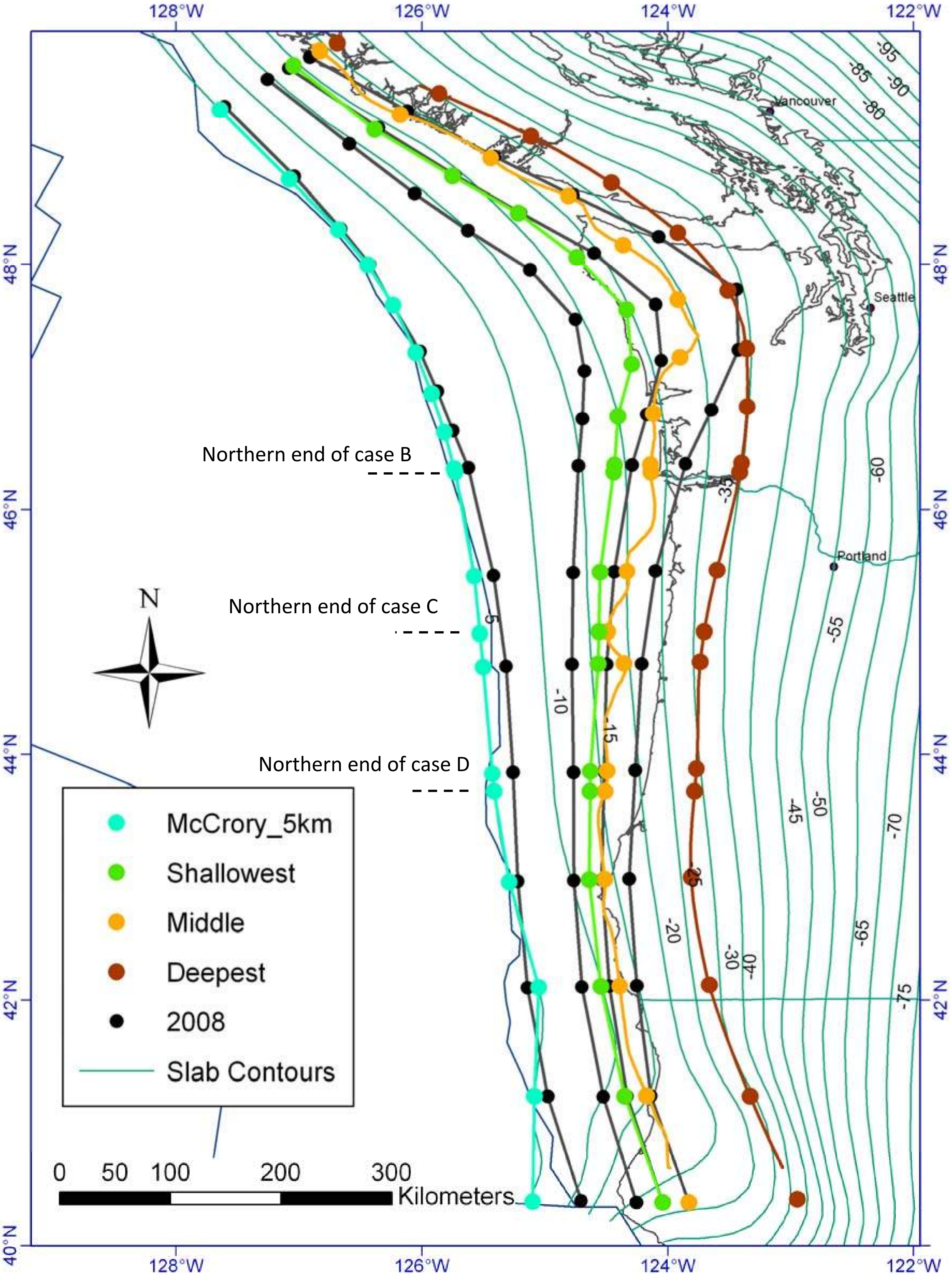
FAULT NUMBER	NAME OF STRUCTURE
714	HELVETIA FAULT
715	BEAVERTON FAULT
716	CANBY-MOLALLA FAULT
717	NEWBERG FAULT
718	GALES CREEK FAULT ZONE
873	MOUNT ANGEL FAULT
874	BOLTON FAULT
875	OATFIELD FAULT
876	EAST BANK FAULT
877	PORTLAND HILLS FAULT
878	GRANT BUTTE FAULT
879	DAMASCUS-TICKLE CREEK FAULT ZONE
880	HAPPY CAMP FAULT

FROM: PERSONIUS, S.F., AND OTHERS, 2003, MAP OF QUATERNARY FAULTS AND FOLDS IN OREGON, USGS OPEN FILE REPORT OFR-03-095.

GRI

LOCAL FAULT MAP

OCT. 2021 JOB NO. 6519-A FIG. 3B



Northern end of case B

Northern end of case C

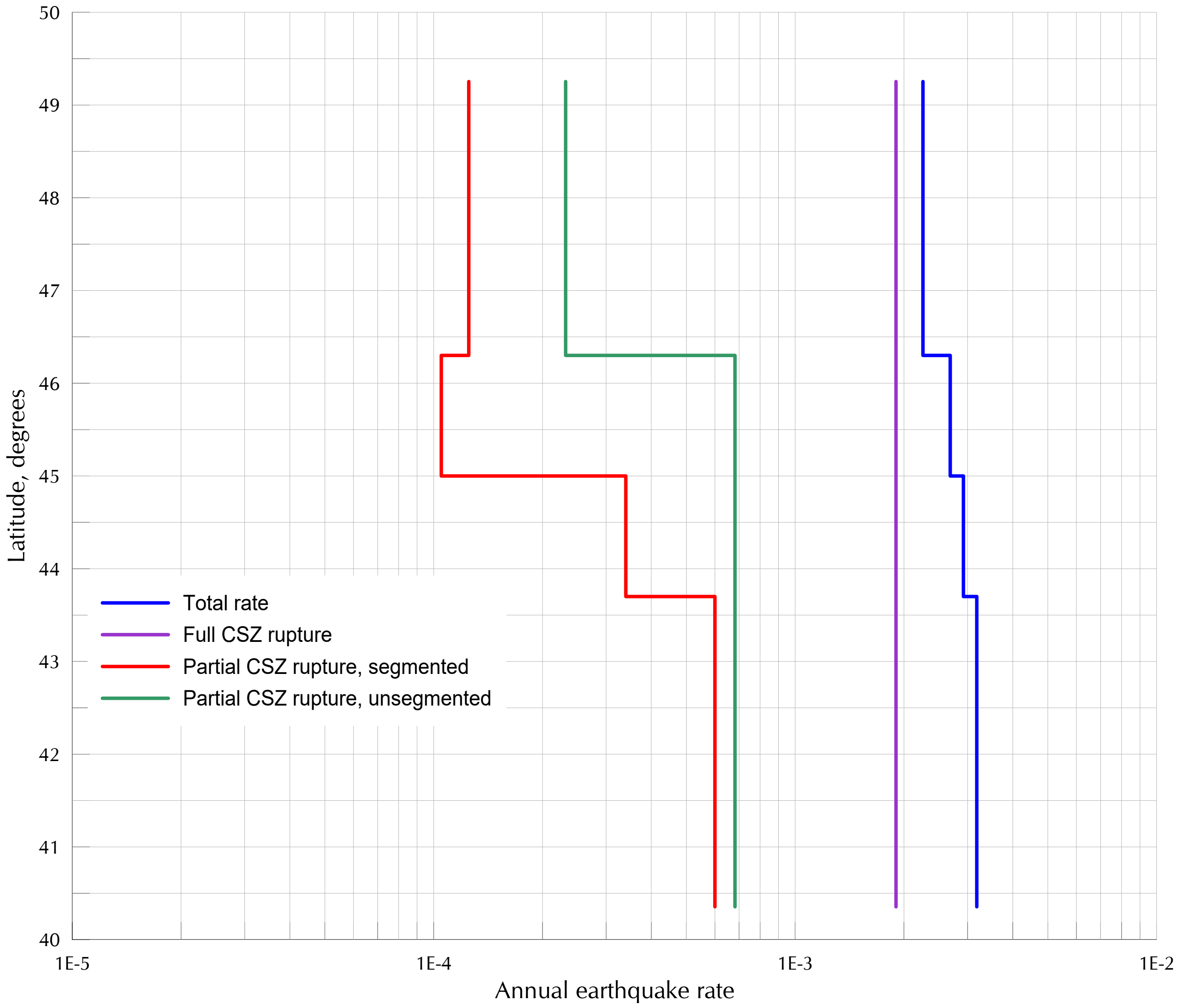
Northern end of case D

REFERENCE:
CHEN ET AL, 2014

GRI

ASSUMED RUPTURE LOCATIONS
(CASCADIA SUBDUCTION ZONE)

OCT. 2021 JOB NO. 6519-A FIG. 4B



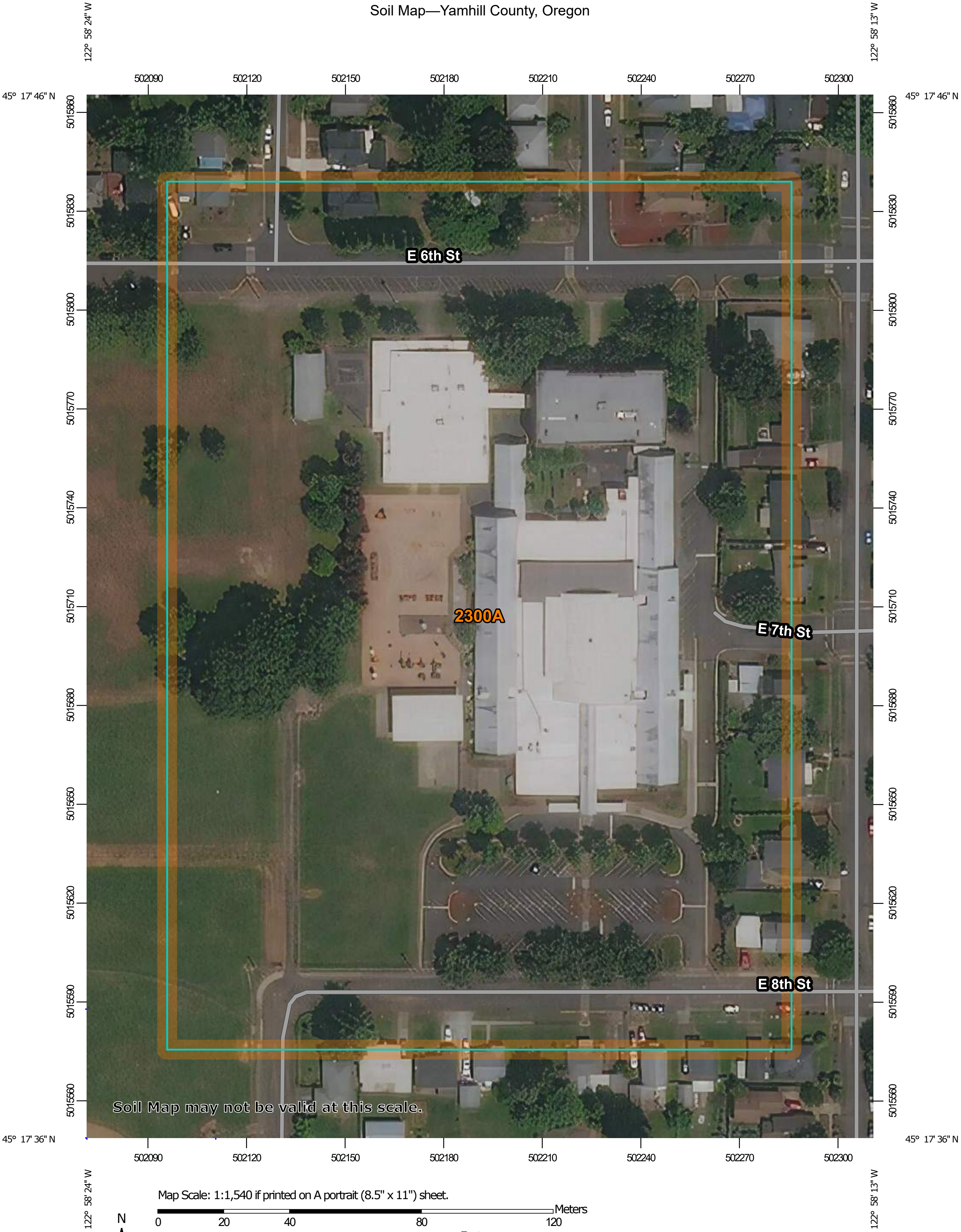
REFERENCE:
CHEN ET AL, 2014
PETERSEN ET AL, 2014

GRI

VARIATION OF EARTHQUAKE RATES
(CASCADIA SUBDUCTION ZONE)

OCT. 2021 JOB NO. 6519-A FIG. 5B

Soil Map—Yamhill County, Oregon



Map Scale: 1:1,540 if printed on A portrait (8.5" x 11") sheet.

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

- ▭ Soil Map Unit Polygons
- 〰 Soil Map Unit Lines
- ■ Soil Map Unit Points

Special Point Features

- Blowout
- ☒ Borrow Pit
- ✱ Clay Spot
- ◇ Closed Depression
- ✕ Gravel Pit
- ∴ Gravelly Spot
- Landfill
- Λ Lava Flow
- Marsh or swamp
- Mine or Quarry
- ◎ Miscellaneous Water
- ○ Perennial Water
- ∨ Rock Outcrop
- + Saline Spot
- ⁘ Sandy Spot
- ⊖ Severely Eroded Spot
- ◇ Sinkhole
- Slide or Slip
- ∅ Sodic Spot
- ≡ Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- △ Other
- Special Line Features

Water Features

- 〰 Streams and Canals

Transportation

- ┼┼┼ Rails
- 〰 Interstate Highways
- 〰 US Routes
- 〰 Major Roads
- 〰 Local Roads

Background

- ▨ Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Yamhill County, Oregon
Survey Area Data: Version 8, Jun 11, 2020

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Aug 19, 2015—Sep 13, 2016

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
2300A	Aloha silt loam, 0 to 3 percent slopes	12.4	100.0%
Totals for Area of Interest		**12.4**	**100.0%**

Yamhill County, Oregon

2300A—Aloha silt loam, 0 to 3 percent slopes

Map Unit Setting

National map unit symbol: 1j8b0
Elevation: 100 to 350 feet
Mean annual precipitation: 40 to 50 inches
Mean annual air temperature: 50 to 54 degrees F
Frost-free period: 165 to 210 days
Farmland classification: Prime farmland if drained

Map Unit Composition

Aloha and similar soils: 96 percent
Minor components: 4 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Aloha

Setting

Landform: Terraces
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Convex
Parent material: Loamy glaciolacustrine deposits

Typical profile

Ap - 0 to 8 inches: silt loam
BA - 8 to 15 inches: silt loam
Bt - 15 to 22 inches: silt loam
Bw1 - 22 to 31 inches: silt loam
Bw2 - 31 to 46 inches: silt loam
Bw3 - 46 to 60 inches: silt loam
C - 60 to 65 inches: very fine sandy loam

Properties and qualities

Slope: 0 to 3 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Somewhat poorly drained
Capacity of the most limiting layer to transmit water (Ksat): Moderately high (0.20 to 0.57 in/hr)
Depth to water table: About 8 to 15 inches
Frequency of flooding: None
Frequency of ponding: None
Available water capacity: Very high (about 12.0 inches)

Interpretive groups

Land capability classification (irrigated): 2w
Land capability classification (nonirrigated): 2w
Hydrologic Soil Group: C/D

Forage suitability group: Somewhat Poorly Drained (G002XY005OR)
Other vegetative classification: Somewhat Poorly Drained (G002XY005OR)
Hydric soil rating: No

Minor Components

Dayton

Percent of map unit: 3 percent
Landform: Terraces
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Concave
Hydric soil rating: Yes

Willamette

Percent of map unit: 1 percent
Landform: Terraces
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Convex
Other vegetative classification: Well drained < 15% Slopes (G002XY002OR)
Hydric soil rating: No

Data Source Information

Soil Survey Area: Yamhill County, Oregon
Survey Area Data: Version 8, Jun 11, 2020

Page left blank for double-sided printing.

Appendix B

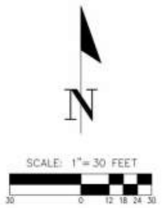
Plans

Basin Map

Page left blank for double-sided printing.

TREE TABLE

POINT #	DBH	TYPE
10273	DECIDUOUS	16
10274	DECIDUOUS	8
10281	DECIDUOUS	18
10679	DECIDUOUS	18
10680	DECIDUOUS	16
10682	DECIDUOUS	16
10683	DECIDUOUS	16
10684	DECIDUOUS	18
10685	DECIDUOUS	22
10733	DECIDUOUS	14
10734	DECIDUOUS	16
11240	DECIDUOUS	28
11241	DECIDUOUS	12
11242	DECIDUOUS	12
11243	DECIDUOUS	8 8
11244	DECIDUOUS	36
11489	DECIDUOUS	46
11490	DECIDUOUS	28
11491	DECIDUOUS	24
11492	DECIDUOUS	30
11493	DECIDUOUS	36
11540	DECIDUOUS	32
11541	DECIDUOUS	30
11542	DECIDUOUS	36
11543	DECIDUOUS	32
11544	DECIDUOUS	32
11546	DECIDUOUS	18
11547	DECIDUOUS	14
11548	DECIDUOUS	14
11549	DECIDUOUS	16
11550	DECIDUOUS	16
11551	DECIDUOUS	16
11552	DECIDUOUS	12
20413	DECIDUOUS	8
20416	DECIDUOUS	10
20417	DECIDUOUS	10
20418	DECIDUOUS	6
20505	DECIDUOUS	36
20506	DECIDUOUS	42
20507	DECIDUOUS	42
20508	DECIDUOUS	36
20509	DECIDUOUS	32
20510	DECIDUOUS	42
20511	DECIDUOUS	50
20512	DECIDUOUS	44
20523	DECIDUOUS	46
20524	DECIDUOUS	48
20525	DECIDUOUS	44
20526	DECIDUOUS	52
20527	DECIDUOUS	60
20918	DECIDUOUS	22
20920	DECIDUOUS	22
20921	DECIDUOUS	28
21131	EVERGREEN	22
21132	EVERGREEN	28
21308	DECIDUOUS	14
21309	DECIDUOUS	20
21310	DECIDUOUS	10
21311	DECIDUOUS	18
21316	DECIDUOUS	6
21318	DECIDUOUS	6
21322	DECIDUOUS	4
21327	EVERGREEN	10
21398	DECIDUOUS	12
21399	DECIDUOUS	14
21400	DECIDUOUS	10
21401	DECIDUOUS	10
21402	DECIDUOUS	10
21403	DECIDUOUS	12
21482	EVERGREEN	18
21483	EVERGREEN	24
21484	EVERGREEN	24
21587	DECIDUOUS	16
21588	DECIDUOUS	44
21589	DECIDUOUS	30
21590	DECIDUOUS	12
21591	DECIDUOUS	18
21716	DECIDUOUS	30
21770	DECIDUOUS	8
22005	DECIDUOUS	22
22158	DECIDUOUS	4
22214	DECIDUOUS	4
22485	DECIDUOUS	16
22570	DECIDUOUS	8
22571	DECIDUOUS	12
22572	DECIDUOUS	16
22573	DECIDUOUS	16



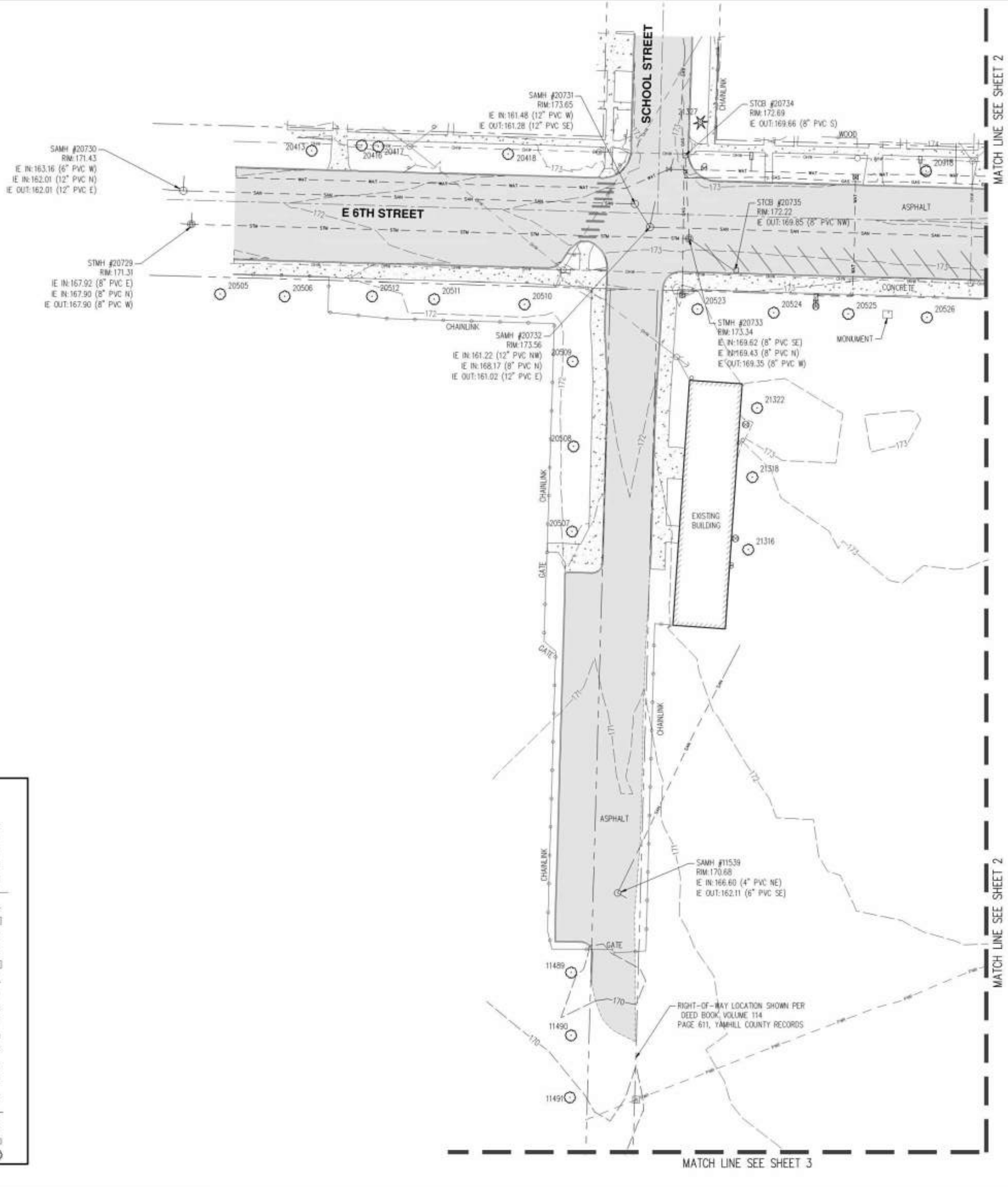
SCALE: 1"= 30 FEET

NOTES:

1. UTILITY LOCATIONS SHOWN ARE PER FIELD LOCATED UTILITY PAINT MARKS & REFERENCE MAPS MADE AVAILABLE BY THE VARIOUS UTILITY PROVIDERS. THE SURVEYOR MAKES NO GUARANTEE THAT THE UNDERGROUND UTILITIES SHOWN COMPRISE ALL SUCH UTILITIES IN THE AREA, EITHER IN SERVICE OR ABANDONED. CONTRACTORS ARE RESPONSIBLE FOR VERIFYING ALL EXISTING CONDITIONS PRIOR TO BEGINNING CONSTRUCTION.
2. FIELD WORK WAS CONDUCTED BETWEEN MAY 11TH AND SEPTEMBER 3RD 2021.
3. VERTICAL DATUM: ELEVATIONS ARE BASED ON NAVD 88.
4. HORIZONTAL DATUM: THE BASIS OF BEARINGS FOR THIS SURVEY IS OREGON STATE PLANE, NORTH ZONE, NAD83, EPOCH 2010.
5. SURVEY IS ONLY VALID WITH SURVEYOR'S STAMP AND SIGNATURE.
6. THIS MAP DOES NOT CONSTITUTE A PROPERTY BOUNDARY SURVEY.
7. CONTOUR INTERVAL IS 1 FOOT.

LEGEND

RIGHT-OF-WAY LINE	—— — — ——— — — ——	STORM SEWER CLEAN OUT	○
BOUNDARY LINE	━━━━━━━━	STORM SEWER CATCH BASIN	□
PROPERTY LINE	————————	STORM SEWER MANHOLE	◎
CENTERLINE	—— - —— - —— - ——	GAS METER	▣
DITCH	— — — —> — — — —> —	GAS VALVE	◫
CURB	════════	GUY WIRE ANCHOR	←
EDGE OF PAVEMENT	- - - - - - - -	POWER POLE	-○-
EASEMENT	- - - - - - - -	POWER VAULT	P
FENCE LINE	—•—•—•—•—	POWER JUNCTION BOX	A
GRAVEL EDGE	········	POWER PEDESTAL	□
POWER LINE	— — — — PWR — — — — PWR —	COMMUNICATIONS VAULT	C
OVERHEAD WIRE	— — — — OHW — — — — OHW —	COMMUNICATIONS JUNCTION BOX	△
COMMUNICATIONS LINE	— — — — COM — — — — COM —	COMMUNICATIONS RISER	○
FIBER OPTIC LINE	— — — — CFO — — — — CFO —	FIRE HYDRANT	⌂
GAS LINE	— — — — GAS — — — — GAS —	WATER BLOWOFF	↑
STORM SEWER LINE	— — — — STM — — — — STM —	WATER METER	▭
SANITARY SEWER LINE	— — — — SAN — — — — SAN —	WATER VALVE	⋈
WATER LINE	— — — — WAT — — — — WAT —	IRRIGATION VALVE	⊗
1-FOOT CONTOUR	— — — — -101 — — — —	SANITARY SEWER CLEAN OUT	○
5-FOOT CONTOUR	— — — — -105 — — — —	SANITARY SEWER MANHOLE	○
		SIGN	—+—
		STREET LIGHT	☆
		MAILBOX	MB
		TREE – DECIDUOUS	⊙



IRONMARK
SURVEYING & ENGINEERING
IRONMARK, LLC
PO BOX 38
SAINT PAUL, OR 97137
P: 503.765.8755
ironmarkllc.com

EDWARDS
ELEMENTARY SCHOOL
NEWBERG OREGON
TAX LOT 4200
TAX MAP 3 2 19 DA

EXISTING
CONDITIONS PLAN
NORTHWEST

DESIGNED BY:
DRAWN BY: BDT
CHECKED BY: JPJ
SCALE: AS NOTED
DATE: 09/08/2021

REGISTERED
PROFESSIONAL
LAND SURVEYOR
OREGON
SEPTEMBER 13, 2016
BRANDT D. THISSELL
84843PLS
RENEWS: 6/30/22

REVISIONS:

JOB NUMBER
1090
SHEET
1

BR|IC
ARCHITECTURE, INC.

NEWBERG
PUBLIC SCHOOLS

kpff
111 SW Fifth Ave., Suite 2600
Portland, OR 97204
O: 503.542.3860
F: 503.274.4681
www.kpff.com

EDWARDS ES
ADDITION AND RENOVATION
715 E. 8TH ST, NEWBERG, OREGON 97132
NEWBERG SCHOOL DISTRICT
T 503-554-5050

NOT FOR CONSTRUCTION

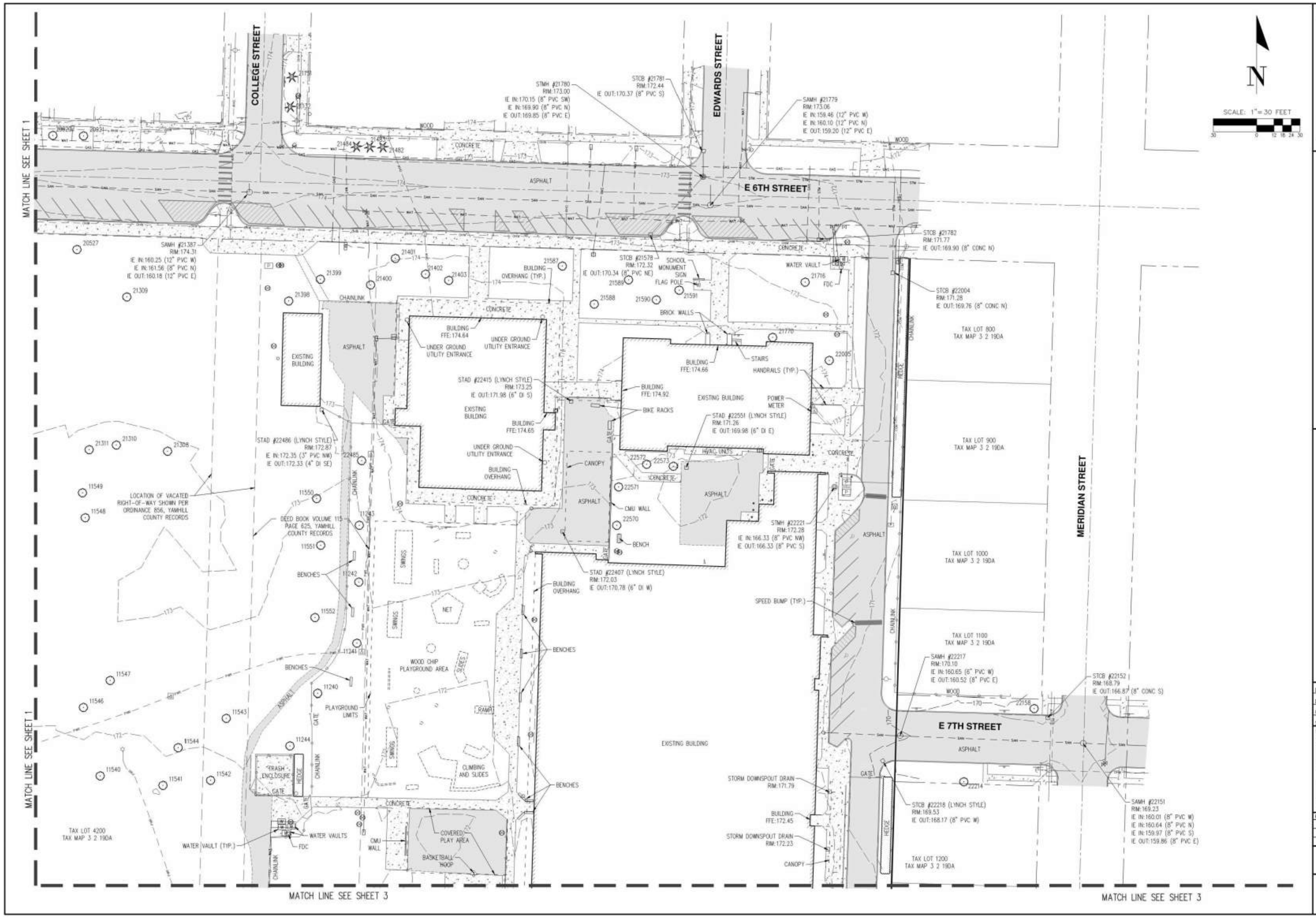
revisions	

phase	LAND USE SUBMITTAL
date	DEC 10, 2021
project	21004

EXISTING CONDITIONS

C1.00

File: N:\c\p\2021\2100142-NSD-Edwards-ES\CAD\PLOT\1142-C1.00-EX-COND.dwg TAB:C1.00
Plotted: 11/29/21 at 11:12am By: AChung



IRONMARK
SURVEYING & ENGINEERING
IRONMARK, LLC
PO BOX 38
SAINT PAUL, OR 97137
P: 503.765.8755
ironmarkllc.com

EDWARDS
ELEMENTARY SCHOOL
NEWBERG OREGON
TAX LOT 4200
TAX MAP 3 2 19 DA

EXISTING
CONDITIONS PLAN
NORTHEAST

DESIGNED BY:
DRAWN BY: BDT
CHECKED BY: JPJ
SCALE: AS NOTED
DATE: 09/08/2021

REGISTERED
PROFESSIONAL
LAND SURVEYOR

OREGON
SEPTEMBER 13, 2016
BRANDT D. THISSELL
84843PLS
RENEWS: 6/30/22

REVISIONS

JOB NUMBER
1090

SHEET
2

BR|IC
ARCHITECTURE, INC

NEWBERG
PUBLIC SCHOOLS

kpff
111 SW Fifth Ave., Suite 2600
Portland, OR 97204
O: 503.542.3860
F: 503.274.4681
www.kpff.com

EDWARDS ES
ADDITION AND RENOVATION

715 E. 8TH ST, NEWBERG, OREGON 97132
NEWBERG SCHOOL DISTRICT
T 503-554-5050

NOT FOR CONSTRUCTION

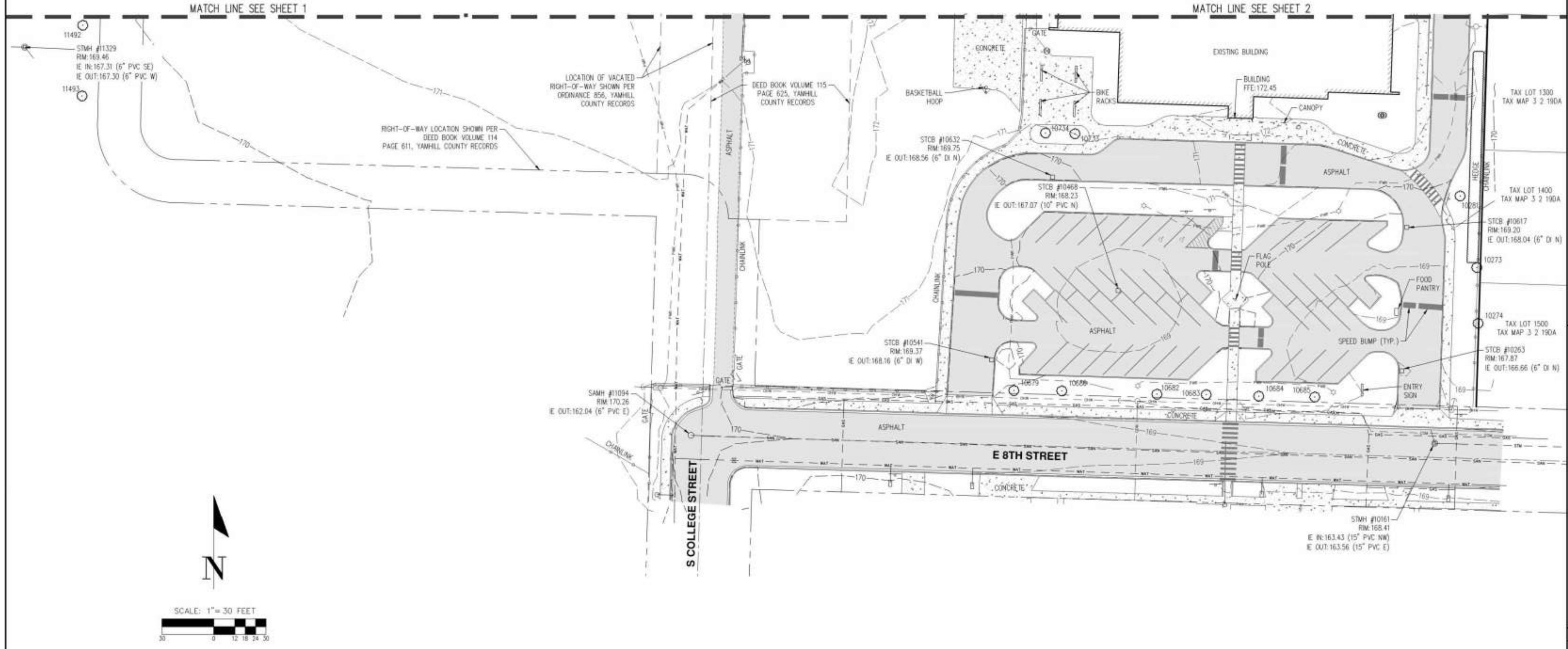
revisions	

phase	LAND USE SUBMITTAL
date	DEC 10, 2021
project	21004

EXISTING CONDITIONS

C1.01

File: N:\cp\2021\2100142-NSD-Edwards-ES\CAD\PLOT\1142-C1.00-EX-COND.dwg TAB:C1.01
Plotted: 11/29/21 at 11:13am By: AChung



IRONMARK
SURVEYING & ENGINEERING

IRONMARK, LLC
PO BOX 38
SAINT PAUL, OR 97137
P: 503.765.8755
ironmarkllc.com

EDWARDS ELEMENTARY SCHOOL
NEWBERG OREGON
TAX LOT 4200
TAX MAP 3 2 19 DA

EXISTING CONDITIONS PLAN SOUTH

DESIGNED BY:	
DRAWN BY:	BDT
CHECKED BY:	JPJ
SCALE:	AS NOTED
DATE: 09/08/2021	

REGISTERED
PROFESSIONAL
LAND SURVEYOR
Brandt Thissell
OREGON
SEPTEMBER 13, 2016
BRANDT D. THISSELL
84843PLS
RENEWS: 6/30/22

REVISIONS

JOB NUMBER
1090
SHEET
3

BR|IC
ARCHITECTURE, INC

NEWBERG
PUBLIC SCHOOLS

kpff
111 SW Fifth Ave., Suite 2600
Portland, OR 97204
O: 503.542.3860
F: 503.274.4681
www.kpff.com

EDWARDS ES
ADDITION AND RENOVATION

715 E. 8TH ST, NEWBERG, OREGON 97132
NEWBERG SCHOOL DISTRICT
T 503-554-5050

NOT FOR CONSTRUCTION

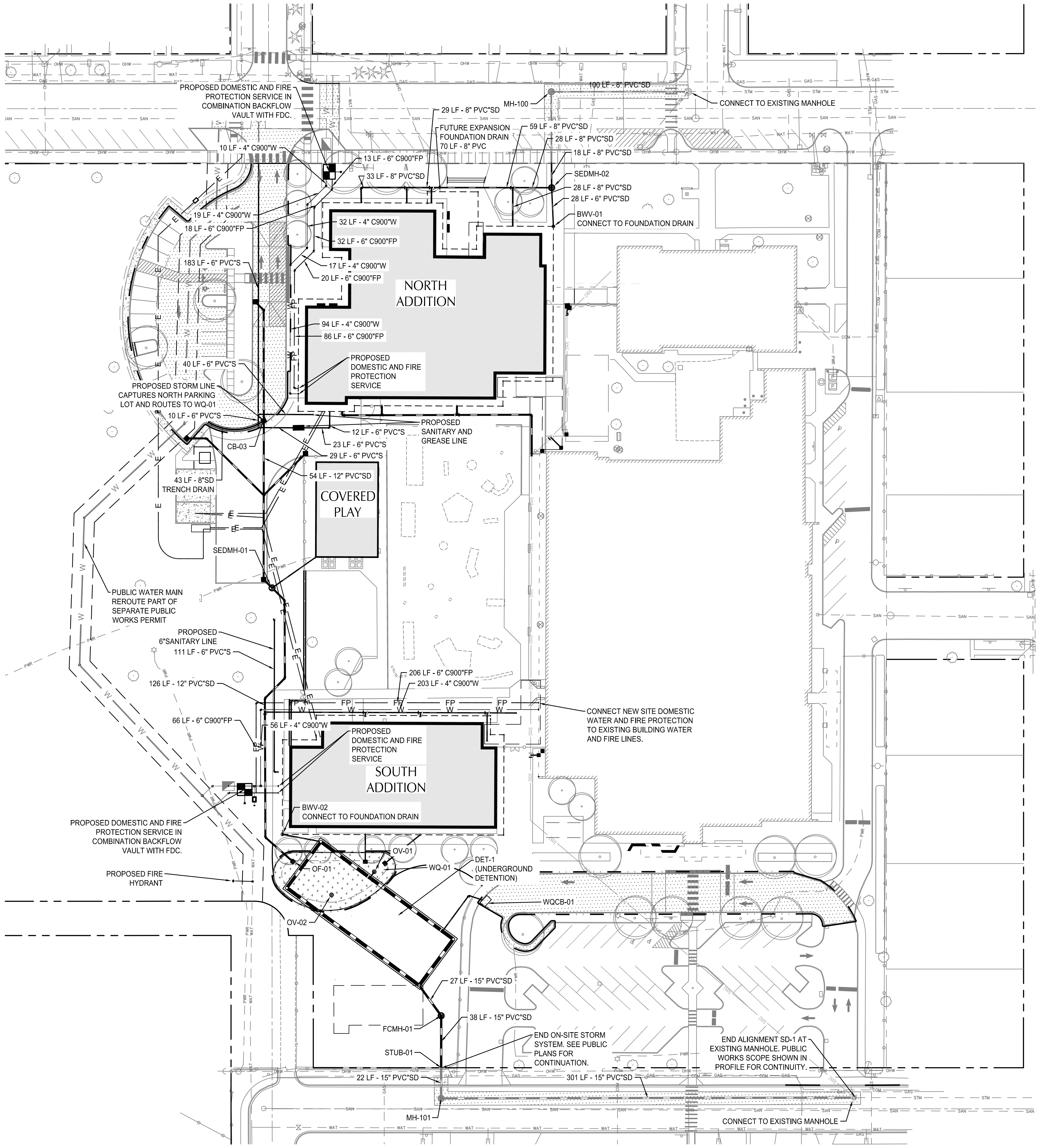
revisions	

phase	LAND USE SUBMITTAL
date	DEC 10, 2021
project	21004

EXISTING CONDITIONS

C1.02

File: N:\c\p\2021\2100142-NSD-Edwards-ES\CAD\PLOT\1142-C1.00-EX-COND.dwg TAB:C1.02
Plotted: 11/29/21 at 11:13am By: AChung



UTILITY LABEL LEGEND

STRUCTURE LABEL

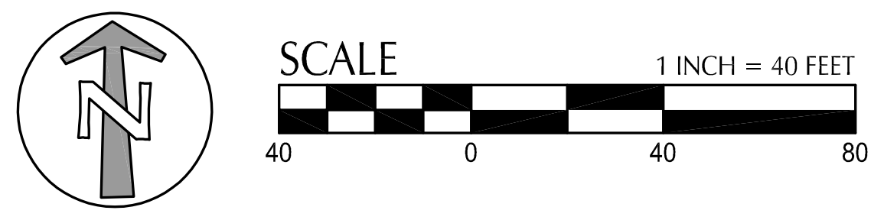
XX XX-XX
UTILITY TYPE (SD=STORM DRAINAGE, S=SANITARY SEWER, W=WATER, FP=FIRE PROTECTION)
STRUCTURE TYPE CALLOUT
ID NUMBER (WHERE APPLICABLE)

PIPE LABEL

XXLF - XX" XX
UTILITY LENGTH
UTILITY SIZE
UTILITY TYPE

STRUCTURE TYPE

CALLOUT	DESCRIPTION
BWV	BACKWATER VALVE
CB	CATCH BASIN
FCMH	FLOW CONTROL MANHOLE
FDC	FIRE DEPARTMENT CONNECTION
FH	FIRE HYDRANT
MH	MANHOLE
OF	OUTFALL
OV	OVERFLOW INLET
SEDMH	SEDIMENTATION MANHOLE
STUB	STUB
TD	TRENCH DRAIN
WQ	WATER QUALITY
WQCB	WATER QUALITY CATCH BASIN



File: N:\c\p\2021\2100142-NSD-Edwards-ES\CAD\PLOT\1142-C5.00-UT-OVERALL.dwg TAB:C5.00
Plotted: 2/16/22 at 2:10pm By: gplenert

BR|IC
ARCHITECTURE, INC.

NEWBERG
PUBLIC SCHOOLS

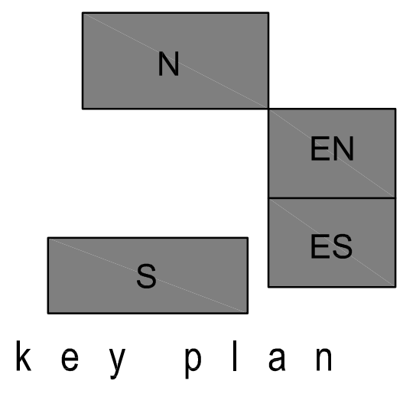
kpff

111 SW Fifth Ave., Suite 2600
Portland, OR 97204
O: 503.542.3860
F: 503.274.4681
www.kpff.com

EDWARDS ES
ADDITION AND RENOVATION

715 E. 8TH ST, NEWBERG, OREGON 97132
NEWBERG SCHOOL DISTRICT
T 503-554-5050

NOT FOR CONSTRUCTION



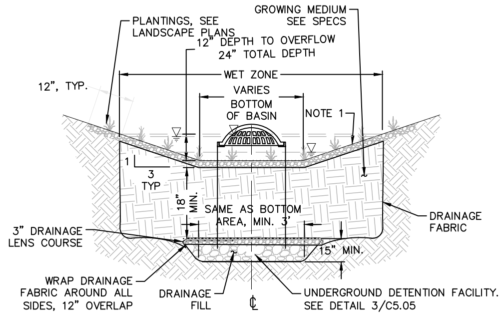
key plan

revisions

phase LAND USE SUBMITTAL
date FEB 17, 2022
project 21004

OVERALL UTILITY PLAN

C5.00

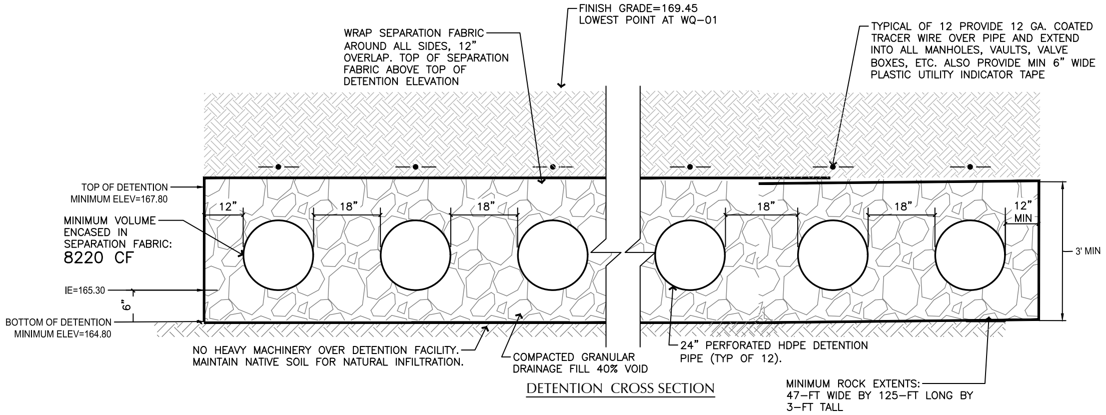


NOTE:

1. INSTALL GEOJUTE PLUS OR COCONUT FIBER MATTING, OR 2" THICK LAYER OF PEA GRAVEL OR OTHER NON-FLOATING MULCH AS APPROVED BY LANDSCAPE ARCHITECT OVER BOTTOM AND SIDE SLOPES OF FACILITY.

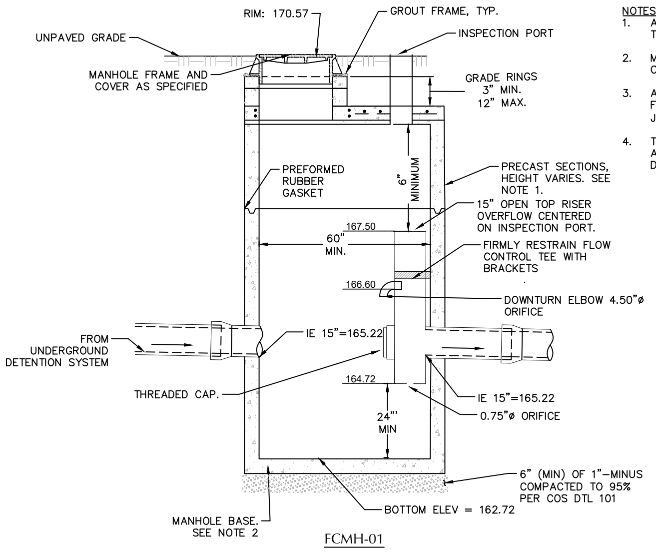
2 VEGETATED FILTRATION BASIN - WQ-01

SCALE: NTS



4 DET-1 PERFORATED PIPE DETENTION SYSTEM SECTION

SCALE: NTS

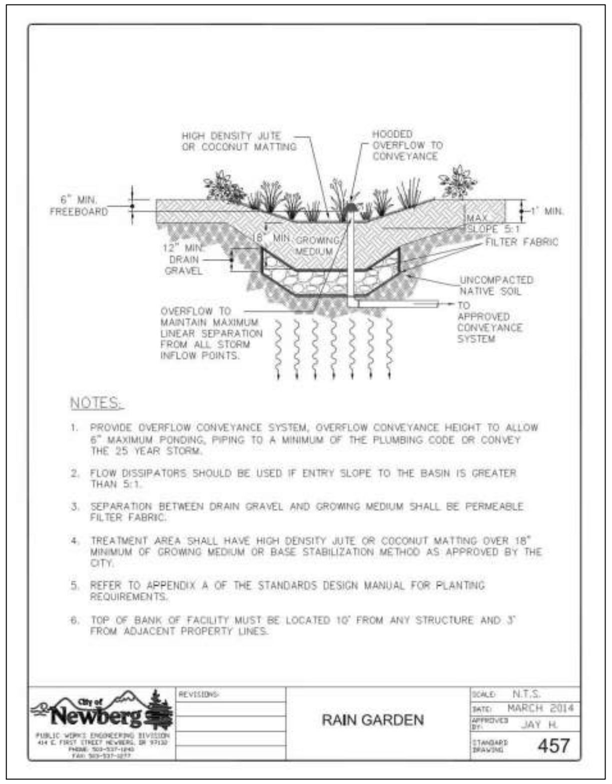


NOTES:

1. ALL PRECAST SECTIONS SHALL CONFORM TO REQUIREMENTS OF ASTM C-478.
2. MANHOLE BASE MAY BE PRECAST OR CAST IN PLACE.
3. ALL CONNECTING PIPES SHALL HAVE FLEXIBLE, GASKETED AND UNRESTRAINED JOINT WITHIN 18" OF MANHOLE VAULT.
4. THE CONTRACTOR SHALL PROVIDE ANTI-BUOYANCY BALLAST IN THE SHOP DRAWINGS.

5 FLOW CONTROL MANHOLE

SCALE: NTS



NOTES:

1. PROVIDE OVERFLOW CONVEYANCE SYSTEM, OVERFLOW CONVEYANCE HEIGHT TO ALLOW 6" MAXIMUM PONDING, PIPING TO A MINIMUM OF THE PLUMBING CODE OR CONVEY THE 25 YEAR STORM.
2. FLOW DISSIPATORS SHOULD BE USED IF ENTRY SLOPE TO THE BASIN IS GREATER THAN 5:1.
3. SEPARATION BETWEEN DRAIN GRAVEL AND GROWING MEDIUM SHALL BE PERMEABLE FILTER FABRIC.
4. TREATMENT AREA SHALL HAVE HIGH DENSITY JUTE OR COCONUT MATTING OVER 18" MINIMUM OF GROWING MEDIUM OR BASE STABILIZATION METHOD AS APPROVED BY THE CITY.
5. REFER TO APPENDIX A OF THE STANDARDS DESIGN MANUAL FOR PLANTING REQUIREMENTS.
6. TOP OF BANK OF FACILITY MUST BE LOCATED 10' FROM ANY STRUCTURE AND 3' FROM ADJACENT PROPERTY LINES.

City of Newberg
PUBLIC WORKS ENGINEERING DIVISION
414 E. FIRST STREET NEWBERG, OR 97132
PHONE: 503-537-1240
FAX: 503-537-1277

REVISIONS:

RAIN GARDEN

SCALE: N.T.S.
DATE: MARCH 2014
APPROVED BY: JAY H.
STANDARD DRAWING 457

1 TYPICAL STORMWATER FACILITY SECTION

SCALE: NTS

BRIC
ARCHITECTURE, INC

NEWBERG
PUBLIC SCHOOLS

kpff
111 SW Fifth Ave., Suite 2600
Portland, OR 97204
O: 503.542.3860
F: 503.274.4681
www.kpff.com

EDWARDS ES
ADDITION AND RENOVATION
715 E. 8TH ST, NEWBERG, OREGON 97132
NEWBERG SCHOOL DISTRICT
T 503-554-5050

NOT FOR CONSTRUCTION

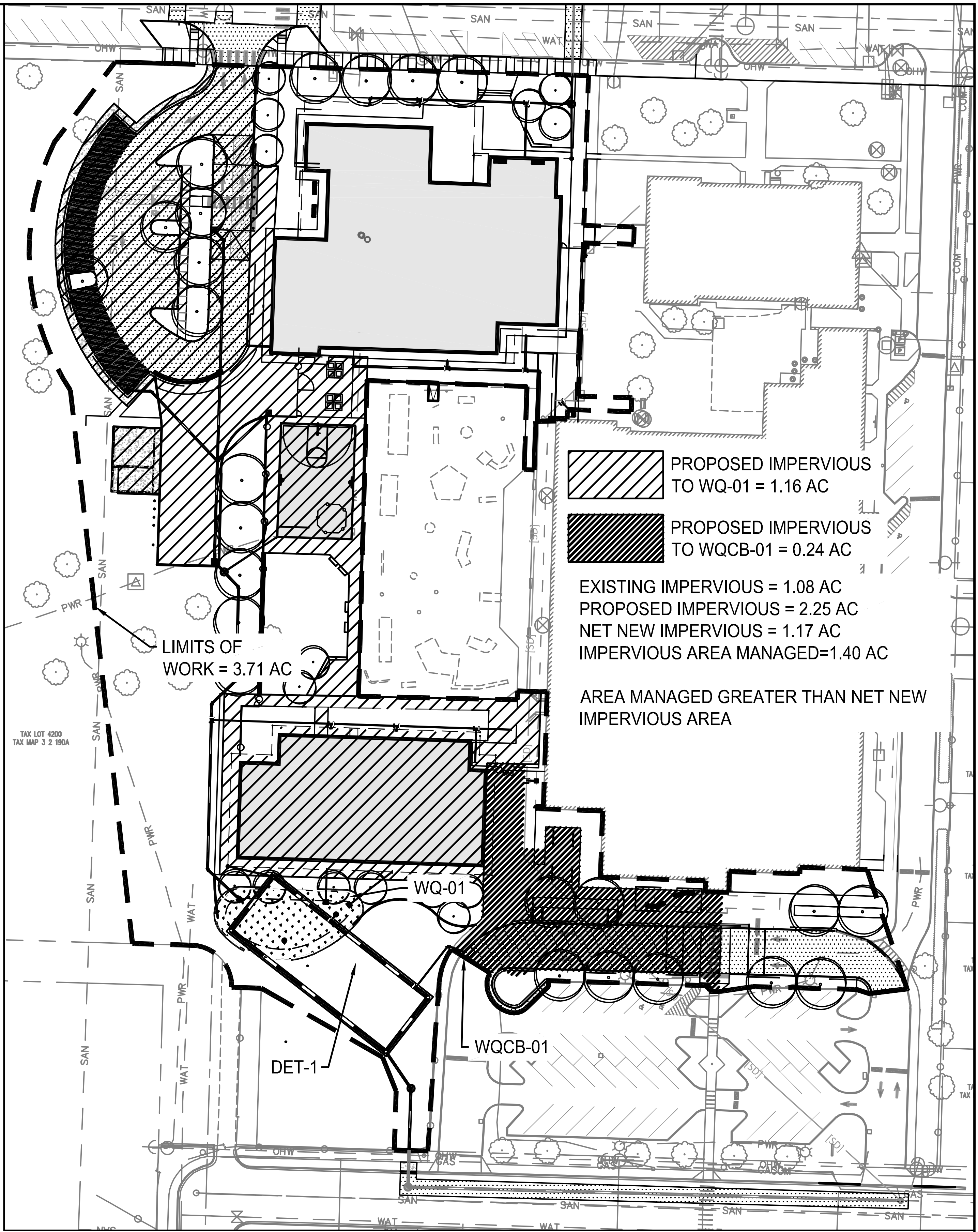
revisions

phase LAND USE SUBMITTAL
date DEC 10, 2021
project 21004

STORM FACILITY DETAILS

C5.05

File: N:\c\p\2021\2100142-NSD-Edwards-ES\CAD\PLOT\1142-C5.05-SD-DETL.dwg TAB:C5.05
Plotted: 11/30/21 at 1:51pm By: AChung



▨ PROPOSED IMPERVIOUS TO WQ-01 = 1.16 AC

▩ PROPOSED IMPERVIOUS TO WQCB-01 = 0.24 AC

EXISTING IMPERVIOUS = 1.08 AC
PROPOSED IMPERVIOUS = 2.25 AC
NET NEW IMPERVIOUS = 1.17 AC
IMPERVIOUS AREA MANAGED=1.40 AC

AREA MANAGED GREATER THAN NET NEW IMPERVIOUS AREA

EDWARDS ELEMENTARY SCHOOL BASIN MAP

SCALE: 1" = 80'

SHEET NO.

EXH-B2

File: N:\c\p\2021\2100142-NSD-Edwards-ES\PROJ-INFO\Planning-Design\03-STORM\2021-BASIN-MAP.dwg TAB: EXH-B2

11X17-EXHIBIT

Plotted: 2/10/22 at 3:19pm By: AChung

Page left blank for double-sided printing.

Appendix C

Hydrologic Summary

Time of Concentration

Page left blank for double-sided printing.

Calculation Spreadsheet:

EDWARDS STORM ASSUMPTIONS AND SUMMARY

Appendix C1

Job Name: EDWARDS ES
KPFF Job #: 2100142
Design Engineer: AC
Check: MJ

ASSUMPTIONS

Total Disturbed Area	3.71	ac
Total Disturbed Impervous Area	2.25	ac
Total Existing Impervious Area	1.08	ac
Net New Impervious Area Requiring Management	1.17	ac
New Impervious Area Managed	1.4	ac

Design Storm	**Storm Event**	**Pre Peak Flow (cfs)**	**Reference**
Water Quality	1	n/a	
1/2 the 2 year	1.25	0.02 cfs	
2 year	2.5	0.102 cfs	City of Newberg Department of Public Works Administrative Rules Design Standards
10 year	3.5	0.226 cfs	
25 year	4	0.299 cfs	
100 year	4.5	0.376 cfs	

Curve Number

Impervious Area =	98	Impervious
Pervious Area =	79	Type D Soil
Pre Developed Area =	79	Type D Soil
Pre-Developed Time of Concentration	76.71	
Minimum Time of Concentration	5 minutes	
Mannings n for PVC Pipe	0.013	

Water Quality Requirements

Facilities shall be able to hold and infiltrate the WQ storm to the maximum extent feasible prior to overflow into storm system.

Water Quantity Requirements

Half 2 year pre peak flow = to Half 2 year post peak flow
2 year pre peak flow equal to 2 year post peak flow
10 year pre peak flow equal to 10 year post peak flow
25 year pre peak flow equal to 25 year post peak flow

Conveyance

Onsite storm system sized to convey 24 hour 25 year design storm. Peak flow calculated using Santa Barbara Urban Hydrograph (SBUH) Method method and pipe size calculated using mannings equation.

Calculation Spreadsheet:

EDWARDS STORM ASSUMPTIONS AND SUMMARY

Appendix C1

Job Name: EDWARDS ES
KPFF Job #: 2100142
Design Engineer: AC
Check: MJ

Water Quality Facility Design

Facility ID	Side Sloped (H:V) or Walled	Storage Volume (cf)	Storage Depth	Freeboard Depth	Total Depth
WQ-01	**3:1**	**2082**	**12"**	**12"**	**24"**

Water Quality Catch Basin Facility Design

Facility ID	WQ Peak Flow (cfs)	WQ Flow per Cartridge (cfs)	# of Cartridges
WQCB-01	0.054	0.033	2

Detention Facility Design

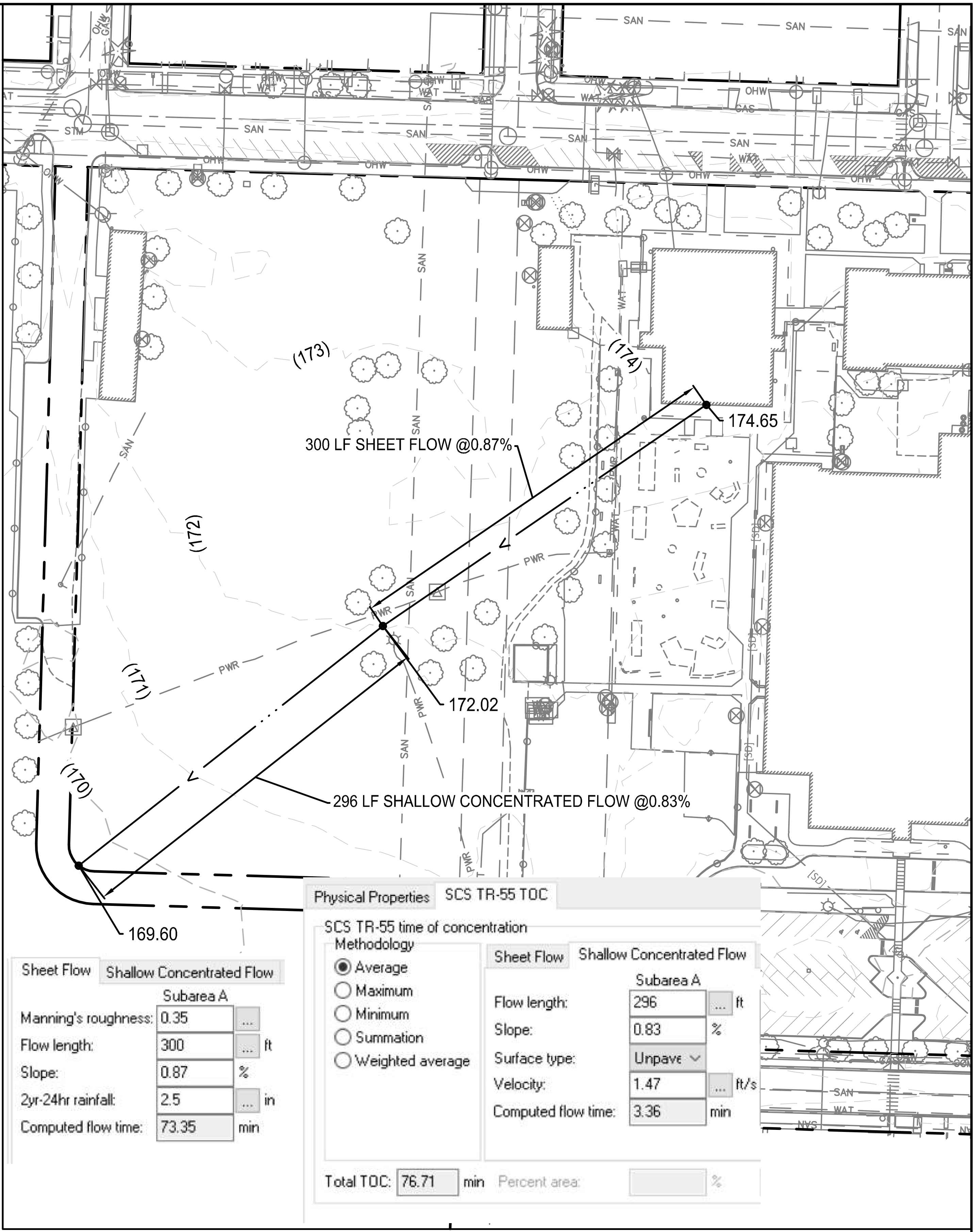
Facility ID	Pipe Diameter	Rock Voids	Required Volume	Total Depth
DET-1	24" (12 Rows)	40%	8220 CF	3ft

Flow Control Manhole

Pipe IE = 15" at 165.22
Lower Orifice = 0.75" at IE 164.72
Upper Orifice = 4.50" at IE 166.6
Overflow = 15" at IE 167.5

Peak Flows

Storm	Undetained Peak Flow	Detained Peak Flow	HGL in DET-1	Less than Pre-Developed?
Half the 2	0.40 cfs	0.02 cfs	1.47 ft	Yes
2	0.85 cfs	0.09 cfs	1.97 ft	Yes
10	1.21 cfs	0.18 cfs	2.09 ft	Yes
25	1.39 cfs	0.26 cfs	2.19 ft	Yes
100	1.57 cfs	0.34 cfs	2.33 ft	Yes



Sheet Flow | Shallow Concentrated Flow

Subarea A

Manning's roughness: 0.35

Flow length: 300 ft

Slope: 0.87 %

2yr-24hr rainfall: 2.5 in

Computed flow time: 73.35 min

SCS TR-55 time of concentration

Methodology

● Average

○ Maximum

○ Minimum

○ Summation

○ Weighted average

Sheet Flow | Shallow Concentrated Flow

Subarea A

Flow length: 296 ft

Slope: 0.83 %

Surface type: Unpave

Velocity: 1.47 ft/s

Computed flow time: 3.36 min

Total TOC: 76.71 min

Percent area: %

EDWARDS ELEMENTARY SCHOOL TC CALCULATION

SCALE: 1" = 100'

SHEET NO.

EXH C2

File: N:\c\p\2021\2100142-NSD-Edwards-ES\PROJ-INFO\Planning-Design\03-STORM\TIME-OF-CONC.dwg TAB:EXH C2

11X17-EXHIBIT

Plotted: 11/30/21 at 10:55am By: AChung

Page left blank for double-sided printing.