REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Dollar Street Middle School
West Linn, Oregon

For
West Linn-Wilsonville School District
October 20, 2020

GeoDesign Project: WLWSchDist-1-01

October 20, 2020

West Linn-Wilsonville School District
22210 SW Stafford Road
Tualatin, OR 97062

Attention: Angela Caffrey

Report of Geotechnical Engineering Services
Dollar Street Middle School
West Linn, Oregon
GeoDesign Project: WLWSchDist-1-01

GeoDesign, Inc. is pleased to submit this report of geotechnical engineering services for the planned Dollar Street middle school site located between Dollar Street and Willamette Falls Drive in West Linn, Oregon. The planned new school will be located near the center of the approximately 20.4-acre site and will have one- and two-story areas with stepped slab-on-grade levels to take advantage of the sloping site. Our services for this project were conducted in accordance with our proposal dated April 30, 2020.

♦ ♦ ♦

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

Sincerely,

GeoDesign, Inc.

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

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SMD:kt
Attachments
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EXECUTIVE SUMMARY

The following is a summary of our findings and recommendations for design and construction of the proposed school. This executive summary is limited to an overview of the project. We recommend that the report be referenced for a more thorough description of the subsurface conditions and geotechnical recommendations for the project.

- No large-scale active slope failures or significant erosion are mapped at the site or were observed during our site observations and explorations. The following report provides specific site grading, wall, and drainage design and construction recommendations for development in consideration of the existing steep slope areas at the site.
- Our stability analysis indicates the proposed grading will have adequate FOS's for global stability. We recommend keying the toe of the fills on the slope above Willamette Falls Drive a minimum of 2 feet below the lowest ground surface. The horizontal base of the keyway should be a minimum of 5 feet wide or 1.5 times the width of the compaction equipment, whichever is greater. A keyway drain consisting of a minimum 2-foot-wide and 2-foot-tall zone of drain rock wrapped in a drainage geotextile with a perforated drainpipe at the bottom should be placed at the inside cut of the keyway to intercept any perched water. Collected water should be routed in non-perforated line(s) to the stormwater system or to a suitable discharge at the base of the slope. The wedge of fill planned to create a 2H:1V slope at the base of the existing steeper slope adjacent to Willamette Falls Drive should be constructed out of crushed rock. We recommend GeoDesign be contacted to review any proposed grading revisions prior to finalizing the plans.
- We understand grading will take place during the wet season between the fall of 2021 and the spring of 2022 and on-site material will be cement amended for placement as structural fill. We recommend installing drainage at the contact of relatively impervious cement-amended fill slopes and overlying topsoil to limit runoff onto the slopes below and erosion of the topsoil. Drainage should consist of angled strip drains on maximum spacings of 30 feet on-center at the contact of the cement-amended fill slopes and overlying topsoil. The strip drains should connect to minimum 2-foot-wide and 2-foot-deep zones of drain rock with perforated collector pipes. The collected water should be routed in non-perforated line(s) to the stormwater system or a suitable discharge at the base of the slope. Water collected from the top of the cement-amended fill should not be connected to the perforated pipe for the keyway drain at the base of the fill.
- We understand the ravine and associated slopes at the east end of the site are mapped as a water resource area. A minimum setback of 25 feet for structural elements (roads, structures, utilities, etc.) from the top of the approximately 10- to 30-foot-high steep slopes is acceptable, provided any structural elements are also located outside a 2H:1V projection from the bottom of the ravine and fills are not placed within the setback area.
- Based on the results of our shallow infiltration tests, the native soil has low infiltration rates. We recommend locating any infiltration facilities below a 5H:1V projection from the base of any slopes and/or walls to limit the potential influence of groundwater on the stability of the slopes and walls. Any stormwater detention facilities within the 5H:1V projection from the base of slopes and/or walls should be lined to prevent infiltration.

- Based on the assumed foundation loads, structures can be supported on shallow foundations bearing on minimum 3-inch-thick granular pads constructed over firm native soil or soil compacted as structural fill as presented in the "Shallow Foundation Recommendations" section.
- The on-site soil can be sensitive to small changes in moisture content and difficult, if not impossible, to adequately compact during wet weather or when the moisture content of the soil is more than a couple of percent above the optimum required for compaction. As discussed in this report, the moisture content of the soil currently is above optimum and drying or cement amendment will be required if used as structural fill.
- The on-site soil will generally provide poor support for construction equipment during the wet construction season or when wet of optimum. Subgrade protection during construction will be important. Granular haul roads and working pads should be employed if earthwork will occur during the wet season or when subgrade is wet of optimum moisture content.

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ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ACP	asphalt concrete pavement
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BGS	below ground surface
CIP	cast-in-place
CPT	cone penetration test
CRBG	Columbia River Basalt Group
CSZ	Cascadia Subduction Zone
DCP	dynamic cone penetrometer
DSHA	deterministic seismic hazard analysis
ESAL	equivalent single-axle load
FOS	factor of safety
fps	feet per second
g	gravitational acceleration (32.2 feet/second^2)
GPS	global positioning system
H:V	horizontal to vertical
km	kilometers
MCE	maximum considered earthquake
MCE_R	risk-targeted maximum considered earthquake
mm/yr	millimeters per year
M_w	moment magnitude
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Standard Specifications for Construction (2018)
OWRD	Oregon Water Resources Department
pcf	pounds per cubic foot
PG	performance grade
psf	pounds per square foot
PSHA	probabilistic seismic hazard analysis
psi	pounds per square inch
SLIDO	Statewide Landslide Information Database for Oregon
SOSSC	State of Oregon Structural Specialty Code
SPT	standard penetration test
USGS	U.S. Geological Survey
Vs_{30}	shear wave velocity for the upper 100 feet (30 meters)

1.0 INTRODUCTION

GeoDesign, Inc. is pleased to present this geotechnical engineering report for the planned Dollar Street middle school to be located between Dollar Street and Willamette Falls Drive in West Linn, Oregon. The planned project includes a new approximately 114,000-square-foot, two-story middle school building with stepped slab-on-grade levels and associated parking lots, access driveways, infrastructure, an athletic field, a track, outdoor play areas, and associated retaining walls. Parent drop-off and visitor access may occur at the main entrance facing the west parking area. Bus drop-off and additional parking for staff may occur on the east side of the site. Figure 1 shows the site location relative to existing topographic and physical features. The existing and proposed topography and exploration locations are shown on Figure 2. Only the existing topographic contours are shown on Figure 3. Figure 4 shows only the proposed topographic contours in grading areas with the surrounding existing topography to show the proposed final topography for the site.

Public improvements include a new extension of Brandon Place through the west portion of the site, as well as off-site half-street improvements to 2,050 feet of the Dollar Street frontage and full-width improvements for 1,750 feet of the Willamette Falls Drive frontage. Pavement recommendations for the public roads will be provided in a separate report. The planned Willamette Falls Drive improvements may include pervious AC for the bicycle tracks.

Structural loads for the building were not available at the time of this report, but maximum column and wall loads for the building are anticipated to be less than 200 kips and 8 kips per foot, respectively. The proposed grading consists of variable cuts and fills. The largest proposed fills range up to approximately 25 feet at several locations southwest of the building and parking lot. The largest cuts generally range up to approximately 10 feet with a larger cut ranging up to approximately 20 feet for the north end of the planned building. A wedge of fill ranging up to approximately 7 feet high is also planned at the toe of the existing steeper slope adjacent to Willamette Falls Drive.

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

2.0 SCOPE OF SERVICES

The purpose of our geotechnical engineering services was to characterize site subsurface conditions and provide geotechnical engineering recommendations for use in design and construction of the proposed school. Our scope of services included the following:

- Reviewed readily available geotechnical reports, geologic mapping, aerial photographs, and topographic data for the site and vicinity.
- Coordinated and managed the field evaluation, including utility locates, site access, and scheduling subcontractors and GeoDesign field staff.

- Conducted the following subsurface investigation:
  - Cleared paths to the exploration locations as needed using a track-mounted excavator.
  - Drilled six borings to depths between 15.5 and 70.6 feet BGS. The exploration logs are presented in Appendix A.
  - Installed a vibrating wire piezometer in one of the borings and recorded groundwater measurements using a data logger.
  - Excavated three test pits to conduct shallow infiltration testing at locations away from the steep slopes at the site. The exploration logs are presented in Appendix A.
  - Advanced three CPT probes to practical refusal at depths between 69.9 and 90.2 feet BGS. Shear wave velocity testing was conducted in one the CPT probes. The CPT probe test results are presented in Appendix B.
  - Performed three DCP tests to evaluate the pavement subgrade modulus on the site.
- Collected disturbed and undisturbed soil samples from the borings and test pits for laboratory testing at select depths and maintained a log of soil and groundwater conditions encountered in each exploration.
- Completed the following laboratory tests on select soil samples:
  - Forty-six moisture content determinations in general accordance with ASTM D2216
  - One Atterberg limits test in general accordance with ASTM D4318
  - Ten particle-size analyses in general accordance with ASTM D1140
  - Two consolidation tests in accordance with ASTM D2435
  - Two sets of direct shear tests in general accordance with ASTM D3080
- Provided recommendations for site preparation, grading and drainage, stripping depths, fill type for imported material, compaction criteria, trench excavation and backfill, use of on-site soil, and wet/dry weather earthwork.
- Created slope cross sections and evaluated the global stability of critical proposed walls and slopes.
- Provided recommendations for proposed retaining walls, including lateral earth pressures, backfill, compaction, and drainage.
- Provided recommendations for the preferred foundation type, including allowable capacity, settlement estimates, and lateral resistance parameters.
- Provided recommendations for preparation of floor slab subgrades.
- Provided recommendations for managing identified groundwater conditions that may affect the performance of structures.
- Evaluated seismic hazards, including liquefaction, lateral spreading, slope stability, and ground rupture.
- Provided recommendations for construction of on-site non-public street AC pavement sections, including subbase, base course, and AC pavement thickness.
- Provided the results of our field infiltration testing and recommendations for on-site stormwater disposal.
- Provided a site-specific seismic hazard study as required by the SOSSC for essential occupancy classified buildings, including site-specific response spectra.
- Prepared this geotechnical report summarizing our explorations, laboratory testing, and recommendations.

3.0 BACKGROUND

GeoDesign previously explored the site for a proposed residential development (GeoDesign, 2006). Our prior explorations at the site included 3 borings drilled to depths between 41.5 and 56.5 feet BGS and 17 test pits excavated to depths between 10.5 and 16.5 feet BGS. The prior exploration and laboratory test results are presented in Appendix C. We also have the results of a boring drilled in Willamette Falls Drive near the northwest end of the site at the east abutment for the bridge over the Tualatin River. The foundation data sheet showing the results of the boring in Willamette Falls Drive is presented in Appendix D.

A former residence and associated outbuildings were located off Dollar Street in the central portion of the site. Based on historical aerial photographs from Google Earth, the residence was demolished in 2009 or 2010. A groundwater monitoring well was previously identified near the former residence but was not observed during our recent explorations. We understand another former residence was also located in the northwest portion of the site and was demolished prior to 1994 based on the oldest available Google Earth aerial photograph.

4.0 SITE CONDITIONS

4.1 GEOLOGIC CONDITIONS

The area of this study is located near the central portion of the Willamette Valley physiographic province, which extends from approximately Cottage Grove, Oregon, in the south to the Columbia River in the north. The Willamette Valley is generally an elongated alluvial/fluvial plain bordered on the west by the Coast Range and on the east by the Cascade Mountains. The site is located along the Tualatin River, northeast of the confluence with the Willamette River (as shown on Figure 1). Basement rocks in the Northern Willamette Valley area generally consist of Eocene Age marine sedimentary rocks that are overlain by basalt flows of the Miocene Age CRBG (Schlicker and Finlayson, 1979). The basement rocks are generally overlain by Pliocene to Holocene sedimentary deposits consisting of glacial outwash and river alluvium.

The site location is immediately underlain by the Willamette Silt Formation, a generally fine-grained, lacustrine unit consisting of unconsolidated fine sand, sandy silt, and clay that was deposited by glacial floods in the late Pleistocene Age (Schlicker and Finlayson, 1979). The Willamette Silt is underlain by older alluvium deposits. Based on a review of the explorations completed for the bridge crossing the Tualatin River near the northwest corner of the site, the depth of the flood deposits and underlying sediments is estimated to range from 70 to 160 feet BGS at the site. The older alluvial deposits are underlain by bedrock of the CRBG. The basalt flows generally consist of gray to black, dense, fine-grained basalt that is locally deeply weathered (Schlicker and Finlayson, 1979).

4.2 SURFACE CONDITIONS

The site is located along the northeast side of the Tualatin River in West Linn, Oregon. Elevations range from approximately 114 feet near northwest end of the site to 208 feet near the east corner of the site. The property is bound by Willamette Falls Drive to the southwest, Dollar Street to the north and northeast, and the Willamette Cove development to the southeast.

The center of the property is covered by a densely wooded, overgrown tree plantation of mature conifers. The remainder of the site consists of overgrown farmland or orchards, with areas of grassy fields, brambles, shrubs, and larger deciduous trees.

A majority of the property slopes gently to the southwest. However, the slopes become moderate to very steep above Willamette Falls Drive (as shown on Figure 2). The road cut for Willamette Falls Drive has formed a steep to very steep slope along the south property boundary. The slope ranges from approximately 30 percent (northwest portion of the site) to greater than 100 percent (southeast portion of the site). SLIDO does not map any landslides at the site. The steeper slopes are mapped as having moderate to high susceptibility to shallow landslides (less than 15 feet deep) and are mapped as a low susceptibility to deep landslides (greater than 15 feet deep).

Signs of recent slope failures or erosion were not observed for the steeper slope above Willamette Falls Drive (as shown on Figure 2). Conifer trees with curved trunks were observed on the steeper slope, suggesting past shallow slope creep and/or erosion. The absence of trees in an area of the upper portion of the larger concave slope near the middle of the site could be indicative of past shallow slope movement. A catch basin is located in the drainage ditch along Willamette Falls Drive below the larger concave slope area.

A ravine with steep slopes overgrown with brush, trees, and blackberries cuts through the east corner of the property and transitions to a wooded slope adjacent to the residential development along the southeast edge of the property (as shown on Figure 2). The ravine and toe of the southeast slope coincide with the approximate location of a utility easement. A stormwater pipe outlets near the upper north end of the ravine and there is a catch basin in the bottom of the ravine roughly 390 feet southwest and downslope of the outlet. Water was not observed in the drainage during our site visit and we understand the bottom of the ravine is not classified as wetlands. Evidence of recent slope failures or erosion were not observed for the slopes. Bent conifers, suggesting past shallow slope creep and/or erosion, were observed on the slopes.

4.3 SUBSURFACE CONDITIONS

4.3.1 General

We explored subsurface conditions by drilling six borings (B-1 through B-6) to depths between 15.5 and 70.6 feet BGS, excavating three test pits (TP-1 through TP-3) to a depth of 11 feet BGS, and advancing three CPT probes (CPT-1 through CPT-3) to refusal at depths between 69.9 and 90.2 feet BGS. Our prior explorations in 2006 at the site included 3 borings drilled to depths between 41.5 and 56.6 feet BGS and excavating 17 test pits to depths between 10.5 and 16.5 feet BGS. The approximate exploration locations and cross section locations are shown on Figures 2 through 4. Typical subsurface profiles for cross sections A-A' and B-B' are shown on Figures 5 and 6, respectively. The recent boring and test pit logs and results of the laboratory testing completed at the site by GeoDesign are presented in Appendix A. CPT logs are presented in Appendix B. Our prior exploration logs and laboratory testing results are presented in Appendix C. A foundation data sheet showing the results of the boring by others in Willamette Falls Drive at the southeast end of the bridge over the Tualatin River is presented in Appendix D.

4.3.2 Topsoil, Root Zone, and Fill

All of the test pits encountered a 2- to 6-inch-thick organic root zone and a topsoil zone that ranged from 6 to 18 inches, with an average of approximately 12 inches thick at the ground surface. The topsoil generally consists of medium stiff, dark brown silt with an increased fraction of organics. Similar material interpreted as fill was also encountered in our prior test pit TP-10 (2006) to a depth of 2.5 feet BGS. Approximately 2 feet of medium dense gravel fill was encountered in borings B-4 and B-5 at the base of the slope along Willamette Falls Drive and 13.5 feet of soft to medium stiff silt fill was encountered in boring B-6, which is on the southwest side of Willamette Falls Drive.

4.3.3 Silt

Silt was encountered under the topsoil zone in most of the explorations at the site. The silt is generally medium stiff, contains variable amounts of fine sand, and sometimes contains variable amounts of clay. The sand content generally increases with depth. The silt was encountered to depths between approximately 3.5 and approximately 14 feet BGS in all of the recent and prior explorations, except TP-3 and prior test pits TP-1 (2006) and TP-4 (2006) where silt was encountered to the depths explored ranging between 11 and 12.5 feet BGS. Tree roots of 0.5 to 1 inch in diameter were encountered in most of the test pits throughout the silt layer. Atterberg limits testing indicates the silt varies from having non-plastic properties to exhibiting low plasticity. Laboratory testing from our recent explorations indicates the moisture content of the silt ranges from 20 to 37 percent.

4.3.4 Sand

Loose to medium dense sand generally underlies the near-surface silt. The sand becomes medium dense with increasing depth and the silt content of the sand ranges from silty to trace silt and generally decreases with depth. Layers and/or lenses of silt with variable sand content are also embedded in the sand. Laboratory testing from our recent explorations indicates the moisture content of the sand ranges from 11 to 31 percent.

4.3.5 Gravel and Sand

Medium dense to very dense gravel with sand and variable silt content was encountered below the silt and sand. The gravel generally transitions to dense to very dense very quickly with depth. In our recent explorations the gravel was encountered at a depth of 64 feet BGS in B-1, 50.3 feet BGS in B-3, 22 feet BGS in B-4, 13.5 feet BGS in B-5, and 16 feet BGS in B-6. Laboratory testing from our recent explorations indicates the moisture content of the gravel ranges from 9 to 23 percent.

Dense sand to gravelly sand was encountered in CPT-1 from 50 feet BGS to refusal presumably on very dense gravel and/or sand at 76.9 feet BGS. CPT-2 encountered refusal presumably on very dense gravel and/or sand at 69.9 feet BGS and CPT-3 encountered refusal at a depth of 90.2 feet BGS, although the tip resistance was not high enough to confirm sand and/or gravel at the refusal depth.

4.3.6 Groundwater

No groundwater or caving was observed in any of the test pit explorations at the site. Groundwater was observed at a depth of 20 feet BGS (approximate elevation 96 feet) during drilling for boring B-4, which was at the toe of the southwest slope along Willamette Falls Drive. The groundwater observed in B-4 may have been perched since free water was not observed after the completion of drilling. Groundwater was observed at a depth of 70 feet BGS (approximately elevation 102 feet) in boring B-1 and was deeper than the vibrating wire piezometer depth of 68 feet BGS in boring B-1 on July 20, 2020. Based on pore water pressure measurements, groundwater was at 57 and 67.7 feet BGS (approximate elevations of 143 and 116 feet) in CPT-1 and CPT-2, respectively. Groundwater depth was not estimated using data from CPT-3 due to slow dissipation of the pore water pressure. Wet areas or groundwater seeps were not observed on the ground surface at the time of our explorations, and other than the groundwater observations mentioned above, groundwater was not observed to the depths explored in any of our other explorations.

Groundwater likely fluctuates with seasonal conditions and the water level in the Tualatin River located nearby to the south and west of the site. Perched groundwater may be encountered at shallower depths, particularly during the wet season.

4.4 INFILTRATION TESTING

Infiltration testing was completed to assist in the evaluation of potential stormwater infiltration facilities for the project. We conducted infiltration testing at shallow depths in test pits TP-1, TP-2, and TP-3. Infiltration testing was performed using the encased falling head method using a 6-inch-inside diameter casing and approximately 1 foot to 3 feet of water head.

Laboratory testing was performed on select soil samples to determine the percent fines content at the infiltration test depths. Table 1 summarizes the unfactored infiltration test results and the amount of fines present at the depth of the infiltration tests.

Table 1. Infiltration Test Results

Location	Depth (feet BGS)	Soil Type at Test Depth	Measured Infiltration Rate (inches per hour)	Fines Content[1] (percent)
TP-1	5	Sandy Silt	0.4	59
TP-2	6	Sandy Silt	Negligible	70
TP-3	8	Silt, minor sand	1.4	91
TP-4 (2006)	9	Sandy Silt	0.5	69
TP-7 (2006)	5	Silt minor sand	0.5	-
TP-13 (2006)	6.5	Silty Sand	1.5	42
TP-15 (2006)	6	Silty Sand	1	45

1. Fines content: material passing the U.S. Standard No. 200 sieve

4.5 DCP TESTING

GeoDesign performed DCP testing of the native subgrade soil in potential future pavement areas to estimate the resilient modulus of the subgrade. Testing was conducted in general accordance with ASTM D6951. We recorded penetration depth of the cone for each blow of the hammer and terminated testing at the end of rod length. Test information and results are summarized in Table 2.

Table 2. DCP Test Results

Test Location	Soil Type	Estimated Resilient Modulus (psi)
DCP-1	Silt	7,000
DCP-2	Silt	4,900
DCP-3	Silt	11,800

5.0 SLOPE STABILITY ANALYSES

We analyzed the global stability of the existing and proposed slope conditions using the slope stability-modeling program Slope/W 2018 by Geo-Slope International, Ltd. Slope/W performs two-dimensional limit equilibrium analysis to compute slope stability. Our analyses used the Morgenstern-Price method. The FOS against slope failure is defined as the ratio of the forces resisting slope movement (e.g., soil strength, soil mass, etc.) to the forces driving slope movement (e.g., gravity, water pressure, earthquake shaking). An FOS of less than 1.0 infers that the model is not in equilibrium and slope movement is likely to occur.

We analyzed global stability using four critical cross sections (A, B, C, and D) provided by KPFF Consulting Engineers and the current and prior geotechnical explorations. Figures 5 and 6 provide summarized cross sections for sections A-A' and B-B'. Our analyses are based on borings, CPTs, and test pits. Groundwater conditions were estimated based on the measured groundwater levels in the borings and in the vibrating wire piezometer installed in boring B-1 and pore water pressure dissipation tests completed in the CPTs.

The modeling parameters used for analysis of site soil were determined based on available current and prior field explorations and laboratory testing data. The soil properties used in the analyses are presented in Table 3. We believe these parameters are conservative values based on the current and prior explorations at the site and laboratory test results.

Table 3. Soil Parameters

Soil Unit	Moist Unit Weight (g_m) [pcf]	Cohesion (c) [psf]	Internal Friction Angle (f_r') [degrees]
Medium Stiff Silt	105	100	28
Loose, Silty Sand	105	50	31
Medium Dense Sand	110	0	34
Dense Gravel and Sand	125	0	36
Structural Fill[1]	110	100	30
Imported Granular Structural Fill[2]	135	0	37

1. Structural fill, as described in the "Structural Fill" section for "On-Site Soil."
2. Imported granular structural fill, as described in the "Structural Fill" section for "Imported Granular Material."

Seismic analysis was completed using a horizontal seismic coefficient equal to 0.14 g. Conservatively, we used the same friction angle and cohesion values for both static and seismic analyses, even though a higher friction angle and cohesion can be used for dynamic short-term loads. The stability analyses included surcharges for construction-related loads, as well as future building and parking lot loads.

Results of the global stability analyses for the existing and proposed slope conditions are summarized in Table 4 and presented in Appendix E. The minimum FOS values considered for long-term slope stability at the site are 1.5 and 1.1 for static and seismic conditions, respectively.

Table 4. Results of Global Stability Analyses

Cross Section	Condition	FOS
Section A - A'	Existing Slope Conditions – Static	1.6
	Existing Slope Conditions – Seismic	1.2
	Proposed Slope Conditions – Static	1.7
	Proposed Slope Conditions – Seismic	1.2
Section B - B'	Existing Slope Conditions – Static	2.8
	Existing Slope Conditions – Seismic	1.8
	Proposed Slope Conditions – Static	1.5
	Proposed Slope Conditions – Seismic	1.1
Section C - C'	Existing Slope Conditions – Static	1.5
	Existing Slope Conditions – Seismic	1.2
	Proposed Slope Conditions – Static	1.6
	Proposed Slope Conditions – Seismic	1.1

Table 4. Results of Global Stability Analyses (continued)

Cross Section	Condition	FOS
Section D-D’	Existing Slope Conditions – Static	1.6
	Existing Slope Conditions – Seismic	1.1
	Proposed Slope Conditions – Static	1.6
	Proposed Slope Conditions – Seismic	1.2

Our analyses indicate the computed FOS’s for existing and proposed slope conditions under static and seismic analyses satisfy the minimum FOS’s for global stability. The FOS’s for slope stability are greater than 1.5 and 1.1 for static and seismic conditions, respectively. However, localized areas of potential shallow instability (e.g., FOS less than 1.5 or 1.1 for static and seismic conditions, respectively) are present on the steep slopes located immediately above Willamette Falls Drive.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our subsurface explorations and engineering analyses, it is our opinion that the site can be developed as proposed. The primary geotechnical considerations for the project are summarized in the “Executive Summary.” Our specific recommendations are provided in the following sections.

7.0 DESIGN

7.1 PERMANENT SLOPES

Permanent cut or fill slopes on the site should not exceed a gradient of 2H:1V, unless specifically evaluated for stability. Slopes that will be maintained by mowing should not be constructed steeper than 3H:1V. Footings, buildings, access rods, and pavement should be located at least 5 feet horizontally from the face of slopes. Slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

7.2 DRAINAGE

7.2.1 Temporary Drainage

During grading at the site, the contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface and drainage onto slopes. During rough and finished grading of the building site, the contractor should keep all footing excavations and building pads free of water.

7.2.2 Surface Drainage

We recommend connecting all roof drains to a tightline leading to storm drain facilities. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. We also recommend sloping ground surfaces adjacent to the building away to facilitate drainage away from the building.

7.2.3 Keyway Drains

We recommend installing a subsurface drain to collect any perched water at the inside of the keyway cut for the fill slopes above Willamette Falls Drive. The drain should consist of a perforated drainpipe covered with a minimum 2-foot-wide and 2-foot-tall zone of drain rock wrapped in a drainage geotextile. Collected water should be routed in non-perforated line(s) to the stormwater system or to a suitable discharge at the base of the slope.

7.2.4 Cement-Amended Slope Drainage

We recommend installing drainage at the contact of relatively impervious cement-amended fill slopes and overlying topsoil to limit runoff onto the slopes below. Drainage should consist of angled strip drains pinned to the cement-amended slope on maximum spacings of 30 feet on-center and connected to minimum 2-foot-wide and 2-foot-deep zones of drain rock with perforated collector pipes. The surface of the cement-amended slopes should be roughened prior to placing the overlying topsoil. Water collected from the top of the cement-amended slopes should be routed in non-perforated line(s) to the stormwater system or a suitable discharge at the base of the slope. The collected water should not be connected to the perforated pipe for the subsurface keyway drain at the base of the fill.

7.2.5 Stormwater Infiltration Systems

We recommend locating any infiltration facilities below a 5H:1V projection from the base of any slopes and/or walls to limit the potential influence of groundwater on the stability of the slopes and walls. Any stormwater detention facilities within the 5H:1V projection from the base of slopes and/or walls should be lined to prevent infiltration near walls and slopes.

Infiltration testing was completed in explorations to evaluate the feasibility of shallow infiltration systems. The infiltration rate will depend on the fines content and consistency of the soil. Tested rates ranged from negligible to 1.5 inches per hour. The unfactored field rates in Table 1 can be used for design. It is the responsibility of the designer to include the appropriate FOS's for the systems.

We recommend that GeoDesign observe the soil conditions and complete confirmation testing during construction to verify the field rates meet the design rates. Due to the presence of variable fines content, it may be necessary to enlarge or deepen systems during construction. Furthermore, we recommend including a contingency to deepen infiltration systems or add additional infiltration systems in other portions of the site during construction if tested rates at the time of construction are unsuitable.

7.2.6 Foundation Drains

Where drains are not already required for embedded building walls, we recommend installing a perimeter foundation drain around the planned new building. The foundation drains should be constructed at a minimum slope of approximately ½ percent and drained by gravity to a suitable discharge. The perforated drainpipe should not be tied to a stormwater drainage system without backflow provisions. The foundation drains should consist of 4-inch-diameter, perforated drainpipe embedded in a minimum 2-foot-wide zone of crushed drain rock that extends up to 6 inches BGS and is wrapped in a drainage geotextile. The invert elevation of the drainpipe should be installed below the base of imported granular fill and base rock for the building and at least 18 inches below the finish floor elevation. The drain rock and drainage geotextile should meet the requirements specified in the "Materials" section.

7.3 SEISMIC DESIGN CRITERIA

7.3.1 ASCE 7-16 Seismic Design Parameters

Since the school is classified as a special occupancy structure, SOSSC requires a site-specific seismic evaluation. Seismic design criteria for this project will be based on the 2019 SOSSC and ASCE 7-16. A site-specific seismic evaluation was completed, the results of which are presented in Appendix F.

7.3.2 Liquefaction and Lateral Spreading

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Saturated silty soil with low plasticity is moderately susceptible to liquefaction or cyclic failure under relatively higher levels of ground shaking. We did not encounter any significant amount of soil considered to be susceptible to liquefaction or cyclic failure at the site. Since the site is not near an open face with saturated conditions and has low susceptibility to liquefaction, lateral spreading is expected to be negligible at this site.

7.4 SHALLOW FOUNDATION RECOMMENDATIONS

7.4.1 General

Based on the results of our explorations and analysis, the proposed school building and other associated structures can be supported by conventional spread footings bearing on a minimum 3-inch-thick layer of crushed rock underlain by undisturbed native soil or structural fill overlying firm native soil. Foundations should not be established on undocumented fill, soft soil, or soil containing deleterious material. If present, this material should be removed and replaced with granular pads.

We recommend placing a minimum 3-inch-thick granular pad over the footing subgrades to protect from disturbance since the silt and silty subgrades will be prone to disturbance during wet weather and the sand or sandy subgrades will be prone to disturbance when dry. If granular pads greater than 6 inches thick are required for the removal of unsuitable materials below footings, the granular pads should extend 6 inches beyond the margins of the footings for every foot excavated below the base grade of the footing. The granular pads should consist of imported granular material, as defined in the "Structural Fill" section. The imported granular material for granular pads 1 foot thick or greater should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557, or until well-keyed, as determined by one of our geotechnical staff. We recommend that a member of our geotechnical staff observe prepared footing subgrades and granular pads.

7.4.2 Dimensions and Capacities

Continuous wall and isolated spread footings should be at least 18 and 24 inches wide, respectively. The bottom of exterior footings should be at least 18 inches below the lowest adjacent exterior grade. The bottom of interior footings should be established at least 12 inches below the base of the slab.

Footings bearing on subgrade prepared as recommended above should be sized based on an allowable bearing pressure of 2,500 psf. This is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and may be doubled for short-term loads, such as those resulting from wind or seismic forces.

7.4.3 Resistance to Sliding

Lateral loads on footings can be resisted by passive earth pressure on the sides of the structure and by friction on the base of the footings. Our analysis indicates that the available passive earth pressure for footings confined by on-site soil and structural fill is 325 pcf, modeled as an equivalent fluid pressure. Typically, the movement required to develop the available passive resistance may be relatively large; therefore, we recommend using a reduced passive equivalent fluid pressure of 250 pcf. Adjacent floor slabs, pavement, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. This value for passive resistance assumes the subgrade is level; so in order to rely on the passive resistance, there should be a horizontal distance of at least 5 feet between the edge of the wall/footing and the face of nearby slopes. For footings in contact with imported granular material, a coefficient of friction equal to 0.40 may be used when calculating resistance to sliding.

7.4.4 Settlement

Based on the anticipated foundation loads, post-construction settlement of footings and floor slabs founded as recommended is anticipated to be less than 1 inch. Differential settlement between similarly loaded, newly constructed foundation elements should be approximately one-half of the total settlement. Differential settlement between structurally isolated new and existing foundation elements may range up to the total estimated settlement. Differential settlement between abutting existing and new foundation elements can be reduced by structurally tying the new and existing foundation elements together.

7.4.5 Subgrade Evaluations

All footing subgrades should be evaluated by a member of our geotechnical staff. Observations should also evaluate whether all loose or soft material, organic material, unsuitable fill, prior topsoil zones, and softened subgrades (if present) have been removed. Localized deepening of footing excavations may be required to penetrate deleterious material, if encountered.

7.5 RETAINING STRUCTURES

7.5.1 General

Retaining walls will be required as part of construction of the school campus. Based on the site proposed grading plan, we anticipate walls will be less than 15 feet high, except for one taller embedded building wall between the east side of the school and Dollar Street that will range up to approximately 25 feet high.

Any geogrid-reinforced walls should be designed and constructed with imported granular material for the reinforced backfill zone and a minimum toe embedment of 1 foot. Foundation loads should not be located above and underground utilities should be located outside the geogrid-reinforced zone for the walls. We recommend GeoDesign be contacted to review any proposed grading revisions and geogrid-reinforced wall designs for compatibility with our recommendations.

7.5.2 Assumptions

Our retaining wall design recommendations are based on the following assumptions: (1) the walls consist of a cantilever, gravity, or conventional CIP concrete walls, (2) the walls will be less than 25 feet in height, (3) the backfill is drained and consists of imported granular material, and (4) the appropriate wall surcharges are included in the design as described in this section.

7.5.3 Wall Design Parameters

Cantilever, gravity, or conventional retaining walls can be designed using the pressures in this section. For unrestrained fill retaining walls, we recommend using an active equivalent fluid pressure of 35 pcf for design. Retaining walls that will be restrained from rotation prior to being backfilled should be designed using an equivalent fluid pressure of 55 pcf. For embedded building walls, a superimposed seismic lateral force should be calculated based on a dynamic force of $6.5H^2$ pounds per lineal foot of wall (where H is the height of the wall in feet). The load should be applied as a distributed load with the centroid located at a distance of 0.6H from the base of the wall.

7.5.4 Wall Surcharges

The design equivalent fluid pressures should be increased for walls that retain sloping soil. We recommend the lateral earth pressures be increased using the following factors (Table 5) when designing walls that retain sloping soil.

Table 5. Lateral Earth Pressure Increase Factors for Slope Soil Backfill

Slope of Retained Soil (degrees)	Lateral Earth Pressure Increase Factor
0	1.00
5	1.06
10	1.12
20	1.33
25	1.52
30	2.27

Lateral earth pressures from building foundations or other surcharges located within a horizontal distance from the back of a wall equal to the height of the wall can be calculated as shown on Figure 7.

7.5.5 Wall Foundations

All retaining wall foundations should be designed and constructed as described in the "Shallow Foundation Recommendations" section.

7.5.6 Wall Backfill and Drains

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

The backfill material placed behind the walls and extending a horizontal distance of ½H (where H is the height of the retaining wall) should consist of imported granular material placed and compacted in conformance with the "Materials" section.

A minimum 6-inch-diameter, perforated collector pipe should be placed at the base of the walls. The pipe should be embedded in a minimum 2-foot-wide zone of angular drain rock that is wrapped in a drainage geotextile fabric and extends up the back of the wall to within 1 foot of the finished grade. The drain rock and drainage geotextile fabric should meet specifications provided in the "Materials" section. Drainage mats can be used in lieu of the 2-foot-wide drain rock zone.

The perforated collector pipes should discharge at an appropriate location away from the base of the wall and any slopes. The discharge pipe(s) should only be tied directly into stormwater drain systems if measures are taken to prevent backflow into the drainage system of the walls.

7.5.7 Construction Considerations

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

7.6 PAVEMENT RECOMMENDATIONS

7.6.1 General

At the time this report was prepared we had not been provided with anticipated traffic volumes and distribution. Based on the school facility proposed, we assume traffic will consist primarily of passenger cars and busses. We anticipate that AC pavement will be used for passenger car drive aisles and parking areas. Pavement should be installed on undisturbed native subgrade, scarified and re-compacted soil, or new engineered fills as described in the "Site Preparation" and "Structural Fill" sections.

Our pavement recommendations are based on the following assumptions:

- A design life of 20 years for AC.
- A resilient modulus of 20,000 psi was estimated for aggregate base.
- Initial and terminal serviceability indices of 4.2 and 2.0, respectively, for AC.
- Reliability of 85 percent and standard deviation of 0.45 for AC.
- Structural coefficients of 0.42 and 0.10 for AC and aggregate base, respectively.
- The number of buses and trucks indicated below, plus trucks are assumed to be 50 percent two-axle and 50 percent three-axle trucks. We have not included a growth factor. Analysis of alternative traffic assumptions can be completed if requested.
- A resilient modulus of 4,500 psi for subgrade prepared in accordance with the "Site Preparation" section.

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

7.6.2 Flexible AC Pavement Recommendations

Based on the traffic assumptions provided above, we recommend the AC pavement sections in Table 6.

Table 6. Recommended Standard Pavement Sections

Pavement Use	Busses per Day	Trucks per Day[1]	ESALs	AC Thickness (inches)	Aggregate Base Thickness (inches)
Automobile Parking	0	0	10,000	2.5	9.0
Limited Truck Drive Aisles	0	10	50,000	3.0	10.0
Bus Areas	10	10	103,000	4.0	12.0
	20	10	161,000	4.5	12.0
	30	10	219,000	4.5	13.0

1. Trucks assumed to be 50 percent two-axle and 50 percent three-axle trucks.

If the subgrade is cement amended to the thicknesses indicated below and the amended soil achieves a seven-day unconfined compressive strength of at least 100 psi, the pavement can be constructed as recommended in Table 7.

Table 7. Recommended Pavement Sections Using Cement Amendment

Pavement Use	Busses per Day	Trucks per Day[1]	ESALs	AC Thickness (inches)	Aggregate Base Thickness (inches)	Cement Amendment[2] (inches)
Automobile Parking	0	0	10,000	2.5	4.0	12.0
Limited Truck Drive Aisles	0	10	50,000	3.0	4.0	12.0
Bus Areas	10	10	103,000	4.0	5.0	12.0
	20	10	161,000	4.5	5.0	12.0
	30	10	219,000	4.5	6.0	12.0

1. Trucks assumed to be 50 percent two-axle and 50 percent three-axle trucks.
2. Assumes a minimum seven-day unconfined compressive strength of 100 psi.

All of the recommended pavement sections with subgrades prepared as recommended are suitable to support an occasional 80,000-pound fire truck. All thicknesses are intended to be the minimum acceptable. Design of the recommended pavement section is based on the assumption that construction will be completed during an extended period of dry weather. Wet weather construction could require an increased thickness of aggregate base. In addition, to prevent strength loss during curing, cement-amended soil should be allowed to cure for at least four days prior to construction traffic or placing the base rock. Lastly, the amended subgrade should be protected with a minimum of 4 inches of base rock prior to construction traffic access.

Construction traffic should be limited to non-building, unpaved portions of the site or haul roads. Construction traffic should not be allowed on new pavement. If construction traffic is to be allowed on newly constructed road sections, an allowance for this additional traffic will need to be made in the design pavement section. For reference relative to the recommended design sections and ESALs in the tables above, a mix of 30 additional two-axle and three-axle trucks per day over a construction duration of two years would result in an increase of approximately 12,000 ESALs.

The AC, aggregate base, and cement amendment should meet the requirements outlined in the "Materials" section.

8.0 CONSTRUCTION

8.1 EROSION CONTROL

When exposed, the soil at this site can be eroded by wind and water; therefore, erosion control measures should be carefully planned and in place before construction begins. Measures employed to reduce erosion include, but are not limited to, silt fences, hay bales, plastic sheeting, buffer zones of natural growth, and sedimentation ponds.

8.2 SITE PREPARATION

8.2.1 Demolition

Demolition includes removal of the existing buildings, pavement, concrete curbs, abandoned utilities, and any subsurface elements. Demolished material should be transported off site for disposal. Excavations remaining from removing basements (if present), foundations, utilities, and other subsurface elements should be backfilled with structural fill where these are below planned site grades. The base of the excavations should be excavated to expose firm subgrade before filling. The sides of the excavations should be cut into firm material and sloped a minimum of 1½H:1V. Utility lines abandoned under new structural components should be completely removed and backfilled with structural fill. Soft or disturbed soil encountered during demolition should be removed and replaced with structural fill.

A monitoring well was observed in 2006 near the former residence off Dollar Street at the site. Any wells on the property, excluding the fully grouted vibrating wire piezometer installed in boring B-1, should be decommissioned in accordance with OWRD regulations. In addition, septic tanks (if present) should be pumped out, removed, and disposed of properly. Septic leach fields (if present) should also be excavated and disposed of properly. Resulting excavations should be backfilled with structural fill.

8.2.2 Stripping

The existing root zone should be stripped and removed from all fill areas. Based on our explorations, the average depth of stripping will be approximately 4 to 5 inches, although greater stripping depths may be required to remove localized zones of loose or organic soil. Greater stripping depths averaging approximately 6 inches should be anticipated in areas with thicker vegetation and along the base of draws. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaped areas.

8.2.3 Topsoil Zone

An approximately 6- to 18-inch-thick topsoil zone was observed over most of the site. Topsoil zones typically have lower densities and contain slightly higher organic contents. The topsoil generally exhibits low strength and does not provide adequate subgrade support for foundation elements or pavement. We recommend improving the topsoil zone during site preparation where it will not be removed by site cuts.

In all structural fill, pavement, and improvement areas the soil in topsoil zones should be removed and replaced with structural fill or scarified and compacted in place. Scarification and compaction of the subgrade is the most economical option for subgrade improvement; it will likely only be possible during extended dry periods and following moisture conditioning of the soil. As discussed further on in this report, cement amendment is an option for conditioning the soil for use as structural fill during periods of wet weather or when drying the soil is not an option.

8.2.4 Subgrade Evaluation

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

8.2.5 Test Pit Locations

The test pit excavations were backfilled using relatively minimal compactive effort; therefore, soft areas can be expected at these locations. We recommend that this relatively uncompacted soil be removed from the test pits located within proposed foundation and paved areas to a depth of 3 feet BGS. The resulting excavation should be brought back to grade with structural fill. Deeper removal depth will be required where foundations are located over test pit locations.

8.3 SUBGRADE PROTECTION

The fine-grained soil present on this site is easily disturbed. If not carefully executed, site preparation, utility trench work, and roadway excavation can create extensive soft areas and significant repair costs can result. Earthwork planning, regardless of the time of year, should include considerations for minimizing subgrade disturbance.

If construction occurs during or extends into the wet season, or if the moisture content of the surficial soil is more than a couple percentage points above the optimum moisture content, site stripping and cutting may need to be accomplished using track-mounted equipment. Likewise, the use of granular haul roads and staging areas will be necessary for support of construction traffic during the rainy season or when the moisture content of the surficial soil is more than a few percentage points above the optimum moisture content. The amount of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor's sequencing of a project and type/frequency of construction equipment. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul roads areas. Stabilization material may be used as a substitute, provided the top 4 inches of material consists of imported granular material. The actual thickness will depend on the contractor's means and methods and, accordingly, should be the contractor's responsibility. In addition, a geotextile fabric should be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic. The imported granular material, stabilization material, and geotextile fabric should meet the specifications in the "Materials" section.

As an alternative to thickened crushed rock sections, haul roads and utility work zones may be constructed using amended subgrades overlain by a crushed rock wearing surface. If the subgrade is amended, the thickness of granular material in staging areas and along haul roads can typically be reduced to between 6 and 9 inches. This recommendation is based on an assumed minimum unconfined compressive strength of 100 psi for subgrade amended to a depth of 12 to 16 inches. The actual thickness of the amended material and imported granular material will depend on the contractor's means and methods and, accordingly, should be the contractor's responsibility. Amendment is discussed in the "Materials" section.

8.4 EXCAVATION

8.4.1 Temporary Cuts

Temporary cuts may be required in order to construct the proposed retaining walls. Excavations into the slopes need to be carefully planned so as not to destabilize the slope. Cuts less than 4 feet should stand vertical. Deeper excavations up to 15 feet high should be cut back at an inclination 1H:1V or flatter and excavations up to 30 feet high should be cut back at an inclination of 1.5H:1V or flatter. Excavations should be flattened or shored if excessive sloughing or raveling occurs or groundwater seepage is encountered. Excavations greater than 10 feet high should also be completed and backfilled in sections not exceeding 100 feet in length. The top of temporary slopes should be located at least 5 feet from pavement, utilities, buildings, or other such structures. Sloughing of temporary slopes can be expected, and maintenance during construction will likely be required, particularly during wet weather. All temporary slopes should be made and maintained in accordance with applicable OSHA and state regulations.

8.4.2 Trench Cuts and Shoring

Most cuts should be readily completed with conventional excavation equipment. Temporary excavation sidewalls should stand vertical to a depth of approximately 4 feet, provided groundwater seepage is not observed in the sidewalls. Open excavation techniques may be used to excavate trenches with depths between 4 and 8 feet, provided the walls of the excavation are cut at a slope of 1H:1V and groundwater seepage is not present. At this inclination, the slopes may slough and require some ongoing repair. Excavations should be flattened to 1½H:1V or 2H:1V if excessive sloughing or raveling occurs. In lieu of large and open cuts, approved temporary shoring may be used for excavation support. A wide variety of shoring and dewatering systems are available. Consequently, we recommend that the contractor be responsible for selecting the appropriate shoring and dewatering systems. If box shoring is used, it should be understood that box shoring is a safety feature used to protect workers and does not prevent caving. If excavations are left open for extended periods of time, caving of the sidewalls may occur. The presence of caved material will limit the ability to properly backfill and compact the trenches. The contractor should be prepared to fill voids between the box shoring and the sidewalls of the trenches with sand or gravel before caving occurs.

If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation. All excavations should be made in accordance with applicable OSHA and state regulations.

8.4.3 Trench Dewatering

Excavations for the proposed development are not anticipated to extend below the static groundwater table, and significant dewatering operations are not expected. Runoff water may accumulate in excavations during periods of precipitation and perched groundwater may be encountered, particularly during the wet season or extended periods of wet weather. A sump located within the trench excavation likely will be sufficient to remove the accumulated water, depending on the amount and persistence of water seepage and the length of time the trench is left open. Flow rates for dewatering are likely to vary depending on location, soil type, and the season during which the excavation occurs. The dewatering systems should be capable of adapting to variable flows.

If groundwater is present at the base of utility excavations, we recommend placing at least 12 inches of stabilization material at the base of the excavations. Trench stabilization material should meet the requirements provided in the "Structural Fill" section.

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

8.4.4 Safety

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

8.5 MATERIALS

8.5.1 Structural Fill

8.5.1.1 General

Fill should be placed on subgrade that has been prepared in conformance with the "Site Preparation" section. A variety of material may be used as structural fill at the site. However, all material used as structural fill should be free of organic material or other unsuitable materials and should meet the specifications provided in OSSC 00330 (Earthwork), OSSC 00400 (Drainage and Sewers), and OSSC 02600 (Aggregates), depending on the application. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill are provided below.

In locations where fill is to be placed on slopes steeper than 5H:1V, level benches should be cut into the existing sloping surfaces. The benches should be a minimum of 10 feet wide or 1½ times the width of the compaction equipment, whichever is wider. Fill slopes with grades of 3H:1V or steeper should also be overbuilt by at least 2 feet and cut back to finish grade.

8.5.1.2 On-Site Soil

The material at the site should be suitable for use as general structural fill, provided it is properly moisture conditioned; free of debris, organic material, and particles over 4 inches in diameter; and meets the specifications provided in OSSC 00330.12 (Borrow Material).

Based on laboratory test results, the moisture content of the on-site silt and silty soil is above the optimum moisture content for compaction. We estimate the optimum moisture content for compaction to be approximately 16 to 19 percent for the on-site soil. Moisture conditioning (drying) will be required to use on-site soil for structural fill. Accordingly, extended dry weather will be required to adequately condition and place the soil as structural fill. It will be difficult, if not impossible, to adequately compact on-site soil during the rainy season or during prolonged periods of rainfall.

When used as structural fill, native soil should be placed in lifts with a maximum uncompacted thickness of 6 to 8 inches and compacted to not less than 92 percent of the maximum dry density for fine-grained soil and 95 percent of the maximum dry density for granular soil, as determined by ASTM D1557.

8.5.1.3 Imported Granular Material

Imported granular material used as structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.14 (Selected Granular Backfill) or OSSC 00330.15 (Selected Stone Backfill). The imported granular material should also be angular, should be fairly well graded between coarse and fine material, should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have at least two fractured faces. Material with a higher fines content of up to 12 percent is permissible, provided compaction can be achieved.

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

8.5.1.4 Stabilization Material

Stabilization material used in staging or haul road areas or in trenches should consist of 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.15 (Selected Stone Backfill). The material should have a maximum particle size of 6 inches, should have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and should have at least two mechanically fractured faces. The material should be free of organic material and other deleterious materials. Stabilization material should be placed in lifts between 12 and 24 inches thick and compacted to a firm condition.

8.5.1.5 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of well-graded granular material with a maximum particle size of 1½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.13 (Pipe Zone Material). The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of well-graded granular material with a maximum particle size of 2½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.14 (Trench Backfill; Class B, C, or D). This material should be compacted to at least 90 percent of the maximum dry density, as determined by

ASTM D1557, or as required by the pipe manufacturer or local building department. The upper 3 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557.

Outside of structural improvement areas (e.g., roadway alignments or building pads) trench backfill placed above the pipe zone may consist of general fill material that is free of organic material and material over 6 inches in diameter and meets the specifications provided in OSSC 00405.14 (Trench Backfill; Class A, B, C, or D). This general trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

8.5.1.6 Drain Rock

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

8.5.1.7 Aggregate Base Rock

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

8.5.2 Geotextile Fabric

8.5.2.1 Subgrade Geotextile

The subgrade geotextile should meet the specifications provided in OSSC Table 02320-4 – Geotextile Property Values for Subgrade Geotextile (Separation). The geotextile should be installed in conformance with OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles. All drainage aggregate and stabilization material should be underlain by a subgrade geotextile. Geotextile is not required where stabilization material is used at the base of utility trenches.

8.5.2.2 Drainage Geotextile

Drainage geotextile should meet the specifications provided in OSSC Table 02320-1 – Geotextile Property Values for Drainage Geotextile. The geotextile should be installed in conformance with OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

8.5.3 Cement Amendment

8.5.3.1 General

Cement amendment can be used to stabilize subgrade and protect it from damage due to repeated construction traffic during wet conditions. Cement amendment can also serve as an alternative to the use of imported granular material for wet weather structural fill. Successful use of soil amendment depends on the use of correct mixing techniques, soil moisture content, and amendment quantities. The amount of cement used during amendment should be based on an assumed soil dry unit weight of 110 pcf.

8.5.3.2 Subbase Stabilization

Specific recommendations based on exposed site conditions for soil amending can be provided if necessary. However, for preliminary design purposes, we recommend a target strength for cement-amended subgrade for building and pavement subbase (below aggregate base) soil of 100 psi. The amount of cement used to achieve this target generally varies with moisture content and soil type. It is difficult to predict field performance of soil to cement amendment due to variability in soil response, and we recommend laboratory testing to confirm expectations. Generally, 5 percent cement by weight of dry soil can be used when the soil moisture content does not exceed approximately 20 percent. If the soil moisture content is in the range of 25 to 35 percent, 6 to 9 percent by weight of dry soil is recommended. The amount of cement added to the soil may need to be adjusted based on field observations and performance. Moreover, depending on the time of year and moisture content levels during amendment, water may need to be applied during tilling to appropriately condition the soil moisture content.

For building and pavement subbase, we recommend assuming a minimum cement ratio of 6 percent (by dry weight). If the soil moistures are in excess of 30 percent, a cement ratio of 7 to 8 percent will likely be needed. Due to the higher organic content and moisture, we recommend using a cement ratio of 8 percent when stabilizing topsoil and tilled zone material for building and pavement subbase and anticipate that the cement will need to be applied in two 4 percent applications followed by multiple tilling passes with each application.

We recommend cement-spreading equipment be equipped with balloon tires to reduce rutting and disturbance of the fine-grained soil. A static sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds should be used for initial compaction of the fine-grained soil. A smooth-drum roller with a minimum applied linear force of 700 pounds per inch should be used for final compaction. The amended soil should be compacted to at least 92 percent of the achievable dry density at the moisture content of the material, as defined in ASTM D1557.

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

Amendment depths for building/pavement, haul roads, and staging areas are typically on the order of 12, 16, and 12 inches, respectively. The crushed rock typically becomes contaminated with soil during construction. Contaminated base rock should be removed and replaced with clean rock in pavement areas. The actual thickness of the amended material and imported granular material for haul roads and staging areas will depend on the anticipated traffic and the contractor's means and methods and should be the contractor's responsibility.

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

8.5.3.3 Cement-Amended Structural Fill

On-site soil that is not suitable for structural fill due to high moisture content may be amended and placed as fill over a subgrade prepared in conformance with the "Site Preparation" section. Cement-amended structural fill should be placed in maximum uncompacted lift thicknesses of 12 inches. The cement ratio for general cement-amended fill can generally be reduced by 1 percent (by dry weight). Typically, a minimum curing of four days is required between amendment and construction traffic access. Consecutive lifts of fill may be amended immediately after the previous lift has been amended and compacted (e.g., the four-day wait period does not apply). However, where the final lift of fill is a building or roadway subgrade, the four-day wait period is in effect for the final lift of cement-amended soil.

8.5.3.4 Other Considerations

Portland cement-amended soil is hard and has low permeability. This soil does not drain well and it is not suitable for planting. Future planted areas should not be cement amended, if practical, or accommodations should be made for drainage and planting. Moreover, cement amending soil within building areas must be done carefully to avoid trapping water under floor slabs. We should be contacted if this approach is considered. Cement amendment should not be used if runoff during construction cannot be directed away from adjacent wetlands (if any).

8.5.3.5 Specification Recommendations

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

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

- Compaction Equipment
  - Use a static, sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds for initial compaction of fine-grained soil (silt and clay) or an alternate approved by the geotechnical engineer.

8.5.4 AC

The AC should be Level 2, ½-inch, dense ACP and compacted to 91 percent of the theoretical maximum density of the mix, as determined by AASHTO T 209. The minimum and maximum lift thicknesses should be 2.0 and 3.0 inches, respectively, for ½-inch ACP. Asphalt binder should be performance graded and conform to PG 64-22 or better.

9.0 OBSERVATION OF CONSTRUCTION

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

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

10.0 LIMITATIONS

We have prepared this report for use by West Linn-Wilsonville School District and members of the design and construction teams for the proposed project. The data and report can be used for bidding or estimating purposes, but our report, conclusions, and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other nearby building sites.

Exploration observations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The site development plans and design details were preliminary at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, configuration, design loads, or type of construction, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

The scope does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in this report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in this area at the time this report was prepared. No warranty, express or implied, should be understood.

♦ ♦ ♦

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

Sincerely,

GeoDesign, Inc.

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

REGISTERED PROFESSIONAL
ENGINEER
63114PE
OREGON
March 14, 2006
SHAWN DIMKE
EXPIRES: 12/31/21

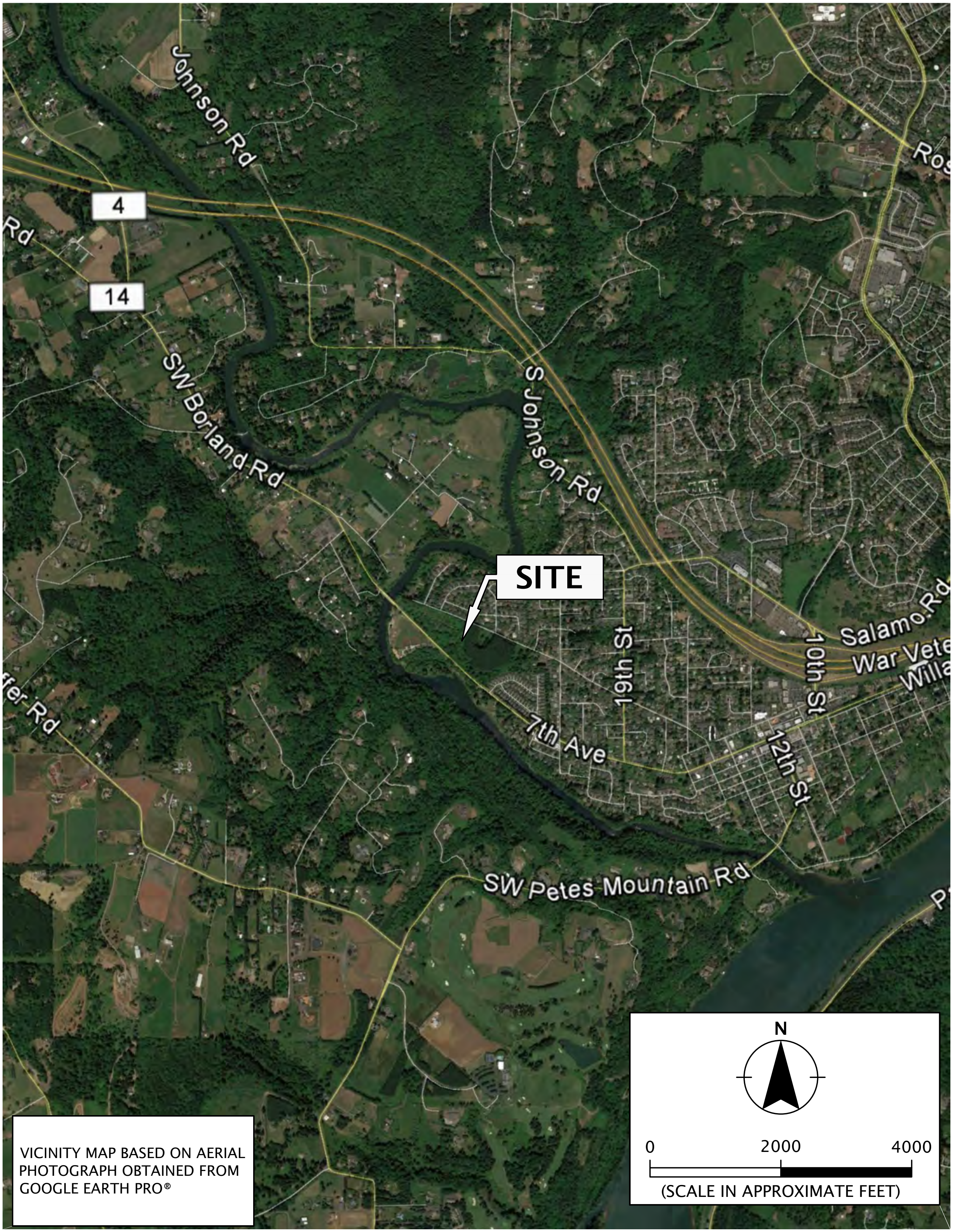
REFERENCES

Burns, W.J., and Watzig, R.J., 2014, *Statewide Landslide Information Layer for Oregon, Release 3 (SLIDO-3.0),* Oregon Department of Geology and Mineral Industries.

GeoDesign, Inc., 2006. *Report of Geotechnical Engineering Services; Renaissance at River Bend Subdivision; 945 Dollar Street; West Linn, Oregon,* dated September 1, 2006.

Schlicker, H.G. and C.T. Finlayson, 1979. *Geology and Geologic Hazards of Northwestern Clackamas County, Oregon*, Oregon Department of Geology and Mineral Industries, Bulletin 99.

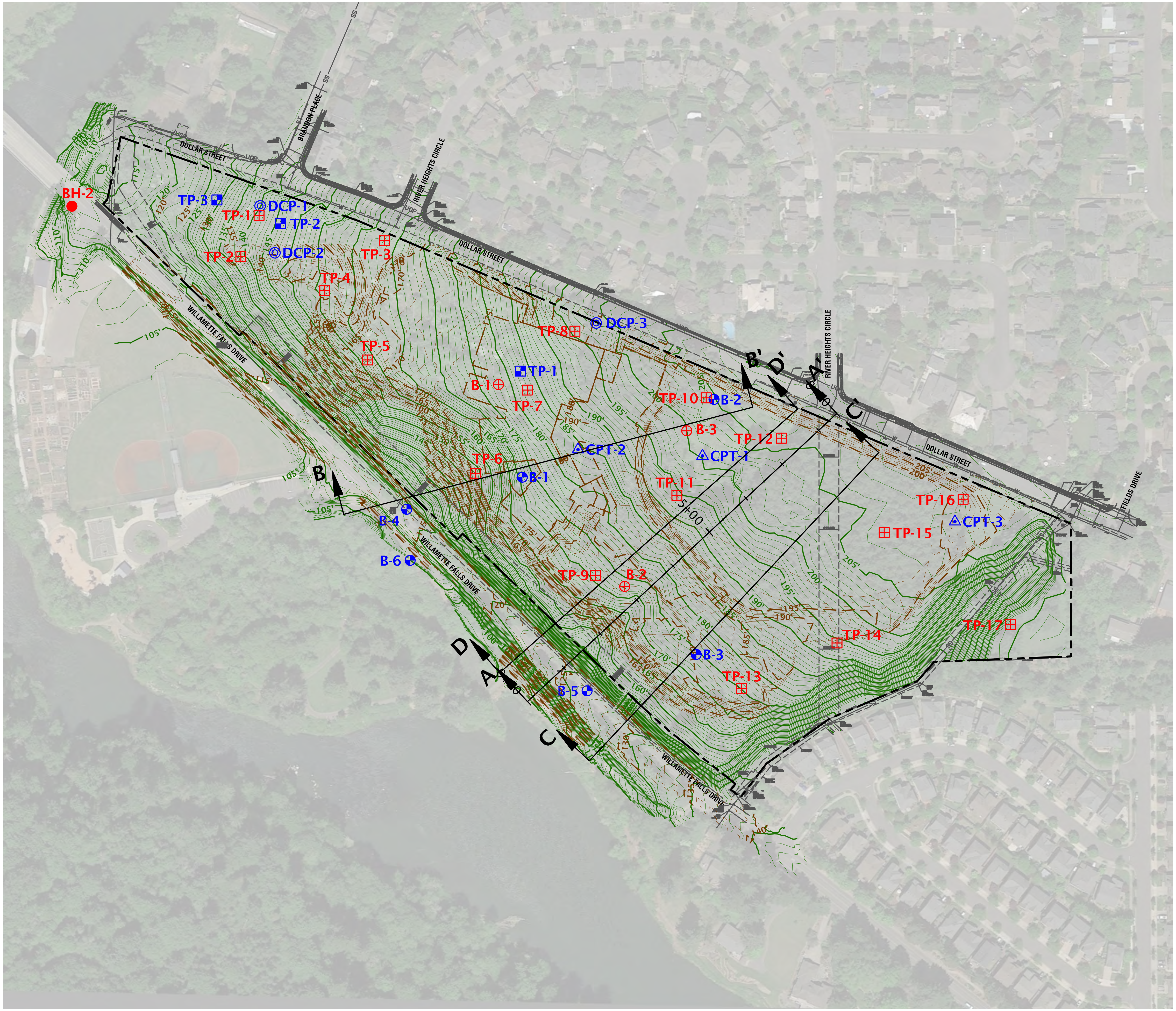
FIGURES



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

GEODESIGN INC AN NV5 COMPANY	WLWSCHDIST-1-01	VICINITY MAP	
	OCTOBER 2020	DOLLAR STREET MIDDLE SCHOOL WEST LINN, OR	FIGURE 1

Printed By: mmiller | Print Date: 10/19/2020 11:13:24 AM
File Name: J:\S-Z\WLWSchDist\WLWSchDist-1\WLWSchDist-1-01\Figures\CAD\WLWSchDist-1-01-VM01.dwg | Layout: FIGURE 1



LEGEND:

- SITE BOUNDARY
- B-1 BORING
- TP-1 TEST PIT
- DCP-1 DYNAMIC CONE PENETROMETER
- CPT-1 CONE PENETRATION TEST
- BH-2 PRIOR BORING (FOUNDATION ENGINEERING, INC. 2007)
- B-1 PRIOR BORING (GEODESIGN 2006)
- TP-1 PRIOR TEST PIT (GEODESIGN 2006)
- EXISTING TOPOGRAPHY
- PROPOSED TOPOGRAPHY
- A A' CROSS SECTION *

* CROSS SECTIONS A-A' AND B-B' ARE USED FOR GEOLOGIC PROFILES AND CROSS SECTIONS C-C' AND D-D' ARE USED FOR SLOPE STABILITY ANALYSIS

N

0 200 400

(SCALE IN FEET)

NOTES:

1. SITE PLAN BASED ON TOPOGRAPHIC SITE MAP DRAWING DATED JANUARY 2020 CREATED BY COMPASS LAND SURVEYORS.
2. AERIAL PHOTOGRAPH (MAY 8, 2019) OBTAINED FROM GOOGLE EARTH PRO.

FIGURE 2

SITE PLAN

DOLLAR STREET MIDDLE SCHOOL
WEST LINN, OR

WLWSCHDIST-1-01

OCTOBER 2020

GEODESIGN INC
AN NV5 COMPANY

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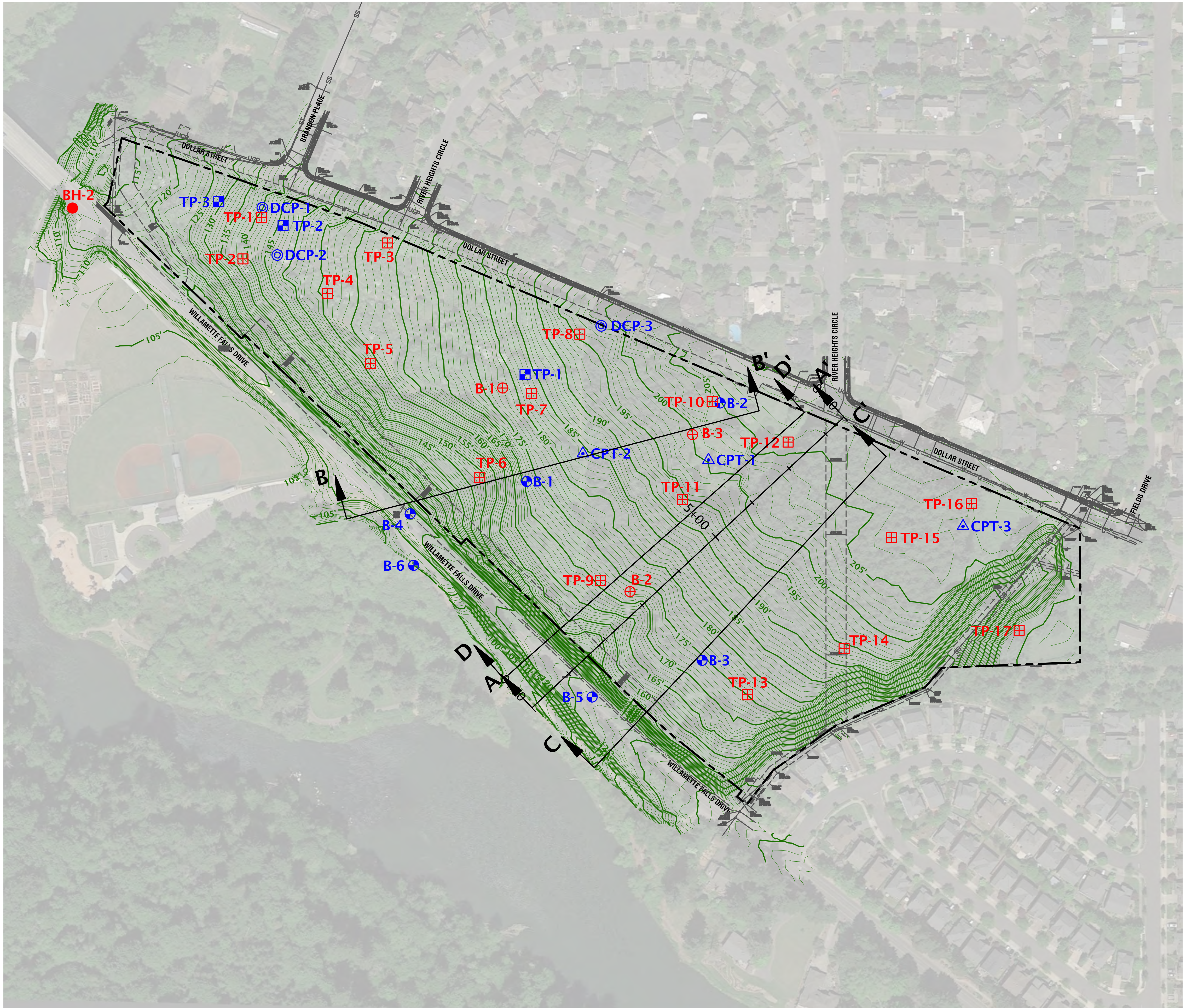

LEGEND:

	SITE BOUNDARY
B-1	BORING
TP-1	TEST PIT
DCP-1	DYNAMIC CONE PENETROMETER
CPT-1	CONE PENETRATION TEST
BH-2	PRIOR BORING (FOUNDATION ENGINEERING, INC. 2007)
B-1	PRIOR BORING (GEODESIGN 2006)
TP-1	PRIOR TEST PIT (GEODESIGN 2006)
	EXISTING TOPOGRAPHY
A A'	CROSS SECTION *

* CROSS SECTIONS A-A' AND B-B' ARE USED FOR GEOLOGIC PROFILES AND CROSS SECTIONS C-C' AND D-D' ARE USED FOR SLOPE STABILITY ANALYSIS

N

0 200 400
(SCALE IN FEET)

NOTES:

1. SITE PLAN BASED ON TOPOGRAPHIC SITE MAP DRAWING DATED JANUARY 2020 CREATED BY COMPASS LAND SURVEYORS.
2. AERIAL PHOTOGRAPH (MAY 8, 2019) OBTAINED FROM GOOGLE EARTH PRO.

GEODESIGN INC
AN NV5 COMPANY

WLWSCHDIST-1-01

OCTOBER 2020

SITE PLAN - EXISTING TOPOGRAPHY

DOLLAR STREET MIDDLE SCHOOL
WEST LINN, OR

FIGURE 3

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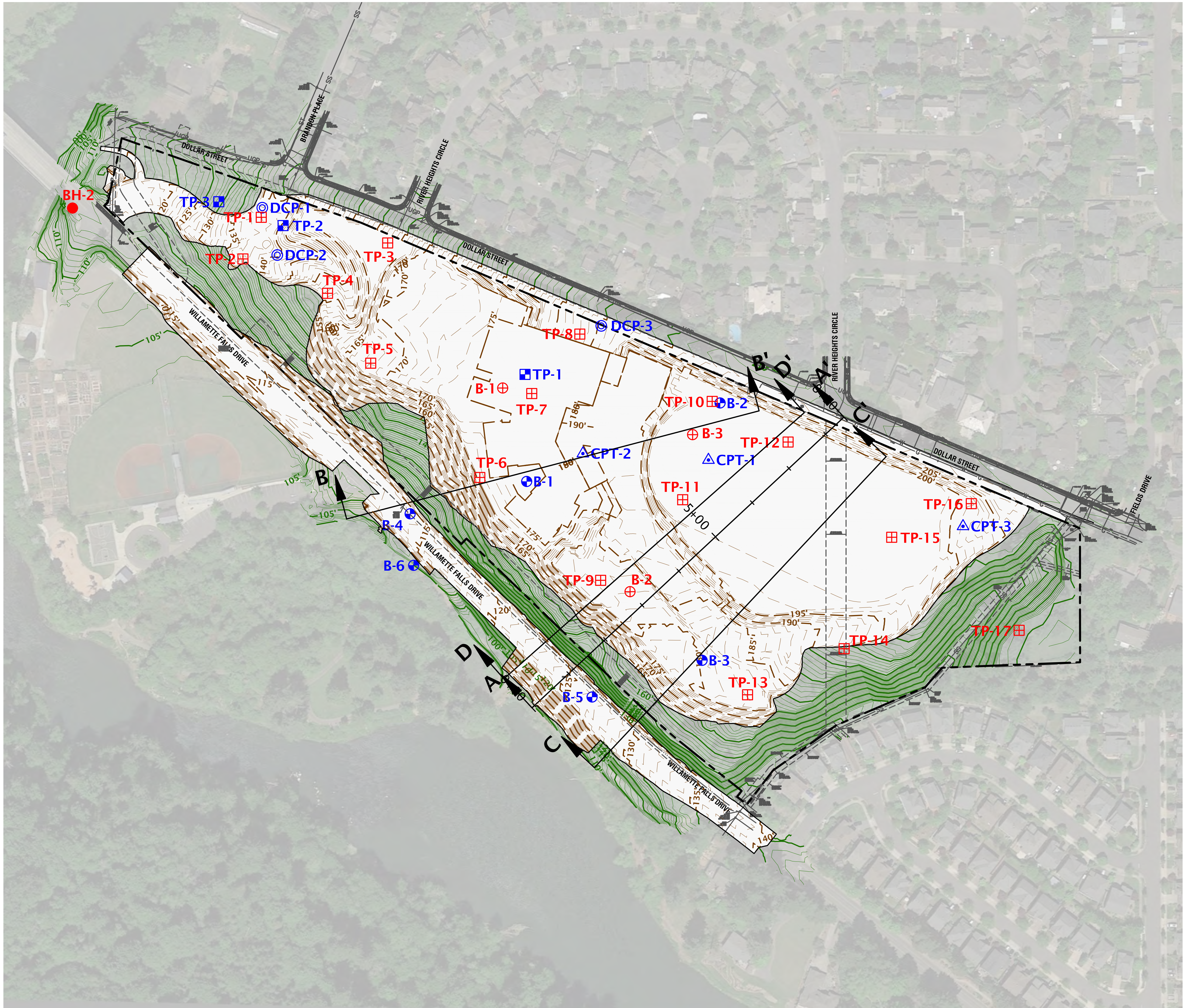

LEGEND:

SITE BOUNDARY

B-1 BORING

TP-1 TEST PIT

DCP-1 DYNAMIC CONE PENETROMETER

CPT-1 CONE PENETRATION TEST

BH-2 PRIOR BORING (FOUNDATION ENGINEERING, INC. 2007)

B-1 PRIOR BORING (GEODESIGN 2006)

TP-1 PRIOR TEST PIT (GEODESIGN 2006)

EXISTING TOPOGRAPHY

PROPOSED TOPOGRAPHY

A A' CROSS SECTION *

* CROSS SECTIONS A-A' AND B-B' ARE USED FOR GEOLOGIC PROFILES AND CROSS SECTIONS C-C' AND D-D' ARE USED FOR SLOPE STABILITY ANALYSIS

N

0 200 400

(SCALE IN FEET)

NOTES:

1. SITE PLAN BASED ON TOPOGRAPHIC SITE MAP DRAWING DATED JANUARY 2020 CREATED BY COMPASS LAND SURVEYORS.
2. AERIAL PHOTOGRAPH (MAY 8, 2019) OBTAINED FROM GOOGLE EARTH PRO.

FIGURE 4

SITE PLAN - FINAL TOPOGRAPHY

DOLLAR STREET MIDDLE SCHOOL
WEST LINN, OR

WLWSCHDIST-1-01

OCTOBER 2020

GEODESIGN INC. AN NV5 COMPANY

Printed By: mmiller | Print Date: 10/19/2020 11:13:48 AM
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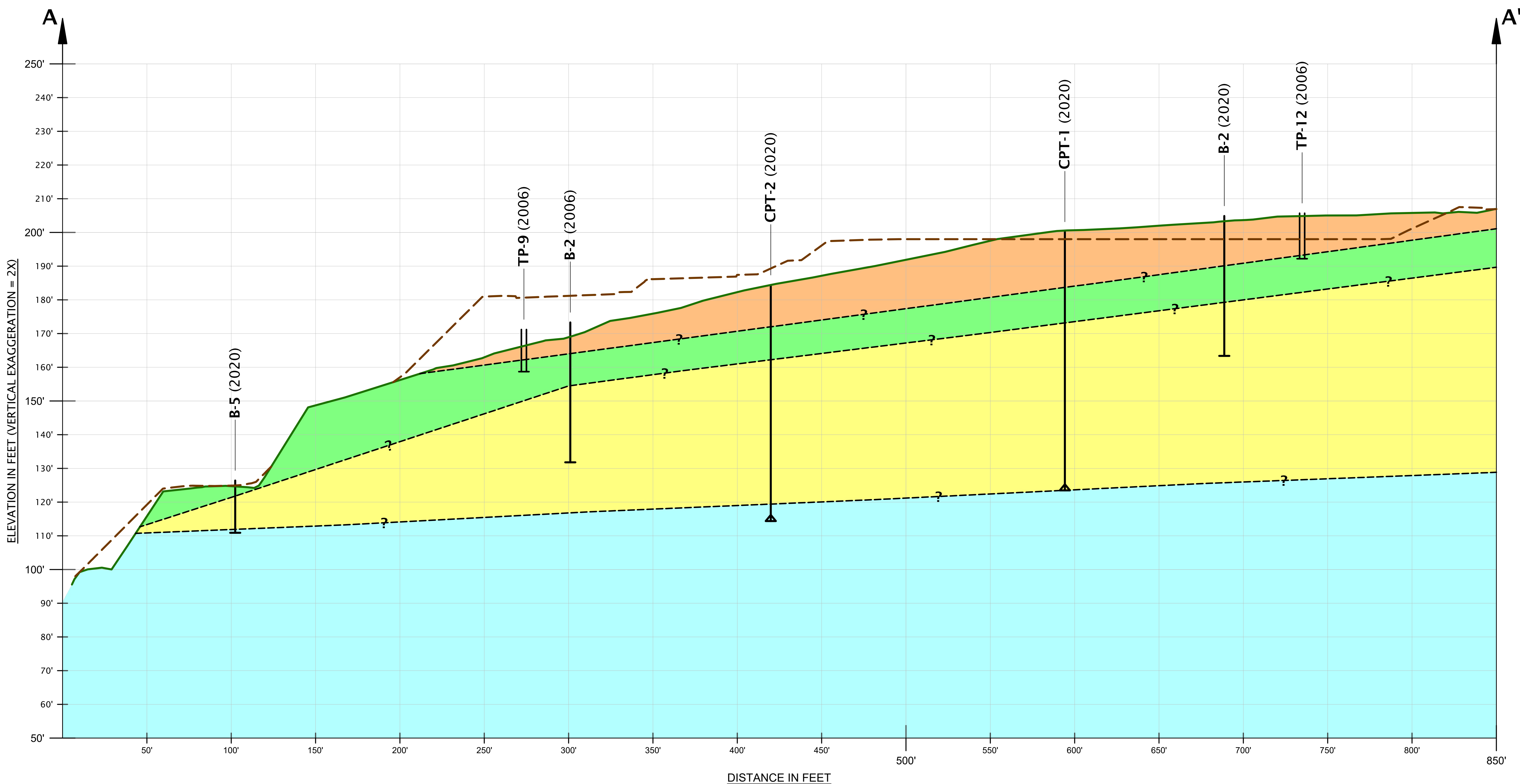

LEGEND:

EXISTING TOPOGRAPHY

PROPOSED TOPOGRAPHY

BORING

CPT

TEST PIT

? ESTIMATED CONTACT BETWEEN STRATIGRAPHIC UNITS

MEDIUM STIFF SILT

LOOSE, SILTY SAND

MEDIUM DENSE SAND

DENSE GRAVEL AND SAND

VERTICAL EXAGGERATION = 2X HORIZONTAL

0 30 60

(VERTICAL SCALE IN FEET)

0 60 120

(HORIZONTAL SCALE IN FEET)

FIGURE 5

CROSS SECTION A-A'

DOLLAR STREET MIDDLE SCHOOL
WEST LINN, OR

WLWSCHDIST-1-01

OCTOBER 2020

GEODESIGN INC
AN NV5 COMPANY

Printed By: mmiller | Print Date: 10/19/2020 11:13:51 AM
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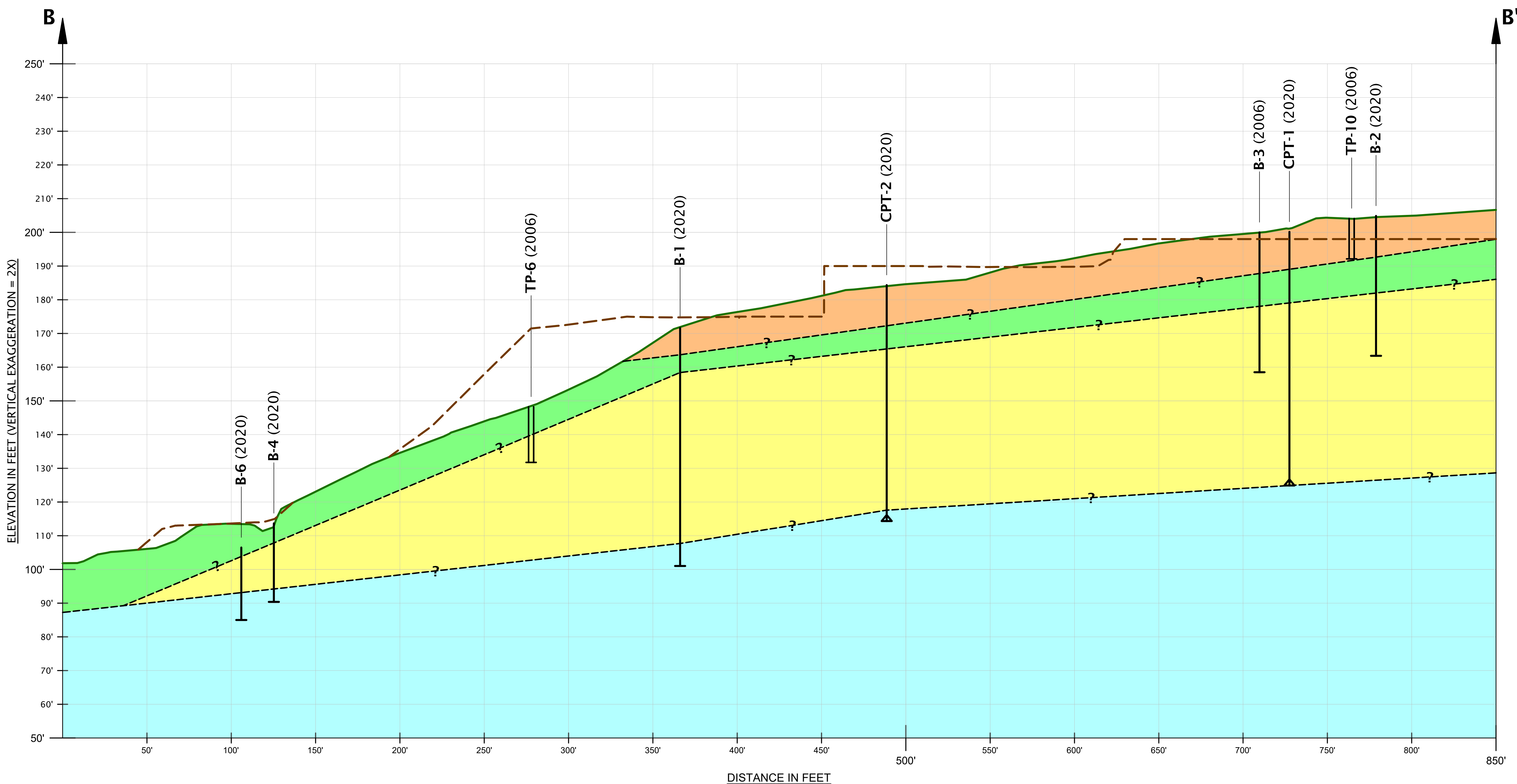

LEGEND:

- EXISTING TOPOGRAPHY
- PROPOSED TOPOGRAPHY
- BORING
- CPT
- TEST PIT
- ? ESTIMATED CONTACT BETWEEN STRATIGRAPHIC UNITS
- MEDIUM STIFF SILT
- LOOSE, SILTY SAND
- MEDIUM DENSE SAND
- DENSE GRAVEL AND SAND

VERTICAL EXAGGERATION = 2X HORIZONTAL

0 30 60
(VERTICAL SCALE IN FEET)

0 60 120
(HORIZONTAL SCALE IN FEET)

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File Name: J:\S-Z\WLWSchDist\WLWSchDist-1\WLWSchDist-1-01\Figures\CAD\WLWSchDist-1-01-SP07.dwg | Layout: FIGURE 6

GEODESIGN INC AN NV5 COMPANY

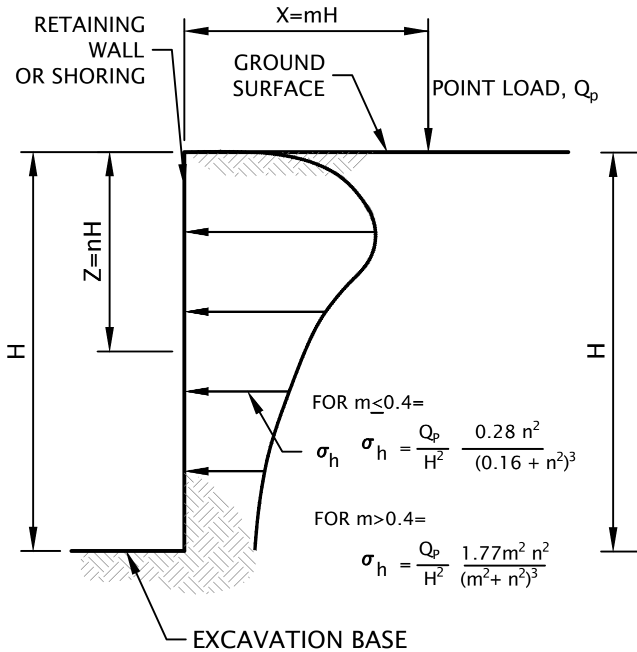
WLWSCHDIST-1-01

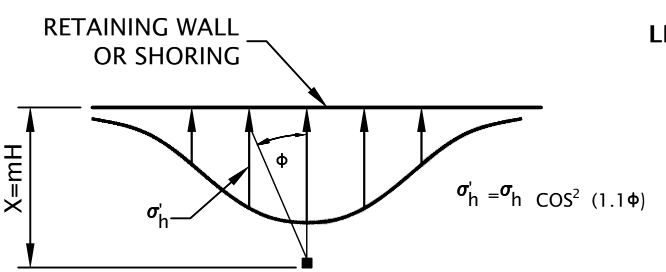
OCTOBER 2020

CROSS SECTION B-B'

DOLLAR STREET MIDDLE SCHOOL
WEST LINN, OR

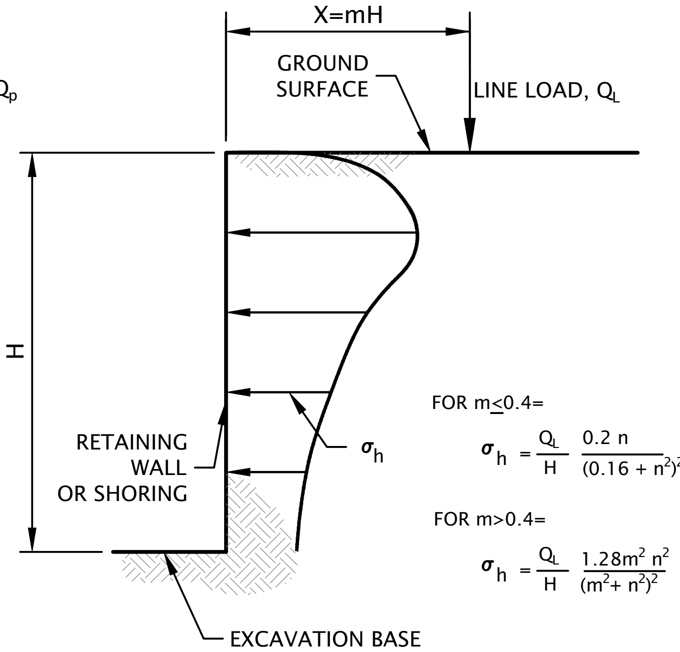
FIGURE 6



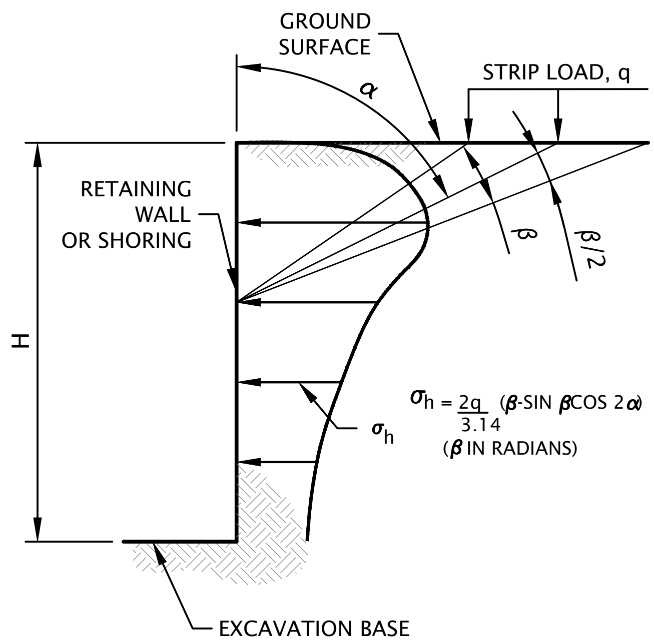


DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD



LINE LOAD PARALLEL TO WALL



STRIP LOAD PARALLEL TO WALL

NOT TO SCALE

NOTES:

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

GEODESIGN INC — AN NV5 COMPANY	WLWSCHDIST-1-01	**SURCHARGE-INDUCED LATERAL EARTH PRESSURES**
	OCTOBER 2020	DOLLAR STREET MIDDLE SCHOOL WEST LINN, OR — **FIGURE 7**

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

Our subsurface exploration program included drilling six borings (B-1 through B-6) to depths between 15.5 and 70.6 feet BGS and excavating three test pits (TP-1 through TP-3) to a depth of 11 feet BGS. Borings B-1 through B-3 and B-6 were drilled using a track-mounted drill rig and mud rotary drilling methods by Western States Soil Conservation, Inc. of Hubbard, Oregon. Borings B-4 and B-5 were drilled using a trailer-mounted drill rig and solid-stem auger drilling methods and the test pits were excavated using a Hitachi mini-tracked excavator by Dan J. Fischer Excavating, Inc. of Forest Grove, Oregon. The test pits were excavated on May 26, 2020 and the borings were drilled on May 27 and 28, 2020. The exploration logs are presented in this appendix. The explorations were observed by members of our geology staff.

Approximate locations of the explorations are shown on Figures 2 through 4. The locations of the explorations were determined using a hand-held GPS or GPS app on a mobile phone. Some locations were adjusted slightly relative to nearby surrounding features. This information should be considered accurate only to the degree implied by the methods used.

SOIL SAMPLING

We collected representative samples of the various soils encountered in the explorations for geotechnical laboratory testing. Sampling methods and intervals are shown on the exploration logs. Soil samples were collected from the borings using the one of following methods:

- 1.5-inch-inside diameter, split-spoon sampler (SPT sampler),
- 3-inch-outside diameter, split-spoon sampler (Dames & Moore sampler), or
- Shelby tubes

The split-spoon sampling was conducted in general accordance with ASTM D1586. The 1.5-inch-inside diameter, split-spoon samplers and 3-inch- outside diameter, split-spoon (Dames & Moore) samplers were driven into the soil with 140-pound hammer free-falling 30 inches. The samplers were driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded in the boring logs, unless otherwise noted.

Grab samples were collected from the test pit walls and/or base using the excavator bucket.

The average efficiency of the automatic SPT hammer used by Western States Soil Conservation, Inc. was 85.6 percent. The calibration testing results are presented at the end of this appendix. The SPTs completed by Dan J. Fischer Excavating, Inc. were conducted using two wraps of the rope around the cathead.

SOIL CLASSIFICATION

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

LABORATORY TESTING

Laboratory testing was conducted on select soil samples to confirm field classifications and determine the index engineering properties and strength characteristics. Locations of the tested samples are shown on the exploration logs. Descriptions of the testing completed are presented below.

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

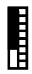
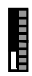
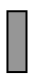
MOISTURE CONTENT
We tested the natural moisture content of select soil samples in general accordance with ASTM D2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

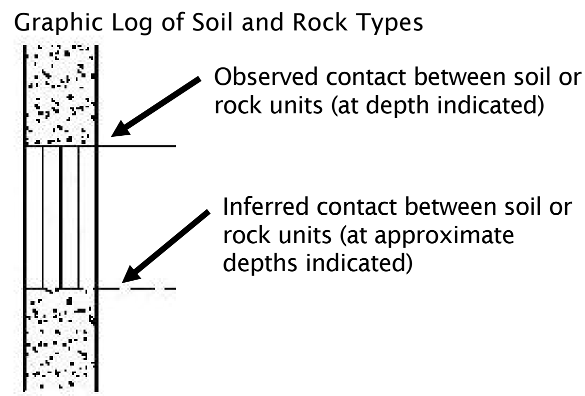
ATTERBERG LIMITS TESTING
Atterberg limits (plastic and liquid limits) testing was performed on a select soil sample in general accordance with ASTM D4318. The plastic limit is defined as the moisture content where the soil becomes brittle. The liquid limit is defined as the moisture content where the soil begins to act similar to a liquid. The plasticity index is the difference between the liquid and plastic limits. The test results are presented in this appendix.

CONSOLIDATION TESTING
We performed one-dimensional consolidation tests on relatively undisturbed soil samples in general accordance with ASTM D2435. The test measures the volume change of a soil sample under predetermined load increases. The consolidation test results are presented in this appendix.

PARTICLE-SIZE ANALYSIS
Particle-size analysis was performed on select soil samples to determine the distribution of soil particle sizes. The testing consisted of percent fines determination (percent passing the U.S. Standard No. 200 sieve) analyses completed in general accordance with ASTM D1140. The test results are presented in this appendix.

DIRECT SHEAR TESTING
Direct shear testing was performed on select soil samples in general accordance with ASTM D3080. The test measures the shear strength of a sample at three different normal pressure values. The results are plotted to provide an estimate of cohesion and friction angle of the soil. The test results are presented in this appendix.

SYMBOL	SAMPLING DESCRIPTION
	Location of sample collected in general accordance with ASTM D1586 using Standard Penetration Test with recovery
	Location of sample collected using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D1587 with recovery
	Location of sample collected using Dames & Moore sampler and 300-pound hammer or pushed with recovery
	Location of sample collected using Dames & Moore sampler and 140-pound hammer or pushed with recovery
	Location of sample collected using 3-inch-O.D. California split-spoon sampler and 140-pound hammer with recovery
	Location of grab sample
	Rock coring interval
	Water level during drilling
	Water level taken on date shown



GEOTECHNICAL TESTING EXPLANATIONS

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

ENVIRONMENTAL TESTING EXPLANATIONS

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

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very Loose	0 – 4	0 – 11	0 – 4
Loose	4 – 10	11 – 26	4 – 10
Medium Dense	10 – 30	26 – 74	10 – 30
Dense	30 – 50	74 – 120	30 – 47
Very Dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOIL

Consistency	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very Soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 – 4	3 – 6	2 – 5	0.25 – 0.50
Medium Stiff	4 – 8	6 – 12	5 – 9	0.50 – 1.0
Stiff	8 – 15	12 – 25	9 – 19	1.0 – 2.0
Very Stiff	15 – 30	25 – 65	19 – 31	2.0 – 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOIL (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVEL WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SAND (<5% fines)	SW or SP	SAND
		SAND WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SAND WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOIL (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOIL			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.

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

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0	Medium stiff, brown SILT (ML), trace sand; moist (6-inch-thick root zone).				Aboveground monument with 4.2 feet of stickup and 1.5 feet of concrete backfill.
				6	1-inch tremie pipe Cement
5	sandy at 5.0 feet			6	
				6	
10			P200	6	P200 = 57%
	Medium dense, gray-brown SAND with silt (SP-SM); moist, sand is fine.	13.0			
15				11	
	Medium dense, brown, silty SAND (SM); moist, sand is fine.	18.0			
20				11	
25				24	
	Stiff, brown, sandy SILT (ML); moist, sand is fine.	28.0			
30			P200	14	P200 = 60%
35	Medium dense, brown, silty SAND (SM); moist, sand is fine.	35.0	DD DS	30	DD = 104 pcf
40					

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

0 50 100

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: L. Gose

COMPLETED: 05/27/20

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

GEODESIGN INC AN NV5 COMPANY	WLWSCHDIST-1-01	**BORING B-1**
	OCTOBER 2020	DOLLAR STREET MIDDLE SCHOOL WEST LINN, OR — **FIGURE A-1**

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
40	(continued from previous page)			14	
	Medium dense, brown, silty SANd (SM); moist, sand is fine.	42.5			
45			P200	24	P200 = 13%
50				53	
55				23	
	Very stiff, light brown SILT (ML), trace sand; moist.	58.5			
60				25	
					Mud loss at 63.0 feet. Rig chatter at 64.0 feet. Driller Comment: gravel at 64.0 feet.
	Dense, gray GRAVEL with silt and sand (GP-GM); moist, sand is fine to coarse.	64.0			
65				45	
					Vibrating wire piezometer #2000365 set at 68.0 feet
70	very dense; moist to wet at 70.0 feet			30-50/1"	70.0 feet during drilling
	Exploration completed at a depth of 70.6 feet. Hammer efficiency factor is 85.6 percent.	70.6			Groundwater below depth of piezometer on 7/20/2020. Surface elevation was not measured at the time of exploration.
75					
80					

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100)

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: L. Gose COMPLETED: 05/27/20

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

GEODESIGN INC AN NV5 COMPANY

WLWSCHDIST-1-01

OCTOBER 2020

BORING B-1
(continued)

DOLLAR STREET MIDDLE SCHOOL
WEST LINN, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0	Soft to medium stiff, brown SILT (ML), minor sand, trace organics (charcoal); moist, sand is fine (5-inch-thick root zone).			4	
5	light brown, sandy, without organics at 5.0 feet			4	
	gray-brown at 7.5 feet			4	
10				5	
	Loose, gray, silty SAND (SM); moist, sand is fine.	12.0			
			DD CON	P	DD = 83 pcf
15				7	
20			P200	7	P200 = 39%
25	loose to medium dense at 25.0 feet			10	
	Medium dense, gray SAND with silt (SP-SM); moist, sand is fine.	28.0			
30				13	
35				18	
40					

▲ BLOW COUNT
● MOISTURE CONTENT %
RQD% CORE REC%
0 50 100

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: L. Gose
COMPLETED: 05/27/20

BORING METHOD: mud rotary (see document text)
BORING BIT DIAMETER: 4 7/8 inches

GEODESIGN INC
AN NV5 COMPANY

WLWSCHDIST-1-01

OCTOBER 2020

BORING B-2

DOLLAR STREET MIDDLE SCHOOL
WEST LINN, OR

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
40		(continued from previous page)				15	
		Exploration completed at a depth of 41.5 feet. Hammer efficiency factor is 85.6 percent.	41.5				Surface elevation was not measured at the time of exploration.
45							
50							
55							
60							
65							
70							
75							
80							

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: L. Gose COMPLETED: 05/27/20

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0	Stiff, light brown, sandy SILT (ML); moist, sand is fine.			9	
5			DD CON	10	DD = 75 pcf
10			DD DS	8	DD = 81 pcf
14	Loose, dark gray-brown, silty SAND (SM); moist, sand is fine to medium.	14.0		8	
20	medium dense at 20.0 feet			13	
25				13	
28	Medium stiff, light brown SILT with sand (ML); moist, sand is fine.	28.0		6	
33.5	Medium dense, light brown-gray, silty SAND (SM); moist, sand is fine.	33.5	P200	20	P200 = 32%
40					

Legend: ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0–50–100)

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: L. Gose COMPLETED: 05/28/20

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

GEODESIGN INC — AN NV5 COMPANY

WLWSCHDIST-1-01

OCTOBER 2020

BORING B-3

DOLLAR STREET MIDDLE SCHOOL
WEST LINN, OR

FIGURE A-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	BLOW COUNT	INSTALLATION AND COMMENTS
40	(continued from previous page)		28	
45	gray at 45.0 feet		18	
50.3	Very dense, dark gray GRAVEL with silt and sand (GP-GM); moist, sand is fine to coarse.	50.3	52	
55	moist to wet at 55.0 feet		50	
56.5	Exploration completed at a depth of 56.5 feet. Hammer efficiency factor is 85.6 percent.	56.5		Surface elevation was not measured at the time of exploration.

Column headings: DEPTH FEET; GRAPHIC LOG; MATERIAL DESCRIPTION; ELEVATION DEPTH; TESTING; SAMPLE; ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0, 50, 100); INSTALLATION AND COMMENTS

Depth scale: 40, 45, 50, 55, 60, 65, 70, 75, 80

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: L. Gose

COMPLETED: 05/28/20

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

GEODESIGN INC — AN NV5 COMPANY

WLWSCHDIST-1-01

OCTOBER 2020

BORING B-3
(continued)

DOLLAR STREET MIDDLE SCHOOL
WEST LINN, OR

FIGURE A-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	MOISTURE CONTENT %	INSTALLATION AND COMMENTS
0	Medium dense, brown GRAVEL with silt and sand (GP-GM); moist - **FILL**.					
2.0	Medium stiff, brown, sandy SILT (ML); moist, sand is fine.	2.0		7		
5	brown-orange at 5.0 feet		P200	5	●	Infiltration test at 5.0 feet. P200 = 53%
7.5	Loose to medium dense, brown, silty SAND (SM); moist, sand is fine to medium.	7.5		10		
10	medium dense at 10.0 feet		P200	16	●	P200 = 15%
15				16		
20				23	●	⊻ 20.0 feet during drilling
22.0	Very dense, brown GRAVEL with sand (GP), trace silt; moist to wet.	22.0		50/3"		Gravel chatter at 22.0 feet.
23.3	Exploration terminated at a depth of 23.3 feet due to practical refusal. SPT completed using two wraps with a cathead.	23.3				No free water observed after drilling. Surface elevation was not measured at the time of exploration.

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

0 50 100

DRILLED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: J. Heidgerken COMPLETED: 05/27/20

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

GEODESIGN INC AN NV5 COMPANY	WLWSCHDIST-1-01	**BORING B-4**
	OCTOBER 2020	DOLLAR STREET MIDDLE SCHOOL WEST LINN, OR — **FIGURE A-4**

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	BLOW COUNT	INSTALLATION AND COMMENTS
0	Medium dense, brown-gray GRAVEL with silt and sand (GP-GM); moist - **FILL**.			
	Medium stiff, light brown, sandy SILT (ML); moist, sand is fine.	2.0	7	
5	Medium dense, brown-gray SAND with silt (SP-SM); moist, sand is fine.	5.0	13	
			12	
10			12	
	Medium dense to dense, gray-brown GRAVEL with silt and sand (GP-GM); moist.	13.5	28	Drill chatter at 13.5 feet.
15	Exploration completed at a depth of 15.5 feet due to practical refusal. SPT completed using two wraps with a cathead.	15.5		Surface elevation was not measured at the time of exploration.
20				
25				
30				
35				
40				

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

0 50 100

DRILLED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: J. Heidgerken COMPLETED: 05/27/20

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

GEODESIGN INC — AN NV5 COMPANY

WLWSCHDIST-1-01	**BORING B-5**	
OCTOBER 2020	DOLLAR STREET MIDDLE SCHOOL WEST LINN, OR	**FIGURE A-5**

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
0	Soft, brown SILT with sand (ML); moist, sand is fine (topsoil to 12 inches, 4-inch-thick root zone) - **FILL**.					
					2	
5	medium stiff, with gravel at 4.5 feet				5	Mud loss from 5.0 to 10.0 feet.
					5	
10	without gravel at 10.0 feet				5	
	Loose, brown, silty SAND (SM); moist, sand is fine.	13.5				
15					25	Hole cased to 15.0 feet.
	Medium dense, dark gray GRAVEL with silt and sand (GP-GM); moist.	16.0				
20	dense at 20.0 feet				42	
	Exploration completed at a depth of 21.5 feet. Hammer efficiency factor is 85.6 percent.	21.5				Surface elevation was not measured at the time of exploration.
25						
30						
35						
40						

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: L. Gose COMPLETED: 05/28/20

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 4 7/8 inches

GEODESIGN INC AN NV5 COMPANY	WLWSCHDIST-1-01	BORING B-6	
	OCTOBER 2020	DOLLAR STREET MIDDLE SCHOOL WEST LINN, OR	**FIGURE A-6**

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT (%) 0 50 100	COMMENTS

TP-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	MOISTURE CONTENT (%)	COMMENTS
0.0	Medium stiff, brown SILT (ML), trace sand and organics (roots); moist, organics are up to 2-inch diameter (topsoil to 12 inches, 5-inch-thick root zone).			☒		
2.5	without organics at 2.5 feet					
	minor clay at 4.0 feet		ATT PP	☒	●	PP = 0.5 tsf LL = 36% PL = 21%
5.0	light brown, sandy at 4.5 feet		P200	☒	●	Infiltration test at 5.0 feet. P200 = 59%
7.5	Loose to medium dense, light brown, silty SAND (SM); moist, sand is fine.	7.5		☒		
10.0						
	Exploration completed at a depth of 11.0 feet.	11.0		☒	●	No groundwater seepage observed to the depth explored. No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
12.5						

TP-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	MOISTURE CONTENT (%)	COMMENTS
0.0	Medium stiff, dark brown SILT (ML), trace sand and organics (roots); moist, organics are less than 0.5-inch diameter (topsoil to 12 inches, 5-inch-thick root zone).			☒		
2.5	without organics at 2.0 feet					
	dark brown with orange mottles, sandy at 3.5 feet		PP	☒		PP = 0.5 tsf
5.0			P200 PP	☒	●	Infiltration test at 6.0 feet. P200 = 70% PP = 0.5 tsf
7.5	Medium dense, brown, silty SAND (SM); moist, sand is fine.	8.0		☒		
10.0						
	Exploration completed at a depth of 11.0 feet.	11.0		☒	●	No groundwater seepage observed to the depth explored. No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
12.5						

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: L. Gose COMPLETED: 05/26/20

EXCAVATION METHOD: mini excavator (see document text)

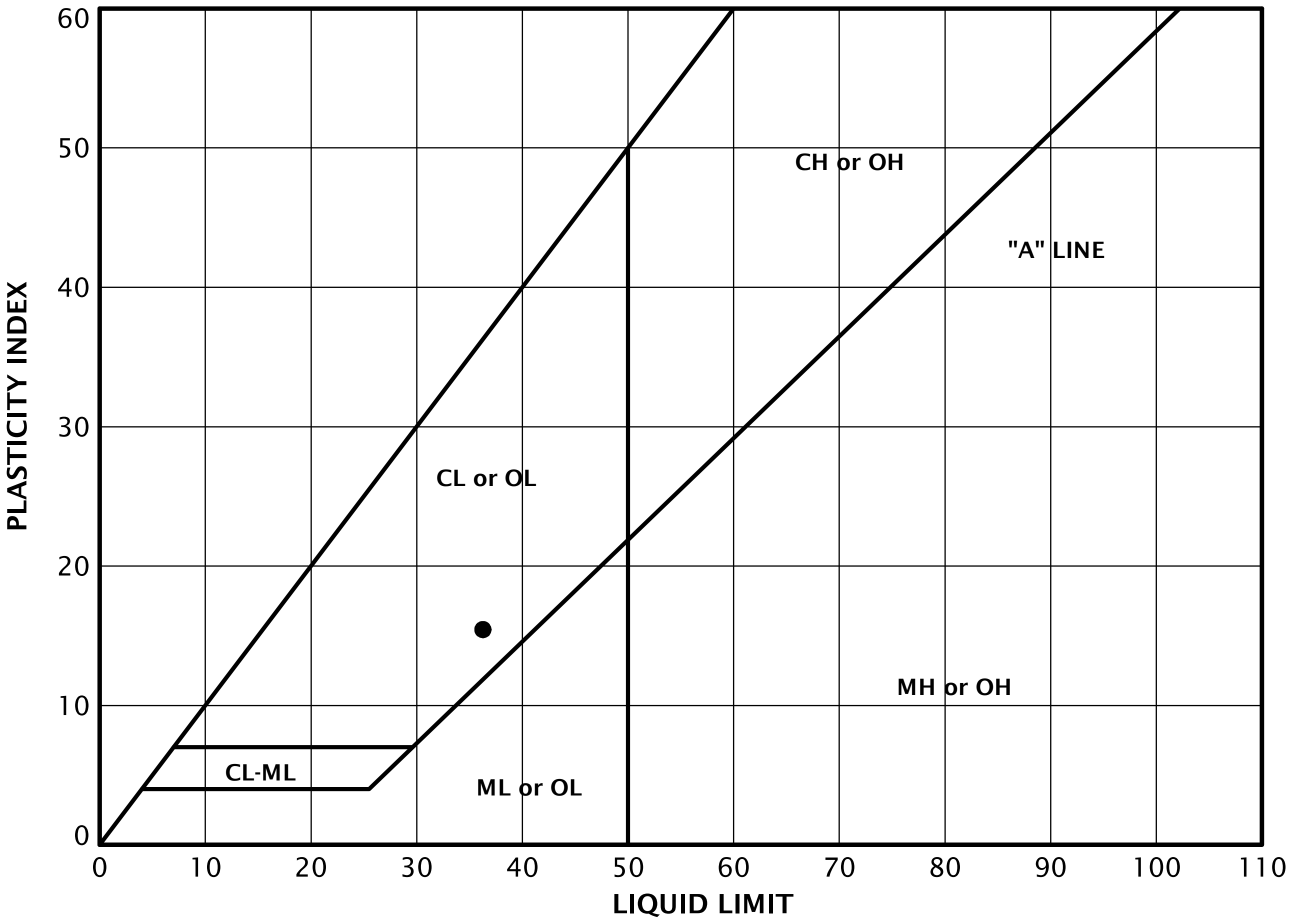
GEODESIGN INC — AN NV5 COMPANY	WLWSCHDIST-1-01	**TEST PIT**
	OCTOBER 2020	DOLLAR STREET MIDDLE SCHOOL WEST LINN, OR — **FIGURE A-7**

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT (%)	COMMENTS
TP-3						0 50 100	
0.0		Medium stiff, dark brown SILT (ML), trace clay, sand, and organics (rootlets); moist (topsoil to 12 inches, 5-inch-thick root zone).			☒		
2.5		light brown, sandy at 3.0 feet		PP	☒	●	PP = 0.5 tsf
5.0							
7.5		minor sand at 7.5 feet		P200	☒	●	Infiltration test at 8.0 feet. P200 = 91%
10.0							
		Exploration completed at a depth of 11.0 feet.	11.0		☒		No groundwater seepage observed to the depth explored. No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
12.5						0 50 100	

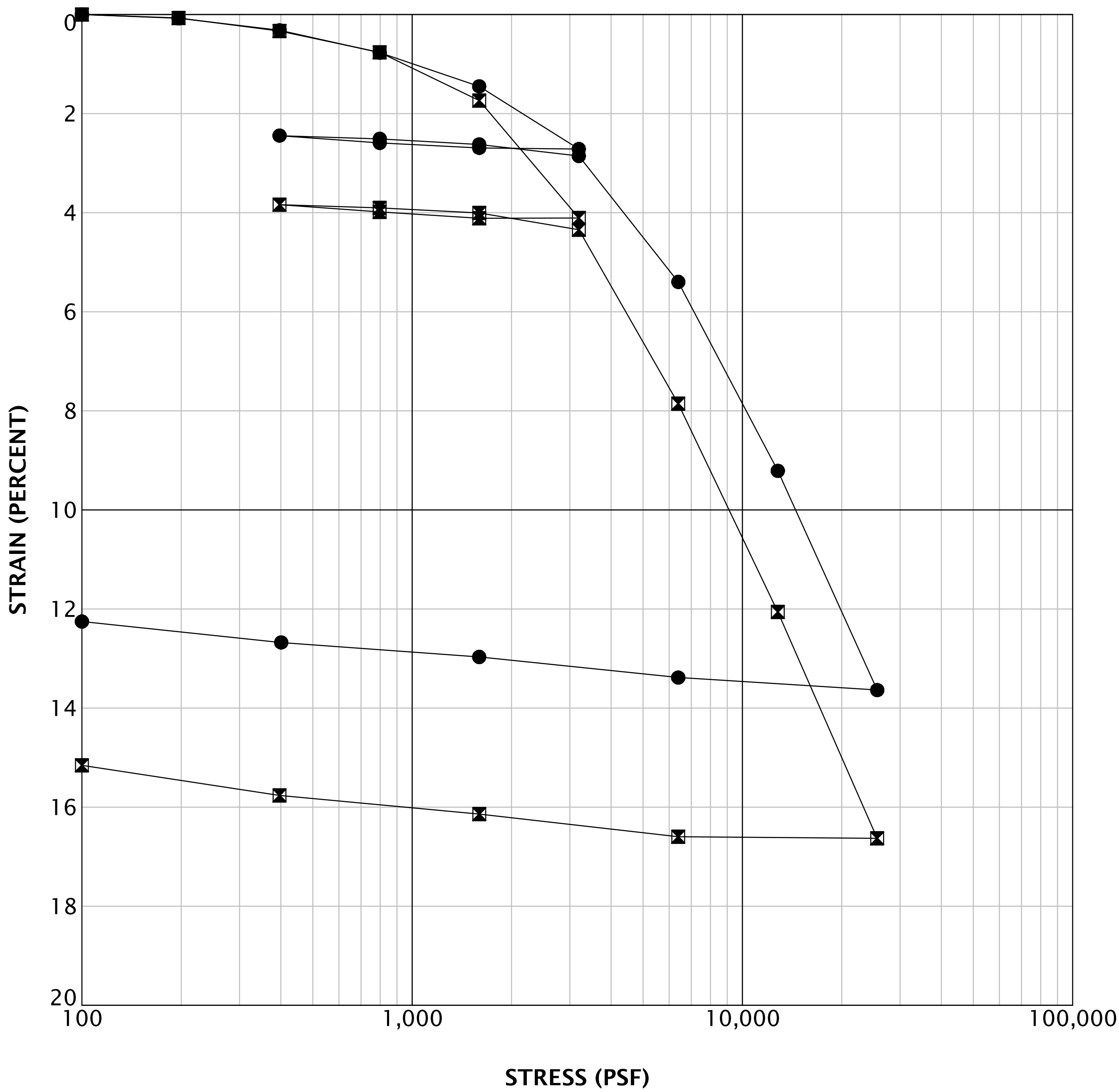
EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: L. Gose COMPLETED: 05/26/20

EXCAVATION METHOD: mini excavator (see document text)

GEODESIGN INC AN NV5 COMPANY	WLWSCHDIST-1-01	**TEST PIT**	
	OCTOBER 2020	DOLLAR STREET MIDDLE SCHOOL WEST LINN, OR	**FIGURE A-8**



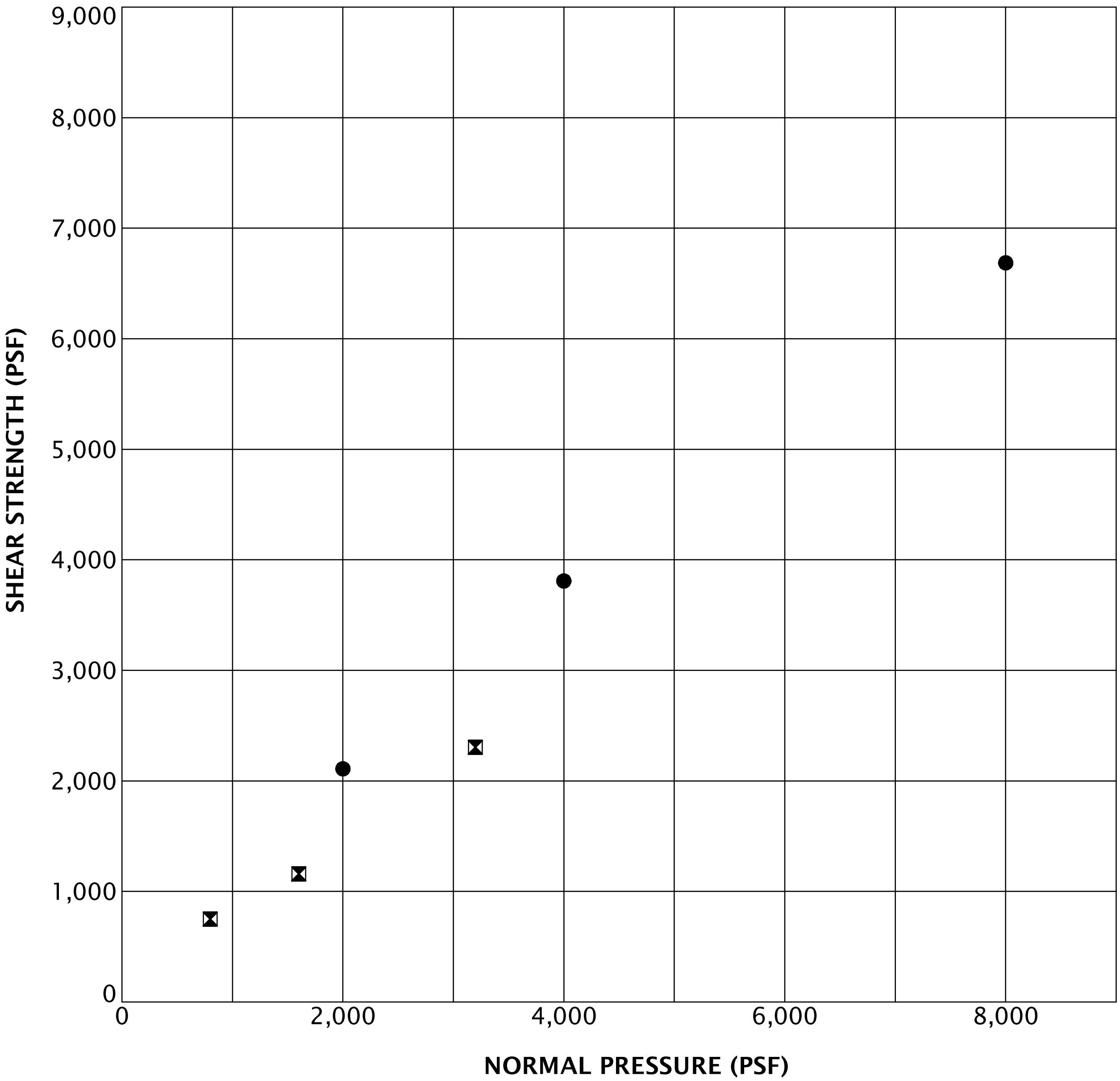
KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	TP-1	4.0	24	36	21	15



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)
●	B-2	13.0	27	83
⊠	B-3	5.0	26	75

GEODESIGN INC
AN NV5 COMPANY

WLWSCHDIST-1-01	CONSOLIDATION TEST RESULTS	
OCTOBER 2020	DOLLAR STREET MIDDLE SCHOOL WEST LINN, OR	FIGURE A-10



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SOAKED
●	B-1	35.0	12	105	YES
⊠	B-3	10.0	27	82	YES

GEODESIGN INC
AN NV5 COMPANY

WLWSCHDIST-1-01	**DIRECT SHEAR TEST RESULTS**	
OCTOBER 2020	DOLLAR STREET MIDDLE SCHOOL WEST LINN, OR	**FIGURE A-11**

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	2.5		37							
B-1	7.5		32							
B-1	10.0		31				57			
B-1	15.0		23							
B-1	20.0		29							
B-1	30.0		24				60			
B-1	35.0		12	104						
B-1	40.0		27							
B-1	45.0		21				13			
B-1	55.0		22							
B-1	60.0		43							
B-1	65.0		9							
B-2	2.5		35							
B-2	10.0		26							
B-2	13.0		27	83						
B-2	20.0		22				39			
B-2	25.0		28							
B-2	30.0		23							
B-2	40.0		11							
B-3	2.5		27							
B-3	5.0		26	75						
B-3	10.0		30	81						
B-3	15.0		19							
B-3	25.0		22							
B-3	30.0		31							
B-3	35.0		22				32			
B-3	45.0		23							

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-3	50.0		11							
B-4	5.0		28				53			
B-4	10.0		18				15			
B-4	20.0		31							
B-5	2.5		20							
B-5	7.5		12							
B-5	14.0		18							
B-6	2.5		25							
B-6	5.0		28							
B-6	10.0		30							
B-6	15.0		14							
B-6	20.0		23							
TP-1	4.0		24					36	21	15
TP-1	5.0		28				59			
TP-1	11.0		17							
TP-2	6.0		27				70			
TP-2	11.0		22							
TP-3	3.0		33							
TP-3	8.0		34				91			

Summary of SPT Test Results

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

EMX: Maximum Energy

ETR: Energy Transfer Ratio - Rated

Start Depth ft	Final Depth ft	**N Value**	**N60 Value**	Average EMX ft-lb	Average ETR %
40.00	41.50	**23**	**32**	307.27	87.8
42.50	44.00	**24**	**34**	294.99	84.3
45.00	46.50	**28**	**39**	296.53	84.7
47.50	49.00	**19**	**27**	296.50	84.7
50.00	51.50	**15**	**21**	305.07	87.2
			Overall Average Values:	299.63	85.6
			Standard Deviation:	7.50	2.1
			Overall Maximum Value:	320.59	91.6
			Overall Minimum Value:	281.10	80.3

APPENDIX B

APPENDIX B

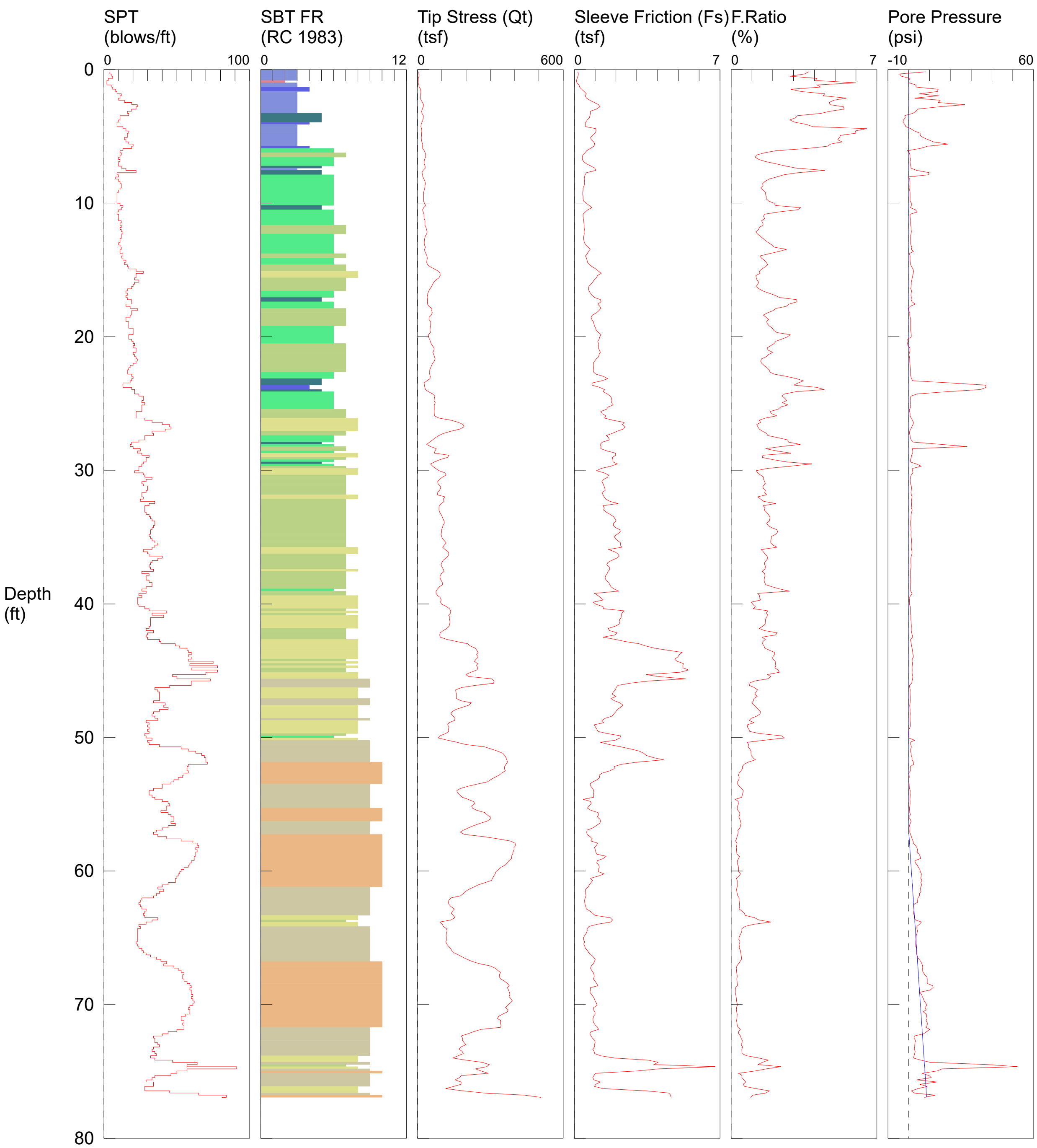
CONE PENETRATION TESTING

Three CPT probes (CPT-1 through CPT-3) were advanced to depths between 69.9 and 90.2 feet BGS. Figures 2 through 4 show the locations of the CPT probes relative to existing site features. The CPTs were performed in general accordance with ASTM D5778 by Oregon Geotechnical Explorations, Inc. of Keizer, Oregon, on May 27, 2020. This CPT logs are presented in this appendix.

The CPT is an in situ test that provides assistance in characterizing subsurface stratigraphy. The test includes advancing a 35.6-millimeter-diameter cone equipped with a load cell, friction sleeve, strain gages, porous stone, and geophone through the soil profile. The cone is advanced at a rate of approximately 2 centimeters per second. Tip resistance, sleeve friction, and pore pressure at are typically recorded at 0.1-meter intervals. Shear wave velocity of the subsurface soil was also measured at 1-meter intervals in CPT-1.

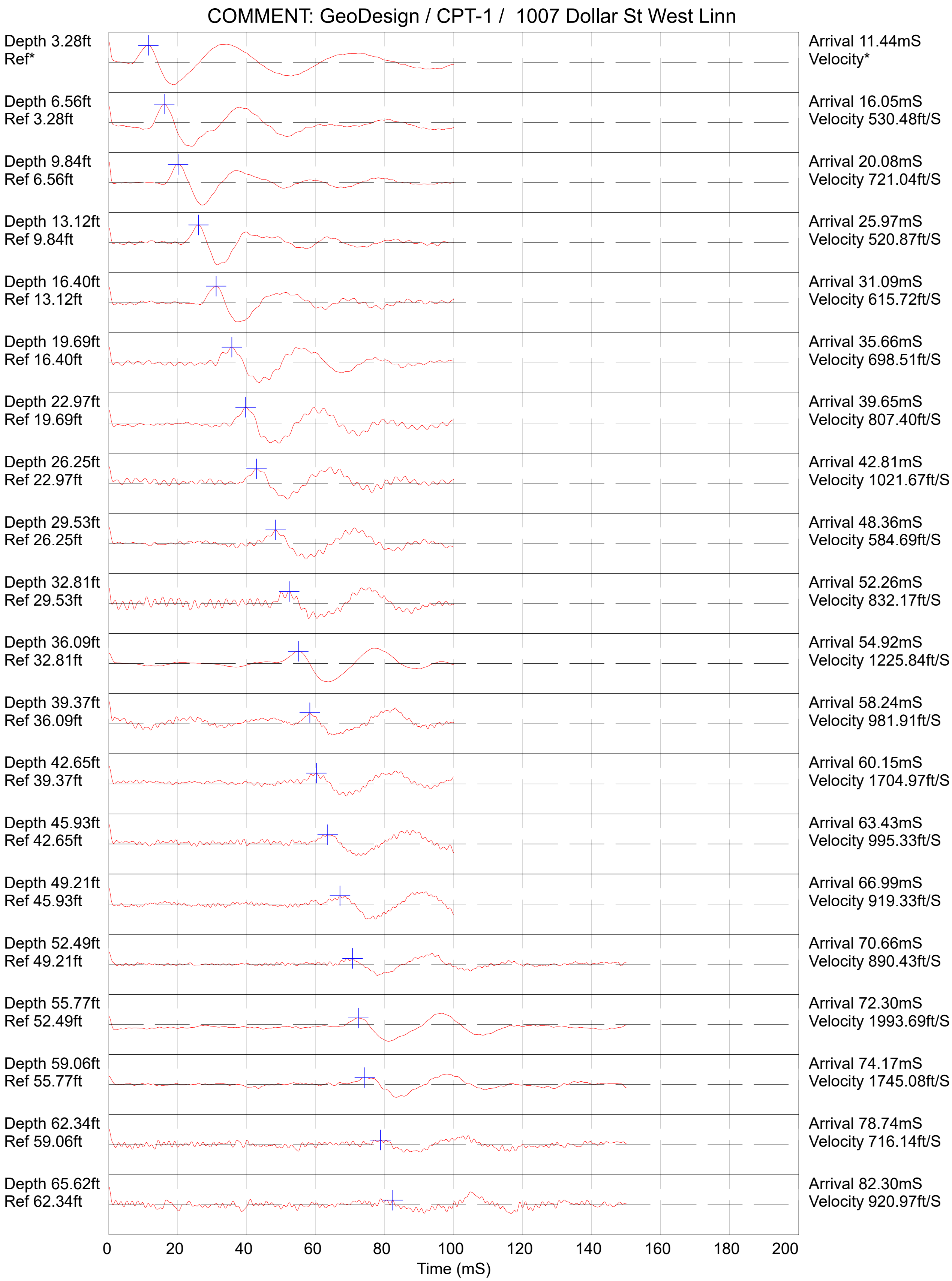
GeoDesign / CPT-1 / 1007 Dollar St West Linn

OPERATOR: OGE DMM
CONE ID: DSG0709
HOLE NUMBER: CPT-1
TEST DATE: 5/27/2020 8:49:02 AM
TOTAL DEPTH: 76.936 ft



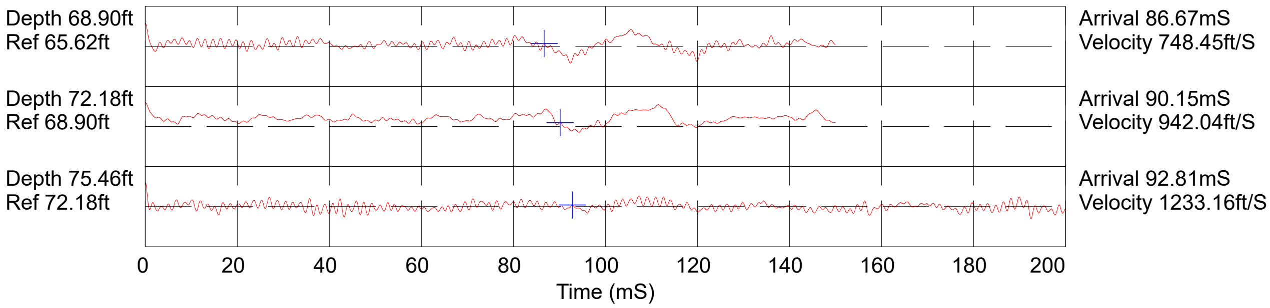
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983



Hammer to Rod String Distance (ft): 4.27
* = Not Determined

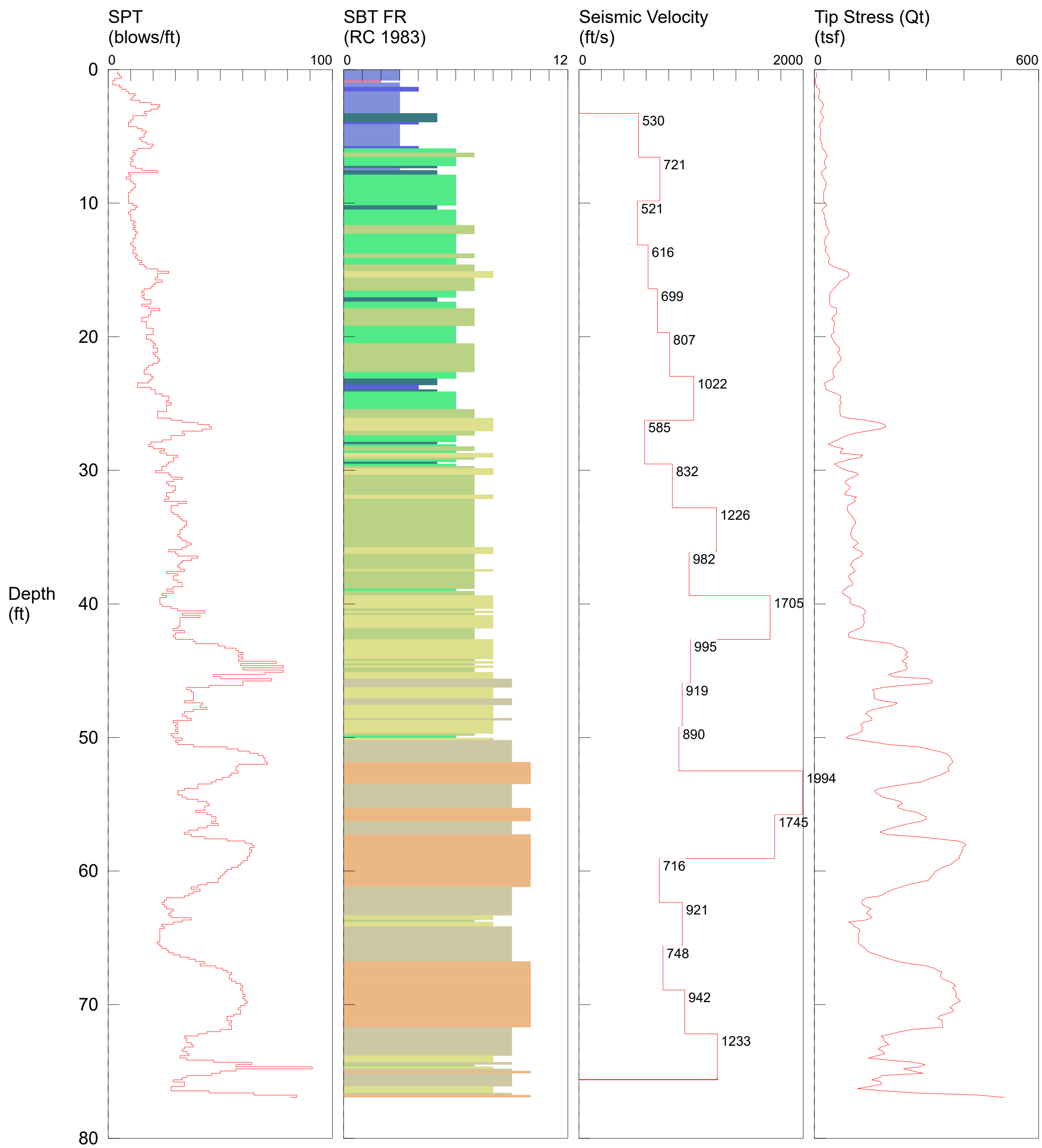
COMMENT: GeoDesign / CPT-1 / 1007 Dollar St West Linn



Hammer to Rod String Distance (ft): 4.27
* = Not Determined

GeoDesign / CPT-1 / 1007 Dollar St West Linn

OPERATOR: OGE DMM
CONE ID: DSG0709
HOLE NUMBER: CPT-1
TEST DATE: 5/27/2020 8:49:02 AM
TOTAL DEPTH: 76.936 ft

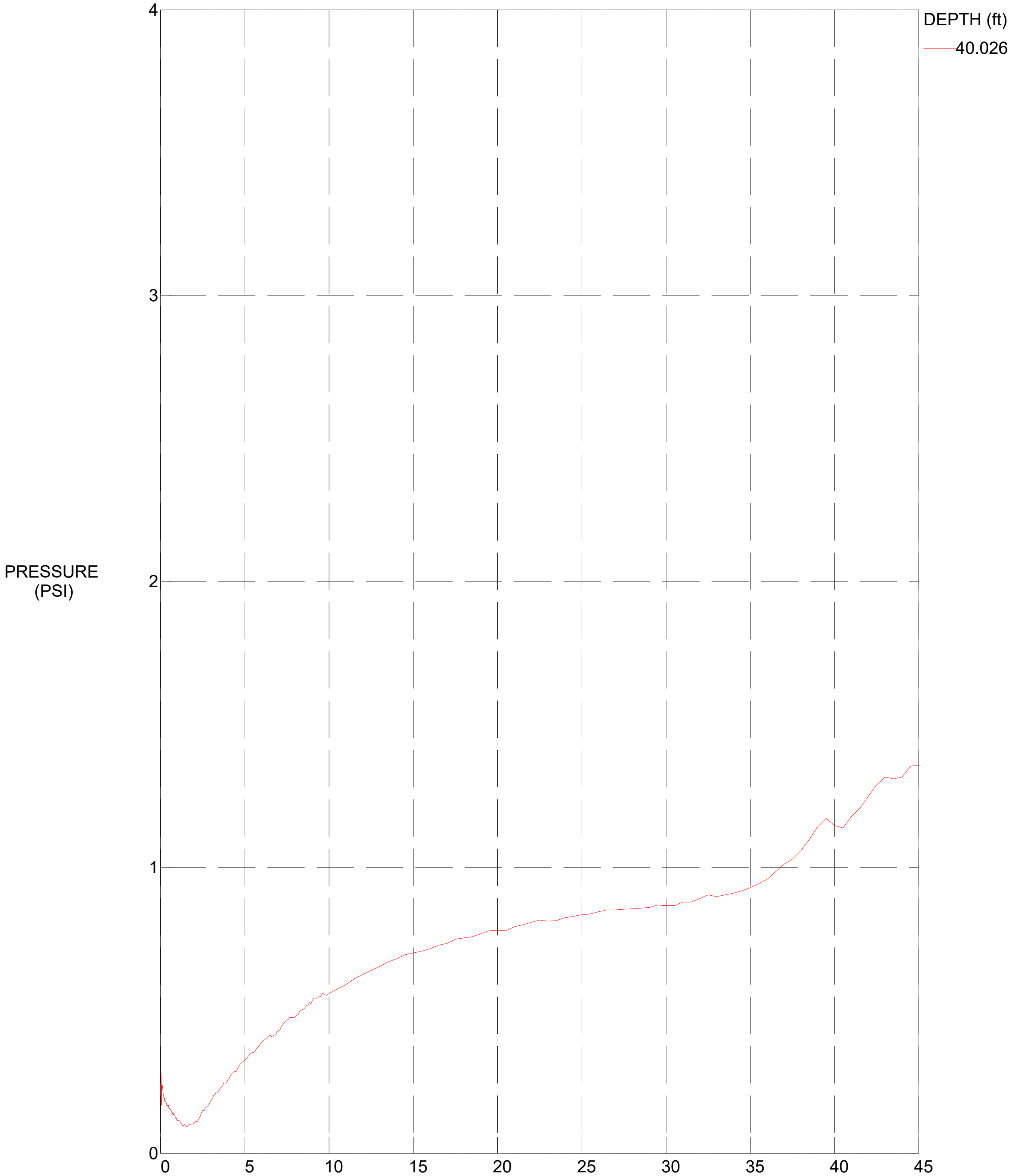


1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GeoDesign / CPT-1 / 1007 Dollar St West Linn

TEST DATE:

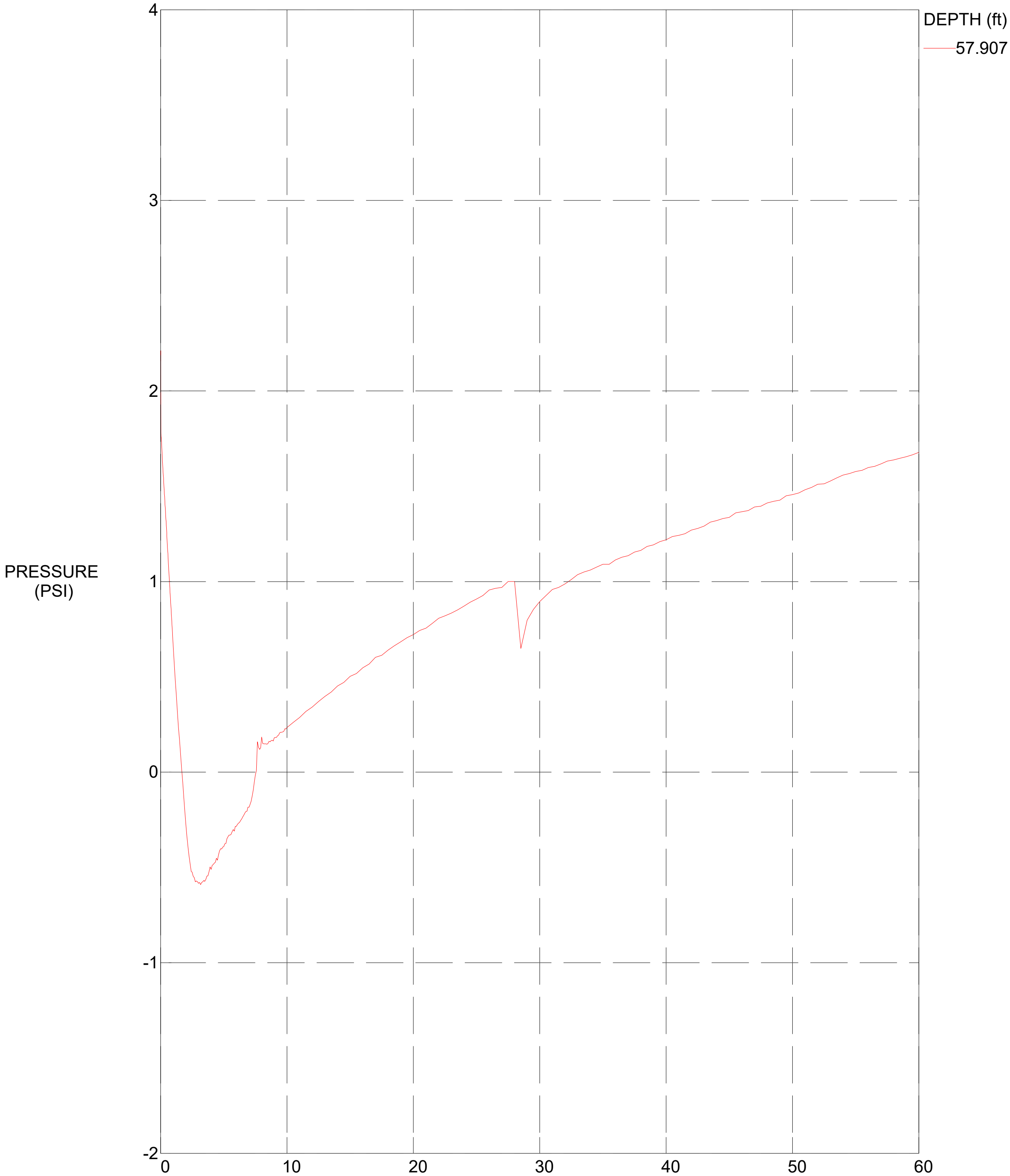


MAXIMUM PRESSURE = 1.358 (PSI)   TIME: (MINUTES)

HYDROSTATIC PRESSURE = 0.0 (PSI), WATER TABLE: 57.09 ft

COMMENT: GeoDesign / CPT-1 / 1007 Dollar St West Linn

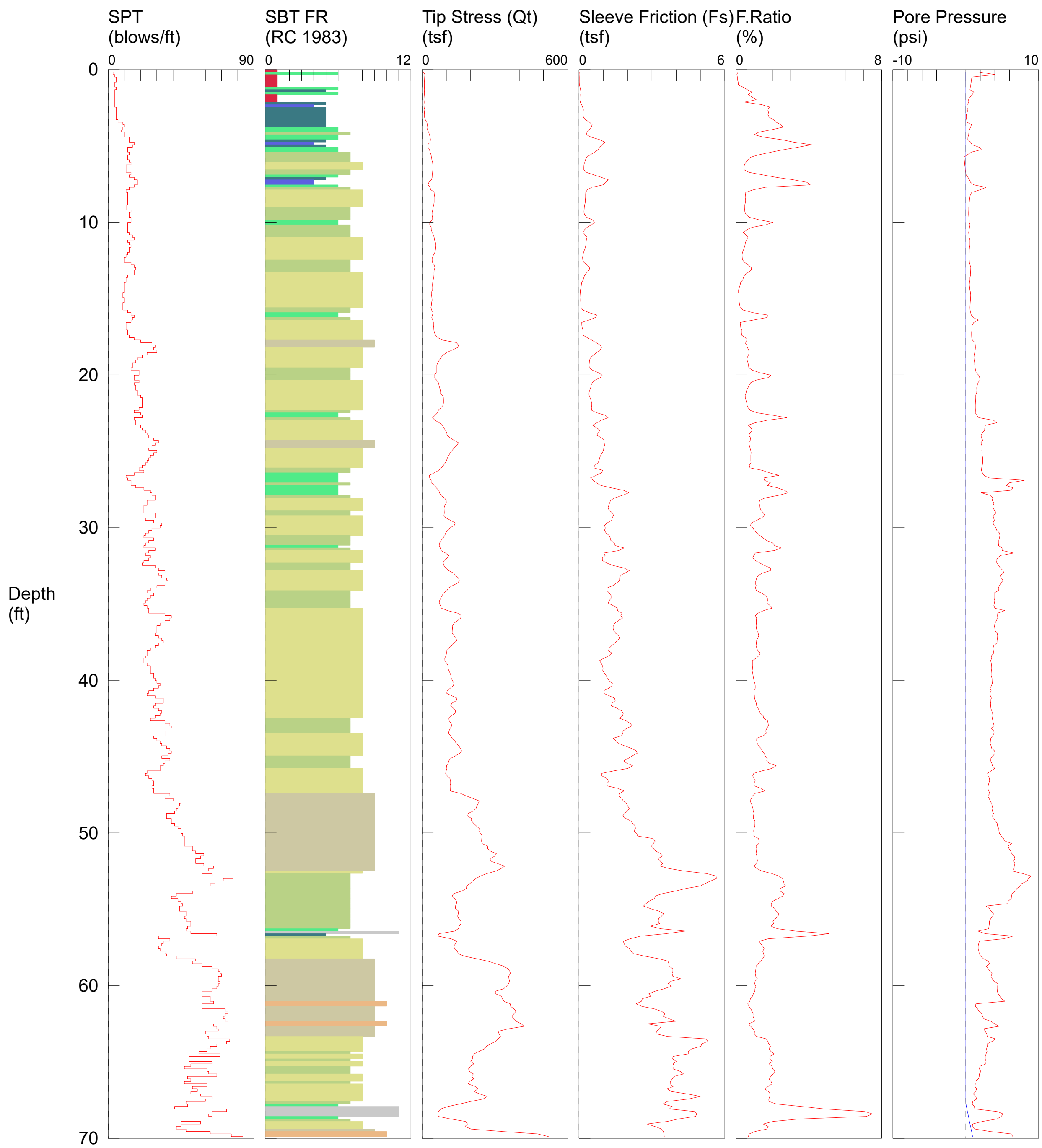
TEST DATE:



MAXIMUM PRESSURE = 2.211 (PSI)   TIME: (MINUTES)
HYDROSTATIC PRESSURE = 0.355 (PSI), WATER TABLE: 57.09 ft

GeoDesign / CPT-2 / 1007 Dollar St West Linn

OPERATOR: OGE DMM
CONE ID: DSG0709
HOLE NUMBER: CPT-2
TEST DATE: 5/27/2020 12:07:07 PM
TOTAL DEPTH: 69.882 ft

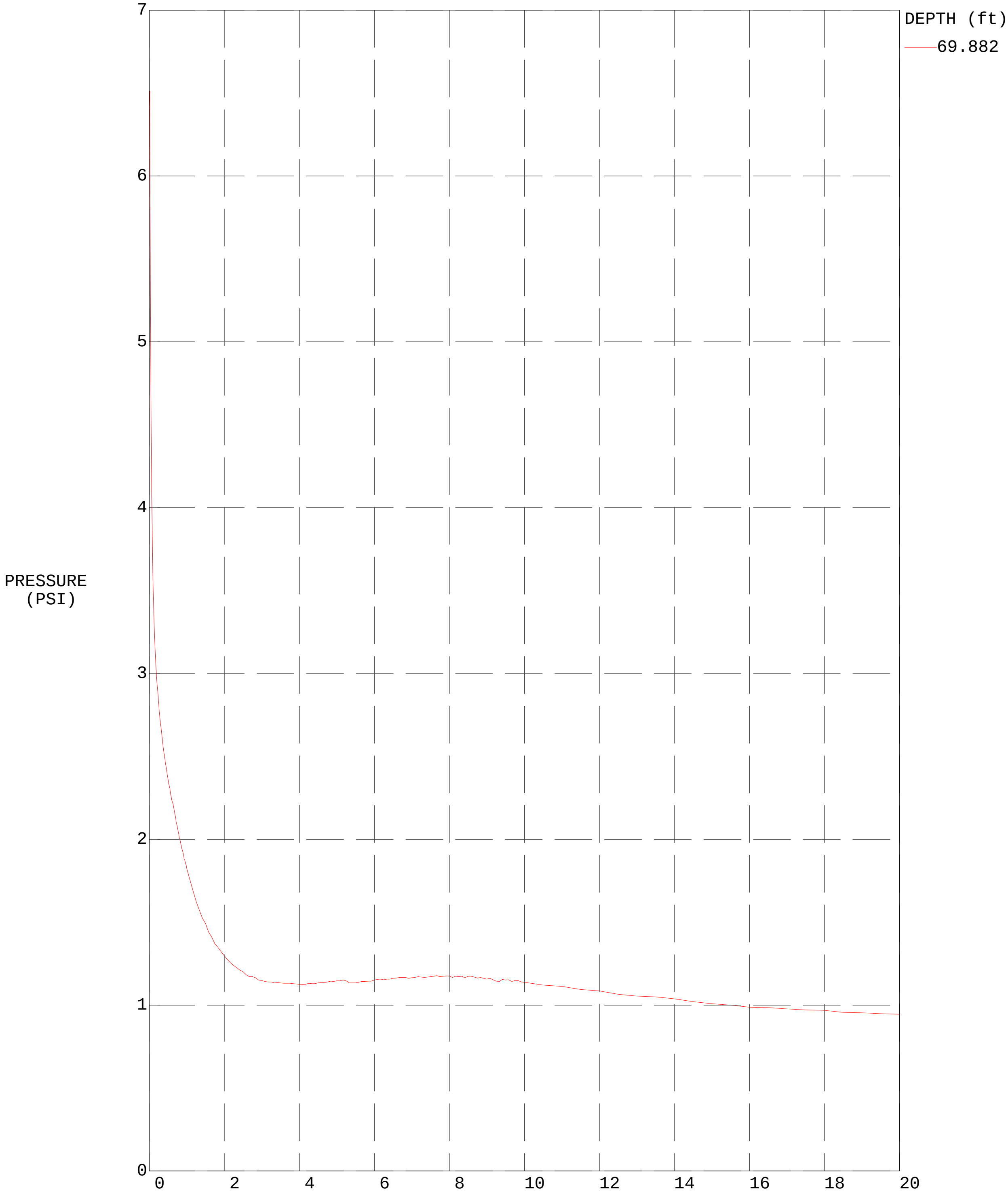


1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GeoDesign / CPT-2 / 1007 Dollar St West Linn

TEST DATE: 5/27/2020 12:07:07 PM

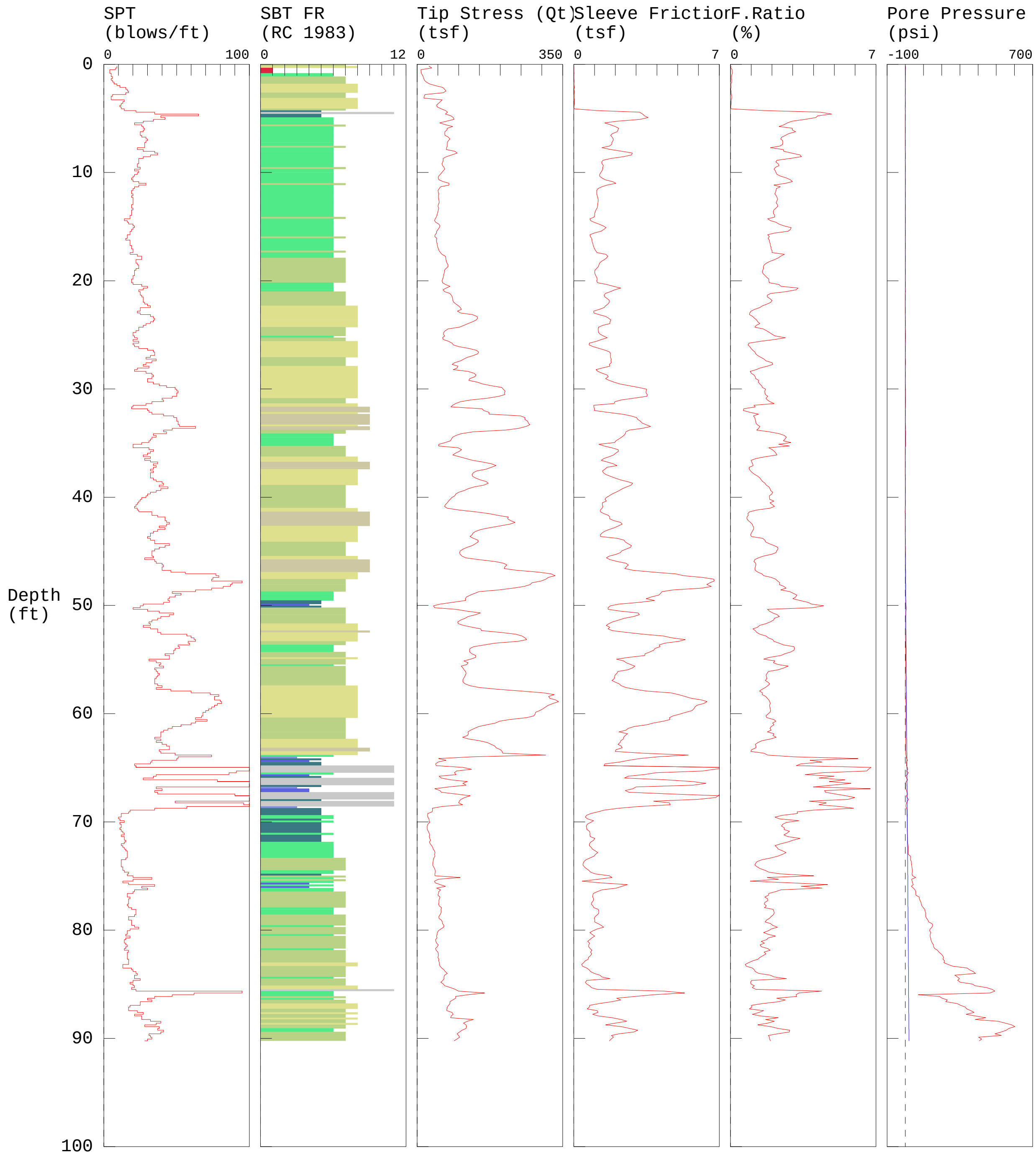


MAXIMUM PRESSURE = 6.513 (PSI)

HYDROSTATIC PRESSURE = 0.944 (PSI), WATER TABLE: 67.70 ft

GeoDesign / CPT-3 / 1007 Dollar St West Linn

OPERATOR: OGE DMM
CONE ID: DSG0709
HOLE NUMBER: CPT-3
TEST DATE: 5/27/2020 1:50:46 PM
TOTAL DEPTH: 90.223 ft

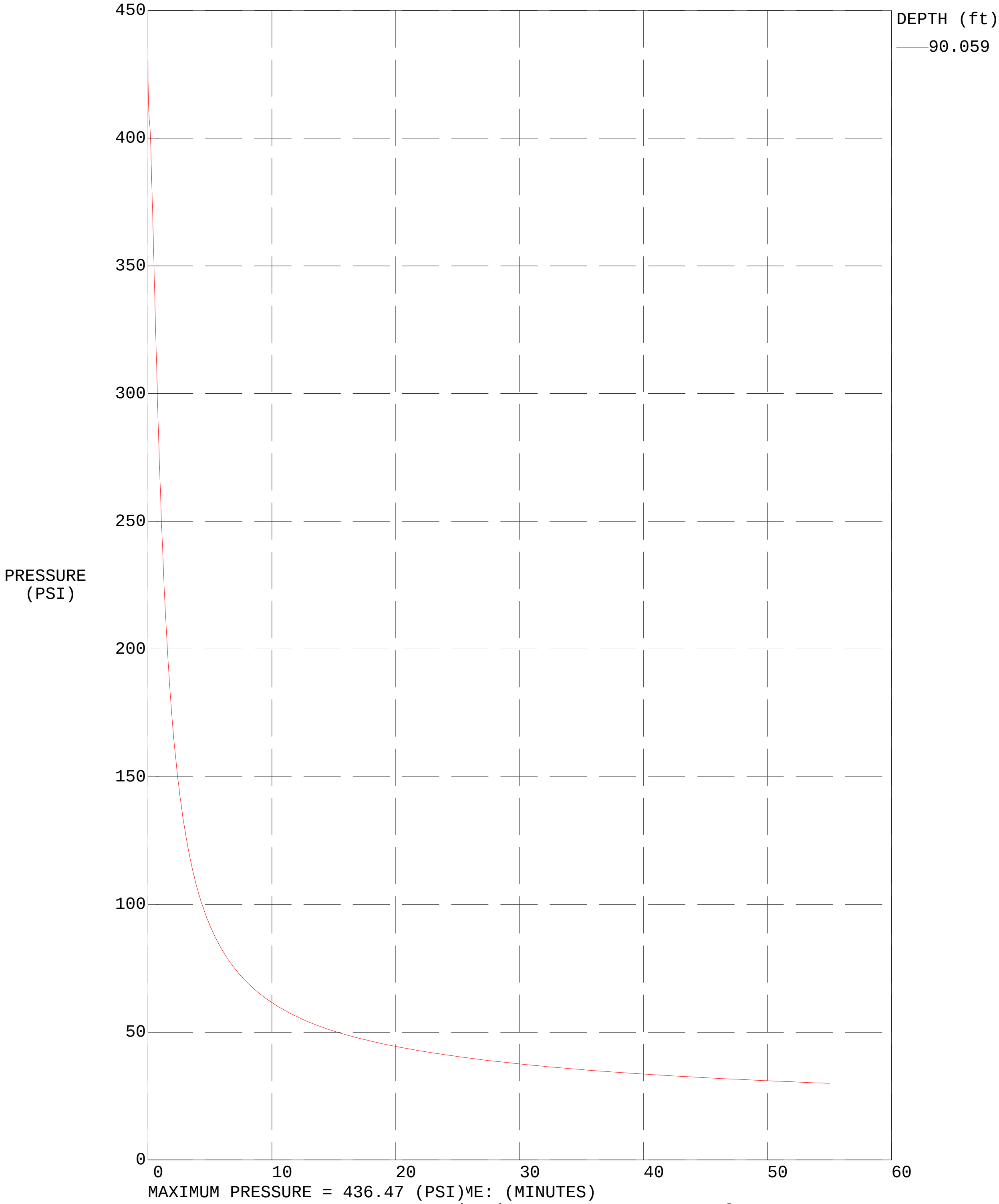


1	sensitive fine gra	4	silty clay to cl	7	silty sand to sandy	10	gravelly sand to sand
2	organic materia	5	clayey silt to silt	8	sand to silty sa	11	very stiff fine grained (*)
3	clay	6	sandy silt to claye	9	sand	12	sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GeoDesign / CPT-3 / 1007 Dollar St West Linn

TEST DATE: 5/27/2020 1:50:46 PM



MAXIMUM PRESSURE = 436.47 (PSI)

HYDROSTATIC PRESSURE = 20.049 (PSI), NO WATER TABLE ESTIMATE DUE TO SLOW DISSIPATION

APPENDIX C

APPENDIX C

PRIOR GEODESIGN EXPLORATIONS AND LABORATORY TESTING

We previously explored subsurface conditions at the site by drilling 3 borings (B-1 through B-3) and excavating 17 test pits (TP-1 through TP-17) at the approximate locations shown on Figures 2 through 4. The borings were completed on July 27, 2006 by Geo-Tech Explorations, Inc. of Tualatin, Oregon, using a truck-mounted CME-75 drill rig equipped for mud rotary drilling methods. The test pits were completed on June 6 and 7, 2006 by Dan J. Fischer Excavating, Inc. using a Komatsu PC60 track-mounted excavator. The approximate locations of the explorations were determined in the field by pacing from existing site features and should be considered approximate.

Laboratory testing conducted on samples collected from the prior explorations included moisture content, Atterberg limits, direct shear, and particle-size analysis, the results of which are presented in this appendix.

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT / ● MOISTURE CONTENT % (RQD% / CORE REC%) 0–50–100	INSTALLATION AND COMMENTS
0	Medium stiff, dark brown SILT; moist (topsoil).	172.0				
1.5	Medium stiff, brown SILT; moist.	170.5 / 1.5				Cuttings are silty at ~2.5 feet.
5	Loose to medium dense, brown, fine SAND with trace to some silt; moist (alluvium, flood deposits).	167.0 / 5.0				Cuttings are sandy at ~5.0 feet
10				X	▲ 10	
15				X	▲ 10 ●	Softer drilling at 16.0 feet.
17			DS	P	●	Stiff push at 17.0 feet.
19	becomes medium dense; stratified with sand at 19.0 feet			X	▲ 13 ●	
22						Softer drilling from 22.0 to 23.0 feet.
25	becomes gray-brown and silty; stratified with sandy silt and silt (2 to 4 inches thick) at 25.0 feet			X	▲ 15	
28						Stiffer drilling at 28.0 feet.
30	becomes stratified with fine, silty sand and dark brown, fine sand at 30.0 feet			X	● ▲ 24	
35	becomes dark brown and without silt at 35.0 feet			X	▲ 29	
39						Softer drilling at 39.0 feet.
40						

DRILLED BY: Geo-Tech Explorations, Inc. LOGGED BY: CMC COMPLETED: 07/27/06

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

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BORING B-1

RENAISSANCE AT RIVER BEND SUBDIVISION
WEST LINN, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	BLOW COUNT / MOISTURE CONTENT % / RQD% / CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
40	with trace medium sand; moist to wet at 40.0 feet				▲ 19	
45					● ▲ 21	
50					▲ 19	
55	becomes silty at 55.0 feet	116.5 / 55.5			▲ 56	
	Very dense, dark brown, fine to medium SAND with trace gravel; moist.	115.5 / 56.5				
	Exploration completed at 56.5 feet.					
60						
65						
70						
75						
80						

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BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

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BORING B-1
(continued)

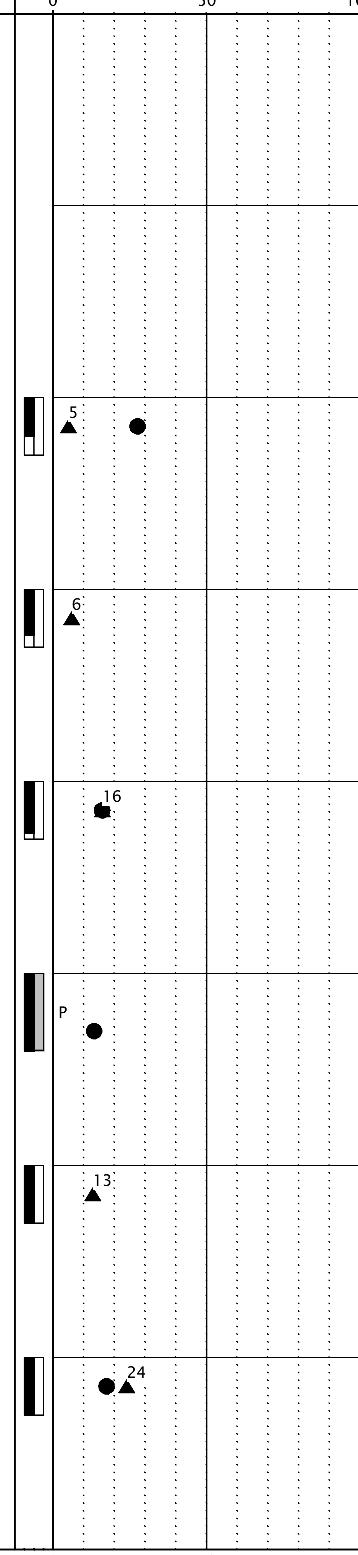
RENAISSANCE AT RIVER BEND SUBDIVISION
WEST LINN, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	INSTALLATION AND COMMENTS
0	Medium stiff, dark brown SILT; moist (topsoil).	166.0	
	Medium stiff, brown SILT; moist.	165.0 / 1.0	Cuttings are silty at ~3.0 feet.
5	Loose, light brown, fine, silty SAND; moist; stratified with tan silt (1 to 2 inches thick) (alluvium, flood deposits).	161.0 / 5.0	Cuttings are sandy at ~5.0 feet.
19	Medium dense, dark brown, fine SAND with trace to no silt; moist.	147.0 / 19.0	Softer drilling at 24.5 feet. Shelby refusal at 26.6 feet.
29.5	Stiff, light gray-brown, sandy SILT with trace mica; moist, nonplastic.	136.5 / 29.5	
33	Medium dense, gray-brown, fine, silty SAND; moist; stratified with tan silt and dark brown, fine sand.	133.0 / 33.0	

TESTING | SAMPLE

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



DRILLED BY: Geo-Tech Explorations, Inc.
LOGGED BY: CMC
COMPLETED: 07/27/06

BORING METHOD: mud rotary (see report text)
BORING BIT DIAMETER: 4 7/8-inch

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BORING B-2

RENAISSANCE AT RIVER BEND SUBDIVISION
WEST LINN, OR

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
40		(continued from previous page)				20	
		Exploration completed at 41.5 feet.	124.5 / 41.5				
45							
50							
55							
60							
65							
70							
75							
80							

DRILLED BY: Geo-Tech Explorations, Inc. LOGGED BY: CMC COMPLETED: 07/27/06

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

BORING LOG RENAIHOMES-4-01-B1-3-TP1-17.GPJ GEODESIGN.GDT PRINT DATE: 9/1/06:SMS:RLF

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BORING B-2
(continued)

RENAISSANCE AT RIVER BEND SUBDIVISION
WEST LINN, OR

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
0		Medium stiff, brown SILT; moist (topsoil).	197.0				
		Medium stiff, brown SILT; moist.	196.0 / 1.0				Encountered white PVC pipe (3-inch diameter) at 1.0 foot. Lost ~40 gallons of drill mud; probable septic drain field. Chipped hole and went down with 3 7/8-inch bit.
5							
		Loose, brown, fine, silty SAND; moist (alluvium, flood deposits).	191.0 / 6.0				
10						▲ 6 ●	LL = NP PL = NP
15						▲ 7 ●	
20		becomes gray-brown; stratified with silt (1 to 2 inches thick) at 20.0 feet				▲ 9	
25		becomes medium dense at 25.0 feet				▲ 12	
30		becomes light gray-brown and stratified with dark brown, fine sand at 30.0 feet				▲ 12 ●	
		Medium dense, dark brown, fine SAND; moist.	165.0 / 32.0				
35						▲ 23	
40							

DRILLED BY: Geo-Tech Explorations, Inc. LOGGED BY: CMC COMPLETED: 07/27/06

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

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BORING B-3

RENAISSANCE AT RIVER BEND SUBDIVISION
WEST LINN, OR

FIGURE A-3

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
40		becomes stratified with silt layers (3 to 4 inches thick); moist to wet at 40.0 feet	155.5 / 41.5			13	
		Exploration completed at 41.5 feet.					
45							
50							
55							
60							
65							
70							
75							
80							

DRILLED BY: Geo-Tech Explorations, Inc. LOGGED BY: CMC COMPLETED: 07/27/06

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

TP-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium stiff, dark brown SILT with trace sand; moist (topsoil, 6-inch-thick root zone).	138.0			
	Medium stiff to stiff, brown SILT with trace clay; moist; 1/2-inch-diameter roots.	137.0 / 1.0	PP		PP = 1.5 tsf
2.5	without roots at 2.0 feet		PP	☒	PP = 2.0 tsf
5.0	with trace fine sand at 5.5 feet				
7.5	becomes gray-brown with some sand at 7.0 feet				
10.0					
12.5	Exploration completed at 12.5 feet.	125.5 / 12.5		☒	No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
15.0					

MOISTURE CONTENT %: 0 50 100

TP-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium dense, dark brown SILT with trace sand; moist (topsoil, 6-inch-thick root zone).	134.0			
	Medium dense to dense, brown SILT with orange and black mottles and trace clay; moist; 1/2-inch-diameter roots.	133.0 / 1.0	PP		PP = 1.0 tsf
2.5	without roots at 2.0 feet		PP	☒	PP = 2.3 tsf
	with trace fine sand at 4.0 feet				
5.0	becomes gray-brown at 6.0 feet				
7.5	with some sand at 8.5 feet				
10.0	Medium dense, gray-brown SAND with some silt and trace mica; moist.	124.0 / 10.0			
12.5	Exploration completed at 12.0 feet.	122.0 / 12.0		☒	No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
15.0					

MOISTURE CONTENT %: 0 50 100

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: RPG COMPLETED: 06/06/06

EXCAVATION METHOD: trackhoe (see report text)

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SEPTEMBER 2006

TEST PIT

RENAISSANCE AT RIVER BEND SUBDIVISION
WEST LINN, OR

FIGURE A-4

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT %	COMMENTS

TP-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	COMMENTS
0.0	Medium stiff, dark brown SILT with trace sand; moist (topsoil, 4-inch-thick root zone).	160.0		
	Medium stiff to stiff, brown SILT with trace clay; moist; occasional 2-inch-diameter roots.	159.0 / 1.0	PP	PP = 2.0 tsf
2.5			PP	PP = 2.5 tsf
	with trace fine sand at 4.0 feet			
5.0	becomes gray-brown and sandy at 6.0 feet			
7.5				
	Medium dense, gray-brown, fine SAND with trace to some silt and trace mica; moist.	151.0 / 9.0		
10.0	Exploration completed at 11.0 feet.	149.0 / 11.0		No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
12.5				
15.0				

Moisture content scale: 0 50 100

TP-4

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	COMMENTS
0.0	Medium stiff, dark brown SILT with trace sand and clay; moist (topsoil, 4-inch-thick root zone).	153.0		
	Medium stiff to stiff, brown SILT with orange and black mottles and trace clay; moist.	152.5 / 0.5	PP	PP = 1.5 tsf
2.5			PP	PP = 1.5 tsf
	with trace fine sand at 4.0 feet			
5.0				
7.5	becomes gray-brown and sandy with trace mica at 8.0 feet		P200	P200 = 69% Infiltration test: 0.5 inches/hour at 9.0 feet.
10.0	Exploration completed at 11.0 feet.	142.0 / 11.0		No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
12.5				
15.0				

Moisture content scale: 0 50 100

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: RPG COMPLETED: 06/06/06

EXCAVATION METHOD: trackhoe (see report text)

TP-5

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium stiff, dark brown SILT with trace sand and clay; moist (topsoil, 4-inch-thick root zone).	154.0			
	Medium stiff to stiff, brown SILT with trace clay; moist; occasional 1/2-inch-diameter roots.	153.0 / 1.0			
2.5			PP		PP = 1.0 tsf
	without roots at 2.0 feet		PP	☒	PP = 2.5 tsf
	with trace sand at 3.0 feet				
5.0	Medium dense, gray-brown, fine SAND with some silt and trace mica; moist.	149.0 / 5.0			
				☒	
7.5	with trace silt at 8.0 feet				
10.0					
12.5				☒	No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
	Exploration completed at 13.5 feet.	140.5 / 13.5			
15.0					

TP-6

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium stiff, dark brown SILT with trace sand; moist (topsoil, 6-inch-thick root zone).	145.0			
	Medium stiff to stiff, brown SILT with trace clay; moist; 1-inch-diameter roots.	144.0 / 1.0	PP		PP = 1.0 tsf
2.5	without roots at 2.0 feet		PP	☒	PP = 1.3 tsf
5.0	with trace to some fine sand at 4.5 feet				
	Medium dense, gray-brown, fine, silty SAND with trace mica; moist.	139.0 / 6.0			
7.5				☒	
10.0	with trace to some silt at 9.0 feet				
12.5				☒	
	Stiff, brown SILT with red oxidation; moist.	131.5 / 13.5			
15.0	becomes sandy at 15.5 feet		ATT	☒	LL = 34% PL = 24%
	Medium dense, gray-brown SAND with some silt; moist.	129.0 / 16.0			No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
17.5	Exploration completed at 16.5 feet.	128.5 / 16.5			

● MOISTURE CONTENT % (0 – 50 – 100)

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: RPG COMPLETED: 06/06/06

EXCAVATION METHOD: trackhoe (see report text)

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RENAIHOMES-4-01	**TEST PIT**	
SEPTEMBER 2006	RENAISSANCE AT RIVER BEND SUBDIVISION WEST LINN, OR	**FIGURE A-6**

TP-7

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium stiff, dark brown SILT with trace sand; moist (topsoil, 6-inch-thick root zone).	179.0			
	Medium stiff to stiff, brown SILT with trace clay and sand with occasional red staining; moist.	177.5 / 1.5	PP		PP = 1.3 tsf
2.5			PP	☒	PP = 2.0 tsf
5.0	with some sand at 5.0 feet				Infiltration test: 0.5 inches/hour at 5.0 feet.
				☒	
7.5	Medium dense, gray-brown, silty SAND with trace mica; moist.	171.5 / 7.5			
10.0					
	Exploration completed at 11.0 feet.	168.0 / 11.0	P200	☒	P200 = 31% No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
12.5					
15.0					

TP-8

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium stiff, dark brown SILT with trace sand; moist (topsoil, 4-inch-thick root zone).	192.0			
	Medium stiff to stiff, brown SILT with trace clay; moist; 1-inch-diameter roots.	191.5 / 0.5	PP		PP = 1.3 tsf
2.5	without roots at 2.0 feet		PP		PP = 1.5 tsf
			PP	☒	PP = 2.3 tsf
	with trace fine sand at 4.0 feet				
5.0					
	Medium dense, gray-brown SAND with some silt and trace mica; moist.	186.5 / 5.5			
7.5				☒	
10.0					
	Exploration completed at 11.5 feet.	180.5 / 11.5		☒	No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
12.5					
15.0					

● MOISTURE CONTENT % (0 50 100)

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: RPG COMPLETED: 06/06/06

EXCAVATION METHOD: trackhoe (see report text)

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 / 50 / 100)	COMMENTS
TP-9			166.5				
0.0		Medium stiff, dark brown SILT with trace sand; moist (topsoil, 4-inch-thick root zone).	165.5 / 1.0				
2.5		Medium stiff to stiff, brown SILT with orange and black mottles and trace clay and sand; moist.		PP			PP = 1.5 tsf
				PP	☒	●	PP = 2.5 tsf
7.5		Medium dense, gray-brown, fine SAND with trace to some silt and trace mica; moist.	159.5 / 7.0		☒	●	
12.5		Exploration completed at 12.5 feet.	154.0 / 12.5		☒		No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
TP-10			201.5				
0.0		Medium dense, dark brown SILT with trace sand and clay; moist (fill, 5-inch-thick root zone).		PP	☒		PP = 1.0 tsf
2.5		Stiff, brown SILT with trace clay; moist.	199.0 / 2.5	PP	☒		PP = 1.3 tsf
				PP			PP = 3.0 tsf
		with trace sand at 6.0 feet					
		becomes sandy at 8.0 feet			☒		
10.0		Medium dense, gray-brown SAND with some silt and trace mica; moist.	191.5 / 10.0				
12.0		Exploration completed at 12.0 feet.	189.5 / 12.0		☒		No groundwater seepage observed to the depth explored. No caving observed to the depth explored.

EXCAVATED BY: Dan J. Fischer Excavating, Inc.   LOGGED BY: RPG   COMPLETED: 06/06/06

EXCAVATION METHOD: trackhoe (see report text)

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TEST PIT

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WEST LINN, OR

FIGURE A-8

TP-11

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	MOISTURE CONTENT %	COMMENTS
0.0	Medium stiff, dark brown SILT with trace sand; moist (topsoil, 3-inch-thick root zone).	190.0				
	Medium stiff to stiff, brown SILT with red and black mottles and trace clay and sand; moist; 1-inch-diameter roots.	189.0 / 1.0	PP			PP = 3.5 tsf
2.5			PP	☒	●	PP = 2.5 tsf
5.0	Medium dense, gray-brown, fine, silty SAND with trace mica; moist; 1/2-inch-diameter roots.	184.5 / 5.5				
7.5				☒		
10.0	without roots at 10.0 feet					
12.5	Exploration completed at 12.0 feet.	178.0 / 12.0		☒		No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
15.0						

TP-12

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	MOISTURE CONTENT %	COMMENTS
0.0	Medium stiff, dark brown SILT with trace sand; moist (topsoil, 3-inch-thick root zone).	201.5				
	Medium stiff to stiff, brown SILT with trace clay; moist, low plasticity.	201.0 / 0.5	PP			PP = 0.8 tsf
2.5			PP	☒	●	PP = 2.0 tsf
	becomes stiff at 3.0 feet		PP			PP = 3.0 tsf
	Medium dense, gray-brown, fine, silty SAND with trace mica; moist.	198.0 / 3.5				
5.0				☒	●	
7.5	with some silt at 8.5 feet			☒		
10.0						
12.5	Exploration completed at 13.5 feet.	188.0 / 13.5		☒		No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
15.0						

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: RPG COMPLETED: 06/06/06

EXCAVATION METHOD: trackhoe (see report text)

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RENAIHOMES-4-01

SEPTEMBER 2006

TEST PIT

RENAISSANCE AT RIVER BEND SUBDIVISION
WEST LINN, OR

FIGURE A-9

TP-13

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT %	COMMENTS
0.0	Medium stiff, dark brown SILT with trace sand; moist (topsoil, 4-inch-thick root zone).	171.0 / 170.5 / 0.5	PP			PP = 1.5 tsf
2.5	Medium stiff to stiff, brown SILT with red and black mottles and trace sand and clay; moist; 1-inch-diameter roots. without roots at 2.0 feet		PP, PP	⊠	●	PP = 2.3 tsf; PP = 2.5 tsf
6.0	Medium stiff, gray-brown, fine, silty SAND with trace mica; moist.	165.0 / 6.0	P200	⊠	●	Infiltration test: 1.5 inches/hour at 6.5 feet. P200 = 42%
11.5	Exploration completed at 11.5 feet.	159.5 / 11.5		⊠	●	No groundwater seepage observed to the depth explored. No caving observed to the depth explored.

TP-14

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT %	COMMENTS
0.0	Medium stiff, dark brown SILT with trace sand; moist (topsoil, 5-inch-thick root zone).	189.0 / 188.0 / 1.0	PP			PP = 2.0 tsf
2.5	Medium stiff to stiff, brown SILT with trace sand and clay; moist; 1-inch-diameter roots. without roots at 2.0 feet		PP, PP	⊠		PP = 3.0 tsf; PP = 3.3 tsf
6.0	Medium dense, gray-brown SAND with some silt and trace mica; moist. becomes silty at 8.5 feet; with trace silt at 10.0 feet	183.0 / 6.0		⊠		
12.5	Exploration completed at 12.5 feet.	176.5 / 12.5		⊠		No groundwater seepage observed to the depth explored. No caving observed to the depth explored.

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: RPG COMPLETED: 06/07/06

EXCAVATION METHOD: trackhoe (see report text)

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RENAIHOMES-4-01 | SEPTEMBER 2006

TEST PIT

RENAISSANCE AT RIVER BEND SUBDIVISION
WEST LINN, OR

FIGURE A-10

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
TP-15			202.0				
0.0		Medium stiff, dark brown SILT with trace sand; moist (topsoil, 2-inch-thick root zone).	201.5 / 0.5	PP			PP = 1.3 tsf
		Medium stiff to stiff, brown SILT with red and black mottles and trace sand and clay; moist; roots.		PP			PP = 1.8 tsf
2.5				PP	☒	●	PP = 2.5 tsf
		with some sand at 3.5 feet					
5.0							
		Medium dense, gray-brown, fine, silty SAND; moist.	196.0 / 6.0				Infiltration test: 1.0 inches/hour at 6.0 feet.
7.5				P200	☒	●	P200 = 45%
10.0							No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
		Exploration completed at 10.5 feet.	191.5 / 10.5		☒	●	
12.5							
15.0							
TP-16			203.0				
0.0		Medium stiff, dark brown SILT with trace sand and gravel; moist (topsoil, 5-inch-thick root zone).	202.0 / 1.0				
		Stiff, brown SILT with orange and black mottles and trace sand and clay; moist; 1-inch-diameter roots.		PP			PP = 3.0 tsf
2.5					☒	●	
5.0							
		without clay and roots at 6.0 feet					
7.5							
		becomes gray-brown and sandy at 8.0 feet					
10.0		Medium dense, gray-brown, fine SAND with some silt and trace mica; moist.	193.0 / 10.0		☒		
							No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
12.5		Exploration completed at 12.5 feet.	190.5 / 12.5				
15.0							

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: RPG COMPLETED: 06/07/06

EXCAVATION METHOD: trackhoe (see report text)

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SEPTEMBER 2006

TEST PIT

RENAISSANCE AT RIVER BEND SUBDIVISION
WEST LINN, OR

FIGURE A-11

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
TP-17			201.0				
0.0		Medium stiff, dark brown SILT with trace sand and clay; moist (topsoil, 4-inch-thick root zone).	200.0 / 1.0	PP			PP = 2.5 tsf
2.5		Medium stiff to stiff, brown SILT with black mottles and trace sand and clay; moist; 1-inch-diameter roots.		PP	⊠	●	PP = 3.3 tsf
				PP			PP = 3.5 tsf
5.0							
		Medium dense, gray-brown, silty SAND with trace mica; moist.	195.0 / 6.0		⊠	●	
7.5							
		with some silt at 9.0 feet					
10.0							
		with trace silt at 10.5 feet					No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
		Exploration completed at 11.5 feet.	189.5 / 11.5		⊠	●	
12.5							
15.0							

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: RPG COMPLETED: 06/07/06

EXCAVATION METHOD: trackhoe (see report text)

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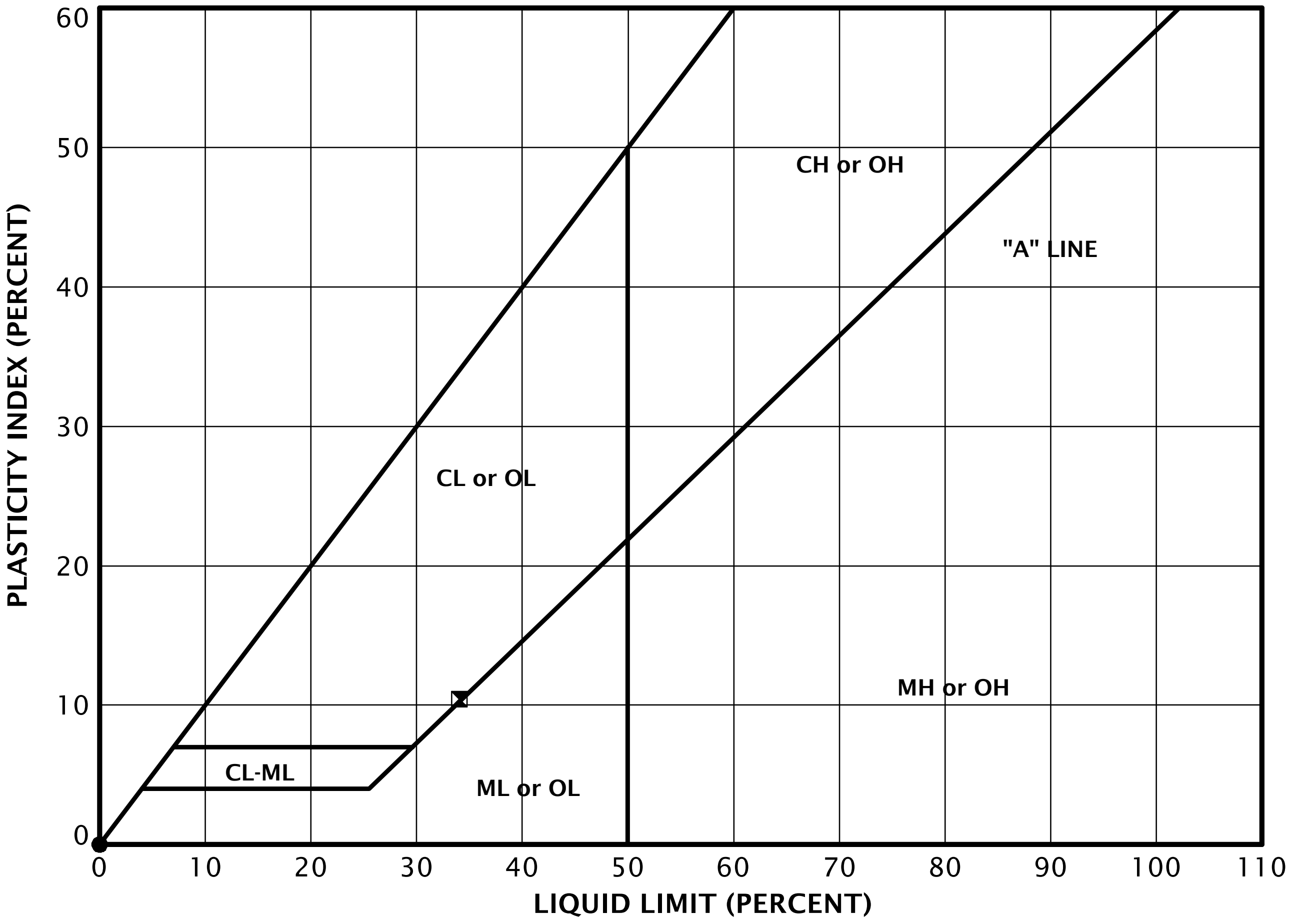
RENAIHOMES-4-01	TEST PIT	
SEPTEMBER 2006	RENAISSANCE AT RIVER BEND SUBDIVISION WEST LINN, OR	FIGURE A-12



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT (PERCENT)	PLASTIC LIMIT (PERCENT)	PLASTICITY INDEX (PERCENT)
●	B-3	10.0	23	NP	NP	NP
⊠	TP-6	15.5	31	34	24	10

ATTERBERG_LIMITS 7 RENAIHOMES-4-01-B1-3-TP1-17.GPJ GEODESIGN.GDT PRINT DATE: 9/1/06:SMS:RLF

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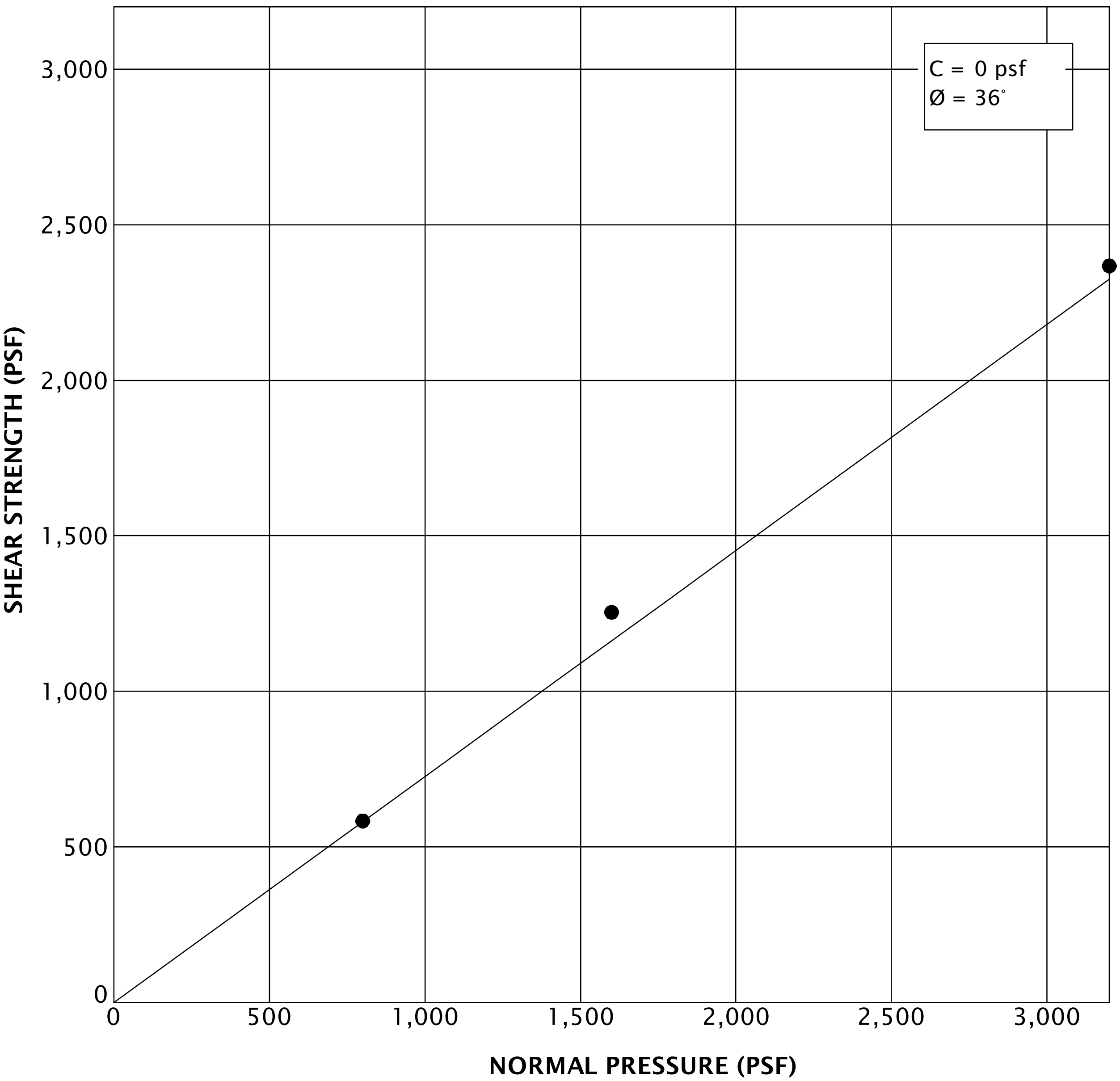
ATTERBERG LIMITS TEST RESULTS

RENAISSANCE AT RIVER BEND SUBDIVISION
WEST LINN, OR

FIGURE A-13



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SOAKED
●	B-1	17.0	16	85	YES

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DIRECT SHEAR TEST RESULTS

RENAISSANCE AT RIVER BEND SUBDIVISION
WEST LINN, OR

FIGURE A-14

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT (PERCENT)	PLASTIC LIMIT (PERCENT)	PLASTICITY INDEX (PERCENT)
B-1	15.0	157.0	19							
B-1	17.0	155.0	16							
B-1	19.0	153.0	18							
B-1	30.0	142.0	17							
B-1	45.0	127.0	15							
B-2	10.0	156.0	27							
B-2	20.0	146.0	16							
B-2	26.5	139.5	13							
B-2	35.0	131.0	17							
B-2	40.0	126.0	16							
B-3	10.0	187.0	23					NP	NP	NP
B-3	15.0	182.0	25							
B-3	30.0	167.0	25							
B-3	40.0	157.0	28							
TP-1	2.5	135.5	31							
TP-1	12.5	125.5	21							
TP-3	3.0	157.0	30							
TP-4	2.5	150.5	30							
TP-4	9.0	144.0	29				69			
TP-4	10.5	142.5	22							
TP-5	3.0	151.0	34							
TP-6	2.5	142.5	31							
TP-6	7.0	138.0	30							
TP-6	12.5	132.5	29							
TP-6	15.5	129.5	31					34	24	10
TP-7	11.0	168.0	17				31			
TP-8	3.0	189.0	33							

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RENAIHOMES-4-01

SEPTEMBER 2006

SUMMARY OF LABORATORY DATA

RENAISSANCE AT RIVER BEND SUBDIVISION
WEST LINN, OR

FIGURE A-15

LAB SUMMARY RENAIHOMES-4-01-B1-3-TP1-17.GPJ GEODESIGN.GDT PRINT DATE: 9/1/06:SMS:RLF

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT (PERCENT)	PLASTIC LIMIT (PERCENT)	PLASTICITY INDEX (PERCENT)
TP-8	7.0	185.0	20							
TP-8	11.0	181.0	15							
TP-9	2.5	164.0	25							
TP-9	7.5	159.0	19							
TP-11	2.5	187.5	26							
TP-12	2.0	199.5	28							
TP-12	5.5	196.0	29							
TP-13	2.5	168.5	25							
TP-13	8.0	163.0	22				42			
TP-13	11.5	159.5	21							
TP-15	2.5	199.5	28							
TP-15	7.0	195.0	22				45			
TP-15	10.5	191.5	19							
TP-16	2.5	200.5	17							
TP-17	2.0	199.0	31							
TP-17	6.5	194.5	23							
TP-17	11.5	189.5	18							

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RENAIHOMES-4-01

SEPTEMBER 2006

SUMMARY OF LABORATORY DATA
(continued)

RENAISSANCE AT RIVER BEND SUBDIVISION
WEST LINN, OR

FIGURE A-15

APPENDIX D

APPENDIX D

PRIOR EXPLORATIONS BY OTHERS

Explorations were conducted in 2007 by Foundation Engineering, Inc. for the Willamette Falls Drive/Borland Road Bridge over the Tualatin River. One of the borings (BH-2) was completed near the southeast abutment at the approximate location shown on Figures 2 through 4. The results of the boring are included on the cross section shown in the foundation data sheet presented in this appendix.

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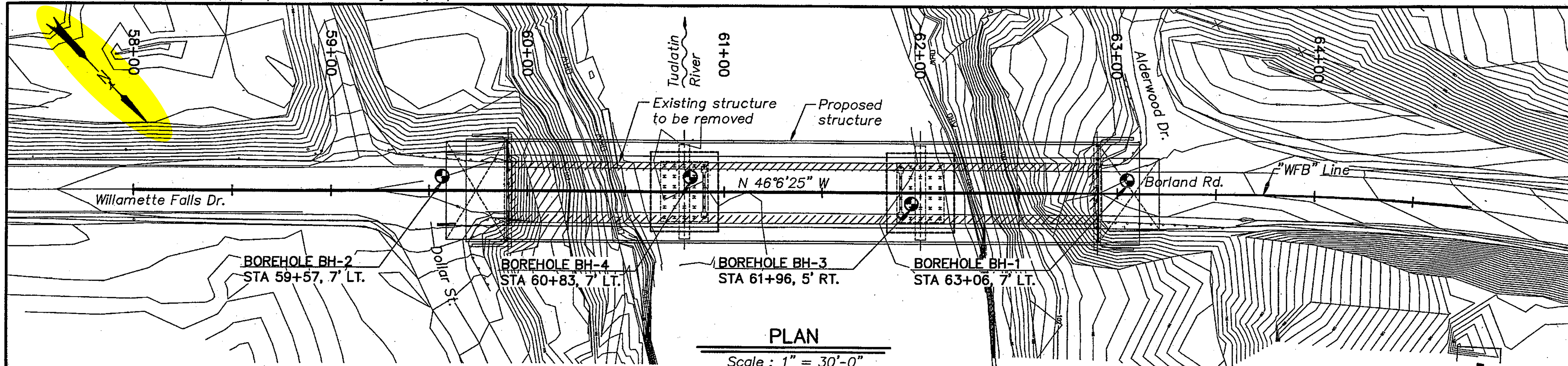


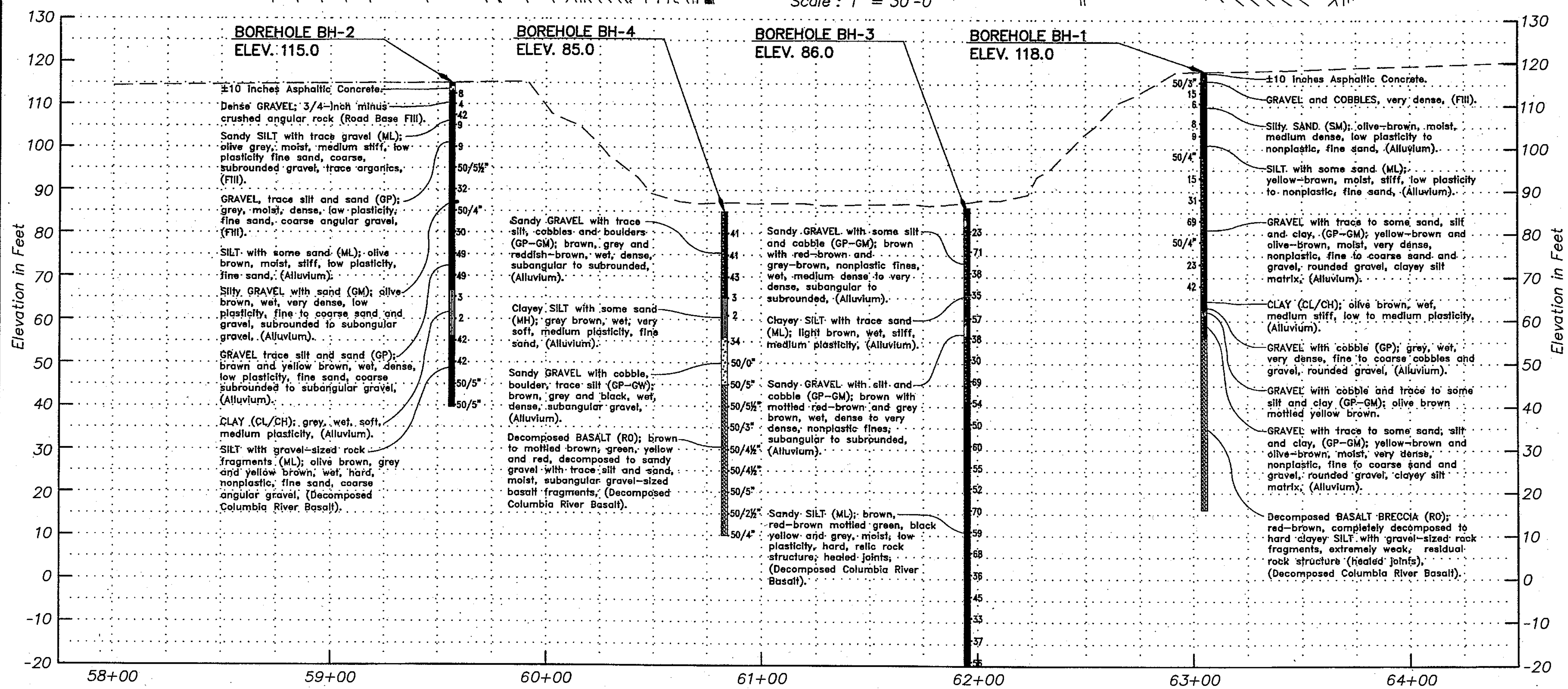
PLAN

Scale : 1" = 30'-0"

LEGEND OF MATERIALS

- ASPHALTIC CONCRETE
- Sandy SILT
- SAND & GRAVEL mixture
- Silty SAND
- GRAVEL, SAND, SILT mixture
- SAND
- Clayey SILT
- SILT
- Silty CLAY, some sand
- Silty GRAVEL & GRAVEL, some silt
- Silty CLAY
- SILTSTONE

14 Standard Penetration Test. N value.



ELEVATION

Horiz. Scale : 1" = 30'-0"
Vert. Scale : 1" = 15'-0"

Foundation data shown on this drawing may be a consolidation of information and/or revision in terminology from the Soils and Geological Exploration Logs. The Soils and Exploration Logs used in compiling this drawing are available upon request for review at the office of David Evans and Associates, Inc., Salem, OR.

Co. Br. No. 06507

FOUNDATION ENGINEERING, INC.
PROFESSIONAL GEOTECHNICAL SERVICES
820 N.W. CORNELL AVENUE
CORVALLIS, OREGON 97330
BUS. (541) 757-7645 FAX (541) 757-7650

△	DATE	REVISION	BY

ACCOMPANIED BY DWGS. 77798-77815, BR140, BR155, BR165, BR203, BR246, BR350

DRAFTER: G. Haffner
DESIGNER: Timothy Pfeifer
CHECKER: Mitch Schaub
REVIEWER: N/A

REGISTERED PROFESSIONAL ENGINEER 18,066 OREGON JULY 21, 1992 TIMOTHY J. PFEIFFER
EXPIRES: 12-31-08

CLACKAMAS COUNTY

DAVID EVANS AND ASSOCIATES INC.
530 Center Street N.E., Suite 605
Salem Oregon 97301
Phone: 503.361.8835

STRUCTURE NO.	20765
DATE	Jan. 2008
CALC. BOOK	-----

TUALATIN RIVER, BORLAND ROAD
CLACKAMAS COUNTY

FOUNDATION DATA

SHEET 3 OF 18

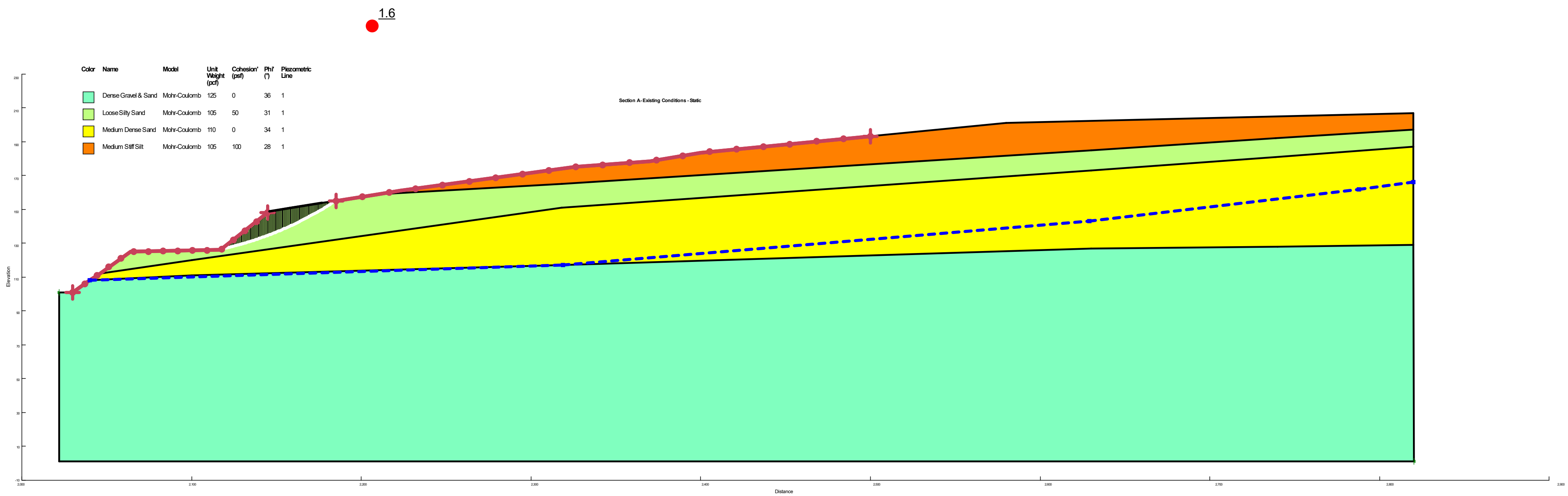
DRAWING NO. 77800

APPENDIX E

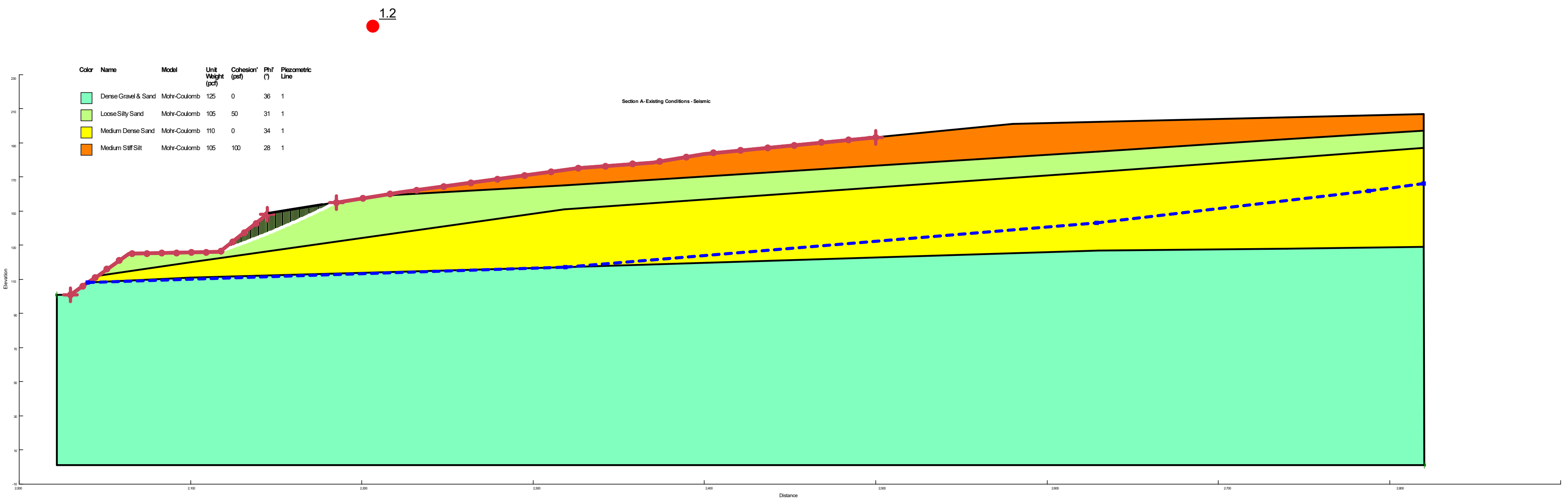
APPENDIX E

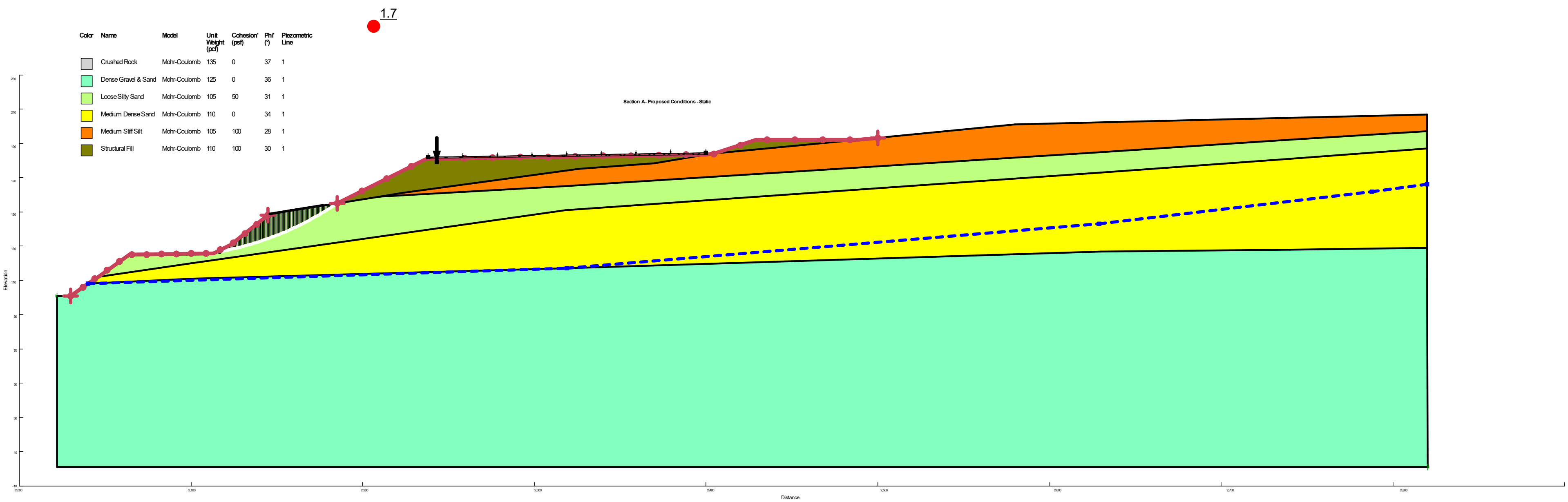
SLOPE/W SLOPE STABILITY ANALYSES

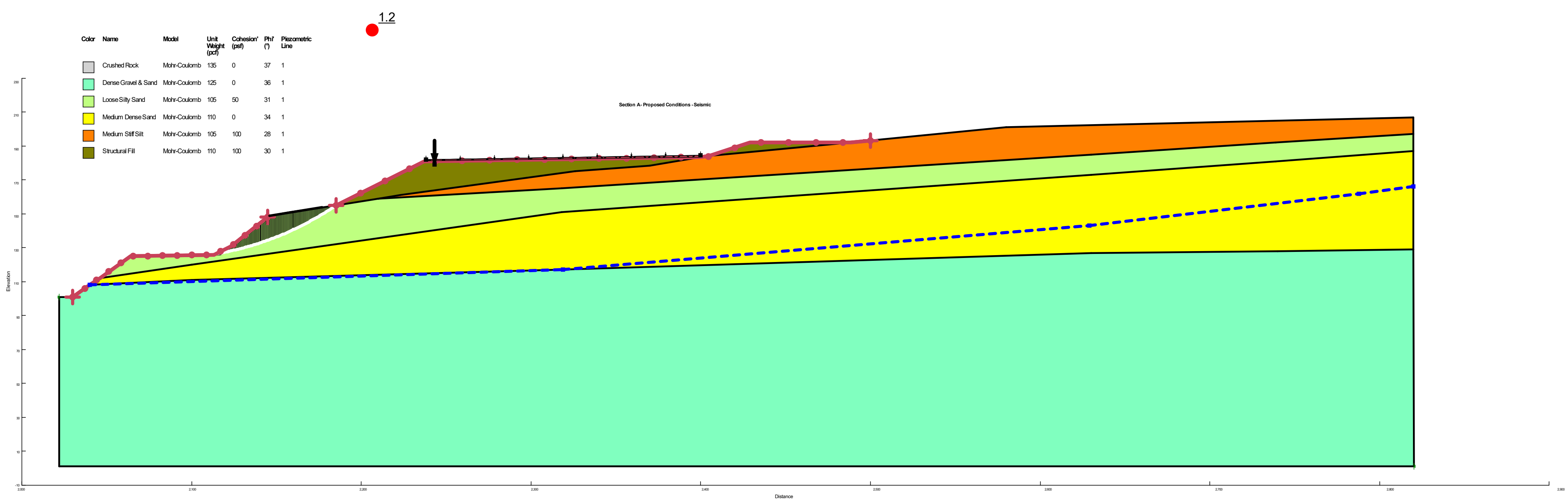
Results of slope stability analyses using the SLOPE/W software package are presented in this appendix.



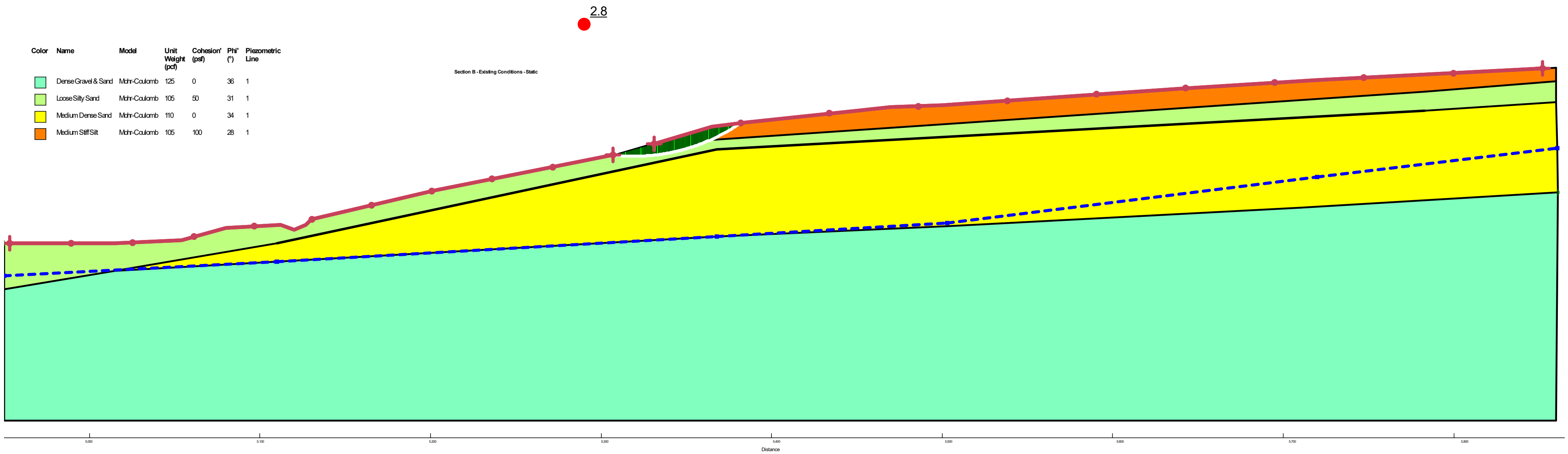
1.6
Section A- Existing Conditions - Static
Color
Name
Model
Unit Weight (pcf)
Cohesion' (psf)
Phi' (°)
Piezometric Line
Dense Gravel & Sand Mohr-Coulomb 125 0 36 1
Loose Silty Sand Mohr-Coulomb 105 50 31 1
Medium Dense Sand Mohr-Coulomb 110 0 34 1
Medium Stiff Silt Mohr-Coulomb 105 100 28 1
Elevation
Distance


1.2
Section A- Existing Conditions - Seismic
Color
Name
Model
Unit Weight (pcf)
Cohesion' (psf)
Phi' (°)
Piezometric Line
Dense Gravel & Sand Mohr-Coulomb 125 0 36 1
Loose Silty Sand Mohr-Coulomb 105 50 31 1
Medium Dense Sand Mohr-Coulomb 110 0 34 1
Medium Stiff Silt Mohr-Coulomb 105 100 28 1
Elevation
Distance

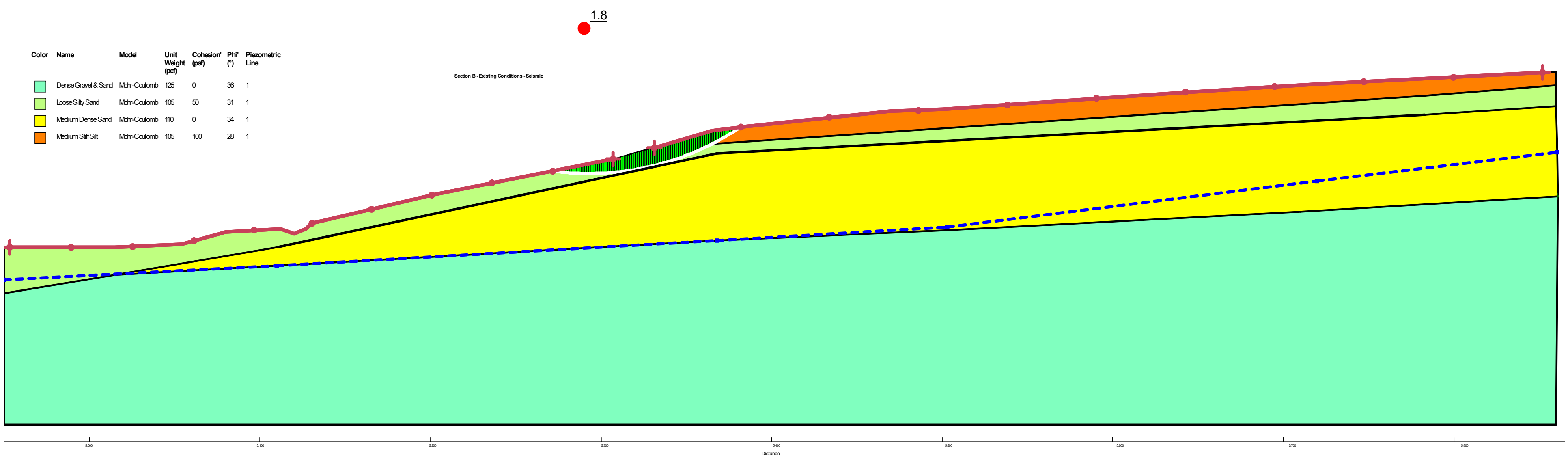

1.7
Color
Name
Model
Unit Weight (pcf)
Cohesion' (psf)
Phi' (°)
Piezometric Line
Crushed Rock Mohr-Coulomb 135 0 37 1
Dense Gravel & Sand Mohr-Coulomb 125 0 36 1
Loose Silty Sand Mohr-Coulomb 105 50 31 1
Medium Dense Sand Mohr-Coulomb 110 0 34 1
Medium Stiff Silt Mohr-Coulomb 105 100 28 1
Structural Fill Mohr-Coulomb 110 100 30 1
Section A- Proposed Conditions - Static
Elevation
230
210
190
170
150
130
110
90
70
50
30
10
-10
2,000
2,100
2,200
2,300
2,400
2,500
2,600
2,700
2,800
2,900
Distance

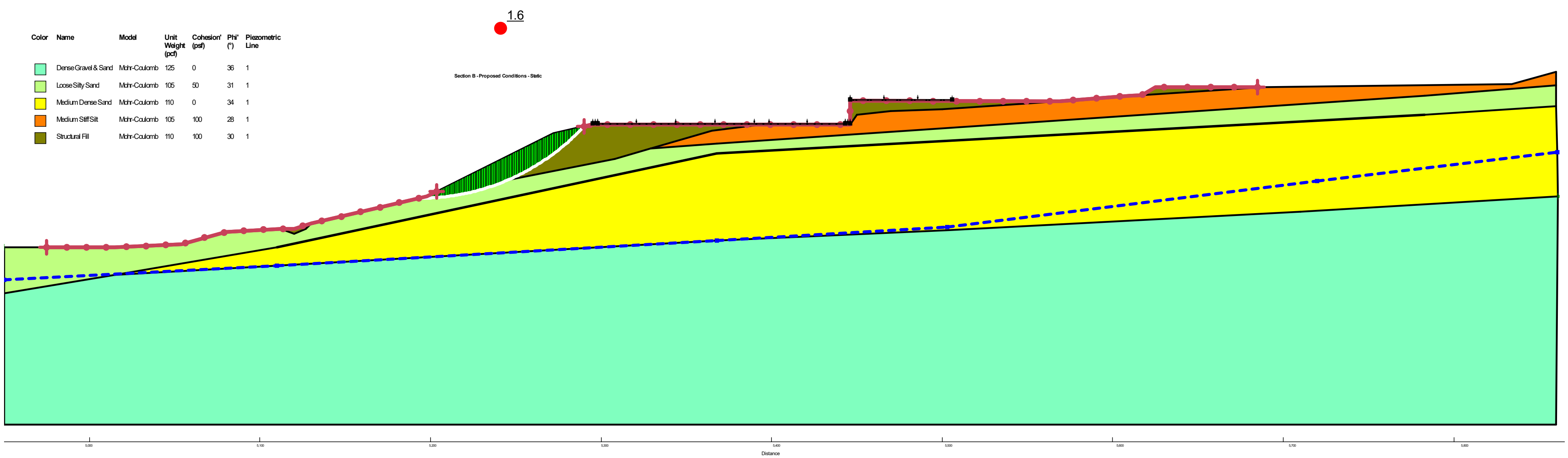


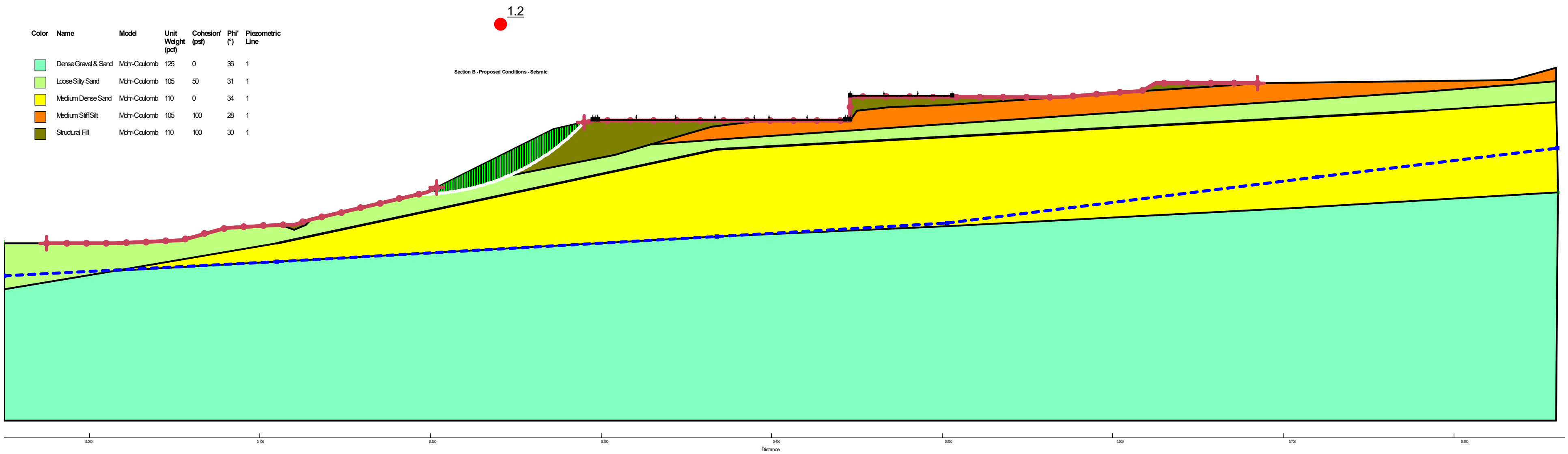
1.2
Color
Name
Model
Unit Weight (pcf)
Cohesion' (psf)
Phi' (°)
Piezometric Line
Crushed Rock Mohr-Coulomb 135 0 37 1
Dense Gravel & Sand Mohr-Coulomb 125 0 36 1
Loose Silty Sand Mohr-Coulomb 105 50 31 1
Medium Dense Sand Mohr-Coulomb 110 0 34 1
Medium Stiff Silt Mohr-Coulomb 105 100 28 1
Structural Fill Mohr-Coulomb 110 100 30 1
Section A- Proposed Conditions - Seismic
Elevation
230
210
190
170
150
130
110
90
70
50
30
10
-10
2,000
2,100
2,200
2,300
2,400
2,500
2,600
2,700
2,800
2,900
Distance



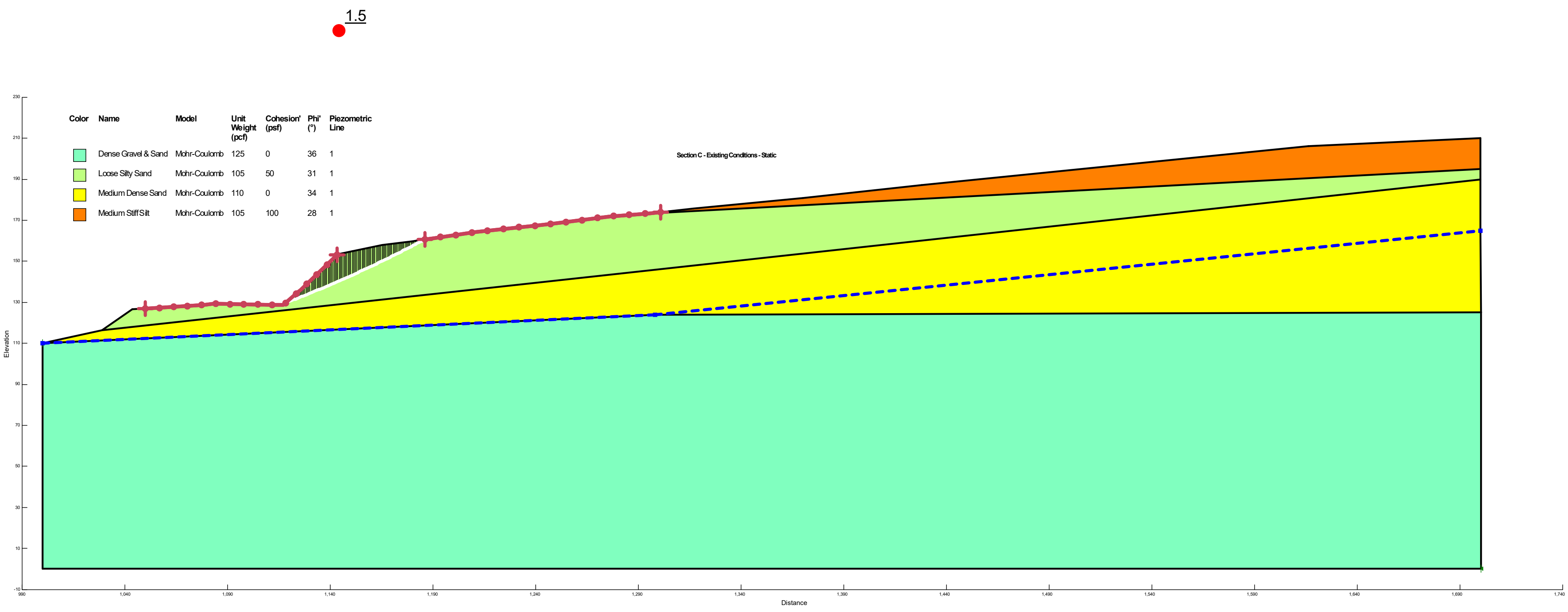
2.8
Section B - Existing Conditions - Static
Color
Name
Model
Unit Weight (pcf)
Cohesion' (psf)
Phi' (°)
Piezometric Line
Dense Gravel & Sand Mohr-Coulomb 125 0 36 1
Loose Silty Sand Mohr-Coulomb 105 50 31 1
Medium Dense Sand Mohr-Coulomb 110 0 34 1
Medium Stiff Silt Mohr-Coulomb 105 100 28 1
5,000
5,100
5,200
5,300
5,400
5,500
5,600
5,700
5,800
Distance


1.8
Color
Name
Model
Unit Weight (pcf)
Cohesion' (psf)
Phi' (°)
Piezometric Line
Dense Gravel & Sand
Mohr-Coulomb
125
0
36
1
Loose Silty Sand
Mohr-Coulomb
105
50
31
1
Medium Dense Sand
Mohr-Coulomb
110
0
34
1
Medium Stiff Silt
Mohr-Coulomb
105
100
28
1
Section B - Existing Conditions - Seismic
5,000
5,100
5,200
5,300
5,400
5,500
5,600
5,700
5,800
Distance

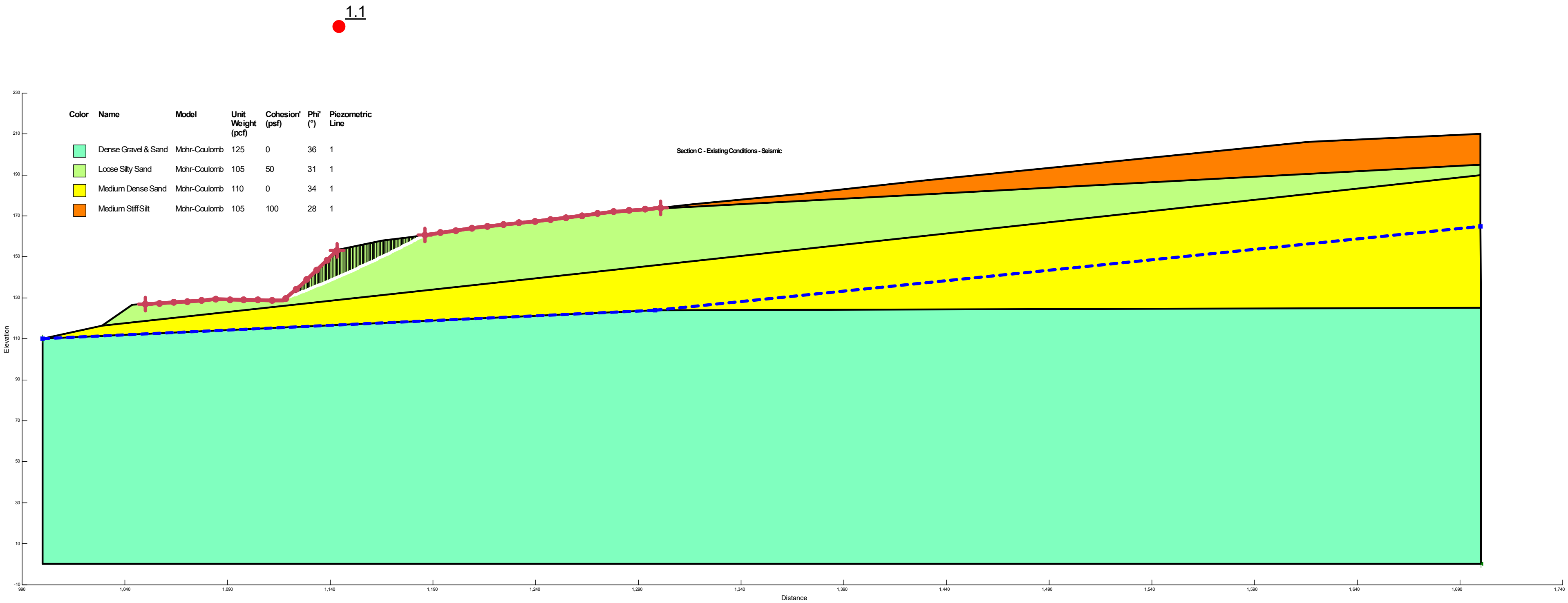

1.6
Section B - Proposed Conditions - Static
Color Name Model Unit Weight (pcf) Cohesion' (psf) Phi' (°) Piezometric Line
Dense Gravel & Sand Mohr-Coulomb 125 0 36 1
Loose Silty Sand Mohr-Coulomb 105 50 31 1
Medium Dense Sand Mohr-Coulomb 110 0 34 1
Medium Stiff Silt Mohr-Coulomb 105 100 28 1
Structural Fill Mohr-Coulomb 110 100 30 1
5,000
5,100
5,200
5,300
5,400
5,500
5,600
5,700
5,800
Distance



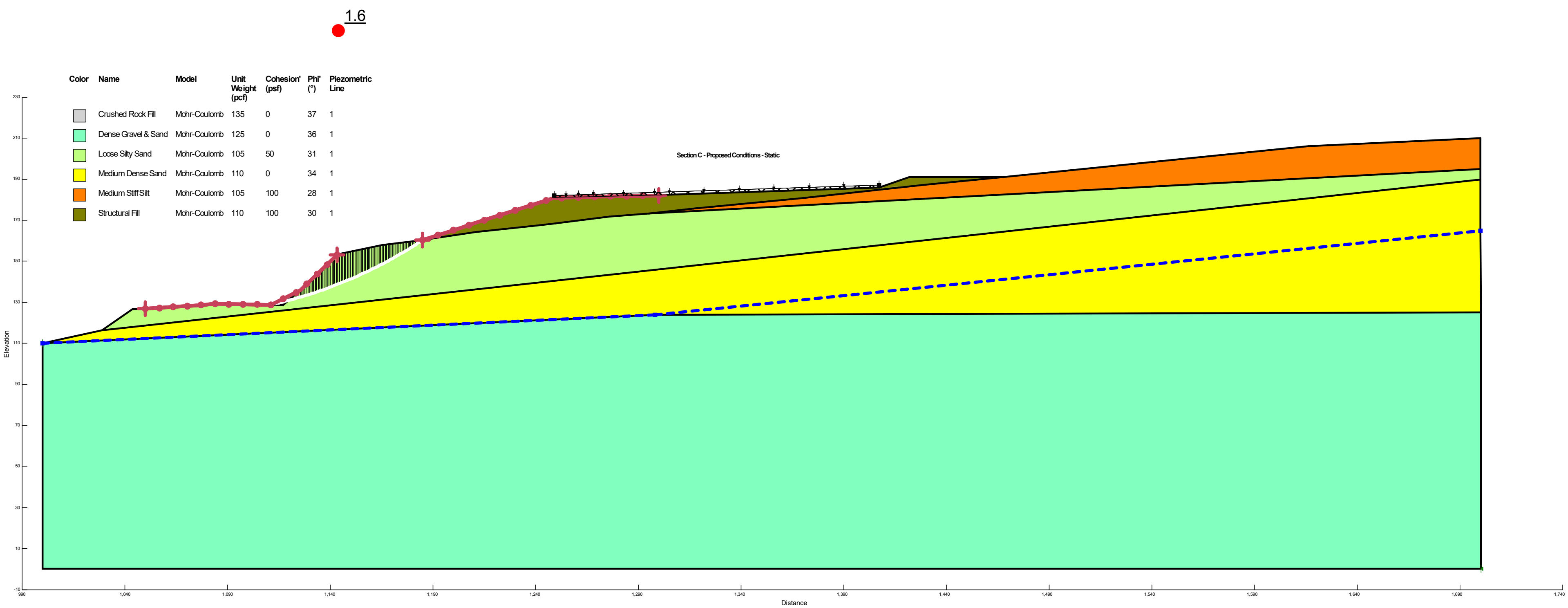
1.2
Section B - Proposed Conditions - Seismic
Color Name Model Unit Weight (pcf) Cohesion' (psf) Phi' (°) Piezometric Line
Dense Gravel & Sand Mohr-Coulomb 125 0 36 1
Loose Silty Sand Mohr-Coulomb 105 50 31 1
Medium Dense Sand Mohr-Coulomb 110 0 34 1
Medium Stiff Silt Mohr-Coulomb 105 100 28 1
Structural Fill Mohr-Coulomb 110 100 30 1
5,000
5,100
5,200
5,300
5,400
5,500
5,600
5,700
5,800
Distance



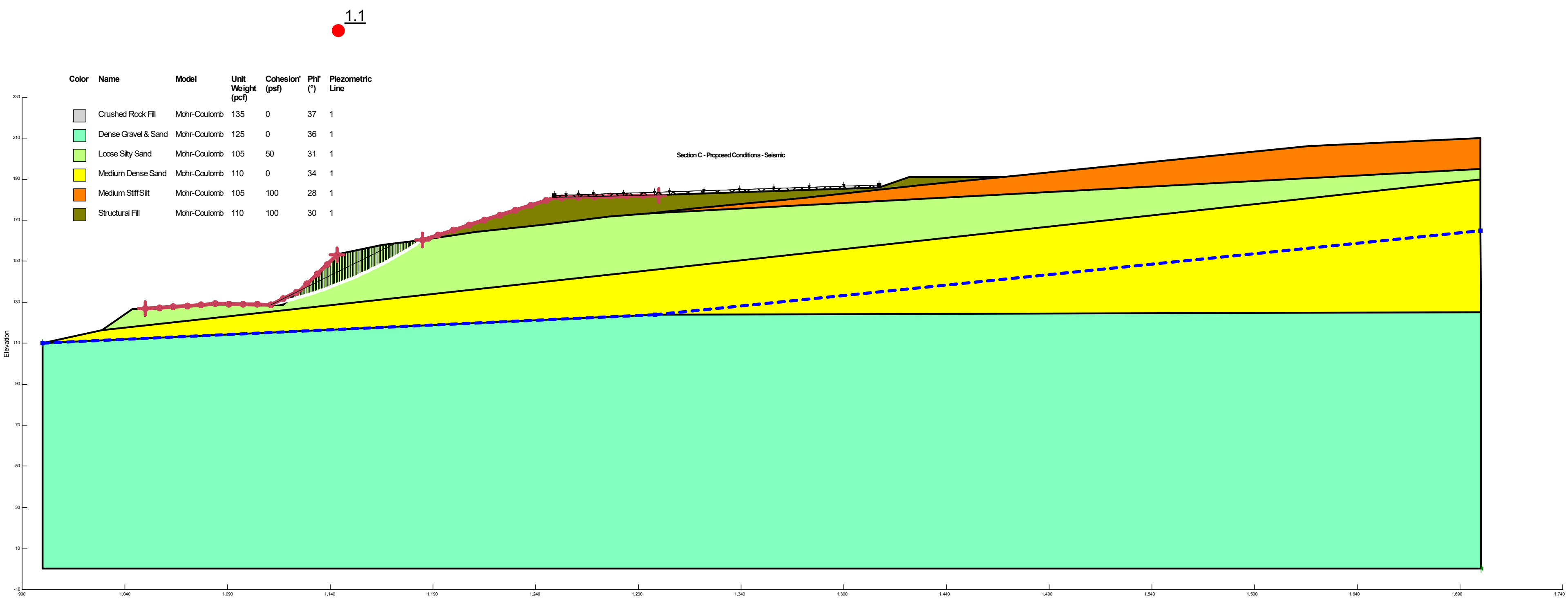
1.5
Section C - Existing Conditions - Static
Color
Name
Model
Unit Weight (pcf)
Cohesion' (psf)
Phi' (°)
Piezometric Line
Dense Gravel & Sand Mohr-Coulomb 125 0 36 1
Loose Silty Sand Mohr-Coulomb 105 50 31 1
Medium Dense Sand Mohr-Coulomb 110 0 34 1
Medium Stiff Silt Mohr-Coulomb 105 100 28 1
Elevation
Distance



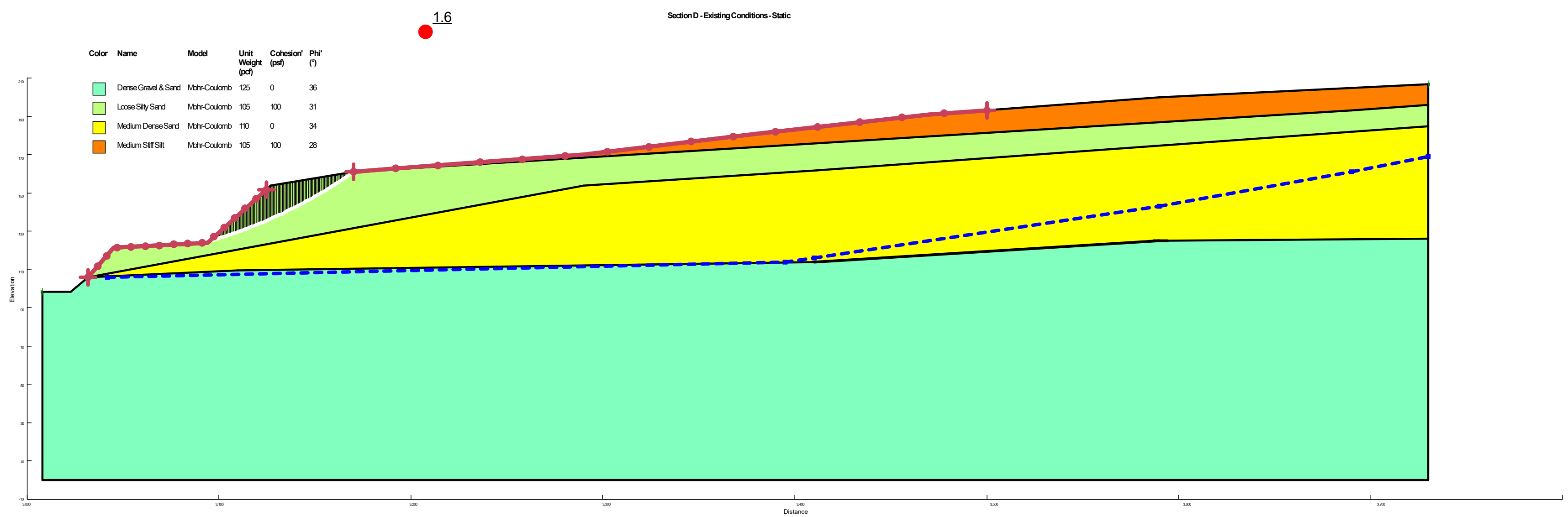
1.1
Section C - Existing Conditions - Seismic
Color | Name | Model | Unit Weight (pcf) | Cohesion' (psf) | Phi' (°) | Piezometric Line
Dense Gravel & Sand | Mohr-Coulomb | 125 | 0 | 36 | 1
Loose Silty Sand | Mohr-Coulomb | 105 | 50 | 31 | 1
Medium Dense Sand | Mohr-Coulomb | 110 | 0 | 34 | 1
Medium Stiff Silt | Mohr-Coulomb | 105 | 100 | 28 | 1
Elevation
Distance



1.6
Section C - Proposed Conditions - Static
Color Name Model Unit Weight (pcf) Cohesion' (psf) Phi' (°) Piezometric Line
Crushed Rock Fill Mohr-Coulomb 135 0 37 1
Dense Gravel & Sand Mohr-Coulomb 125 0 36 1
Loose Silty Sand Mohr-Coulomb 105 50 31 1
Medium Dense Sand Mohr-Coulomb 110 0 34 1
Medium Stiff Silt Mohr-Coulomb 105 100 28 1
Structural Fill Mohr-Coulomb 110 100 30 1
Elevation
230
210
190
170
150
130
110
90
70
50
30
10
-10
990
1,040
1,090
1,140
1,190
1,240
1,290
1,340
1,390
1,440
1,490
1,540
1,590
1,640
1,690
1,740
Distance



1.1
Section C - Proposed Conditions - Seismic
Color | Name | Model | Unit Weight (pcf) | Cohesion' (psf) | Phi' (°) | Piezometric Line
Crushed Rock Fill | Mohr-Coulomb | 135 | 0 | 37 | 1
Dense Gravel & Sand | Mohr-Coulomb | 125 | 0 | 36 | 1
Loose Silty Sand | Mohr-Coulomb | 105 | 50 | 31 | 1
Medium Dense Sand | Mohr-Coulomb | 110 | 0 | 34 | 1
Medium Stiff Silt | Mohr-Coulomb | 105 | 100 | 28 | 1
Structural Fill | Mohr-Coulomb | 110 | 100 | 30 | 1
Elevation
Distance



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Section D - Existing Conditions - Static
Color Name Model Unit Weight (pcf) Cohesion' (psf) Phi' (°)
Dense Gravel & Sand Mohr-Coulomb 125 0 36
Loose Silty Sand Mohr-Coulomb 105 100 31
Medium Dense Sand Mohr-Coulomb 110 0 34
Medium Stiff Silt Mohr-Coulomb 105 100 28
Elevation
210
190
170
150
130
110
90
70
50
30
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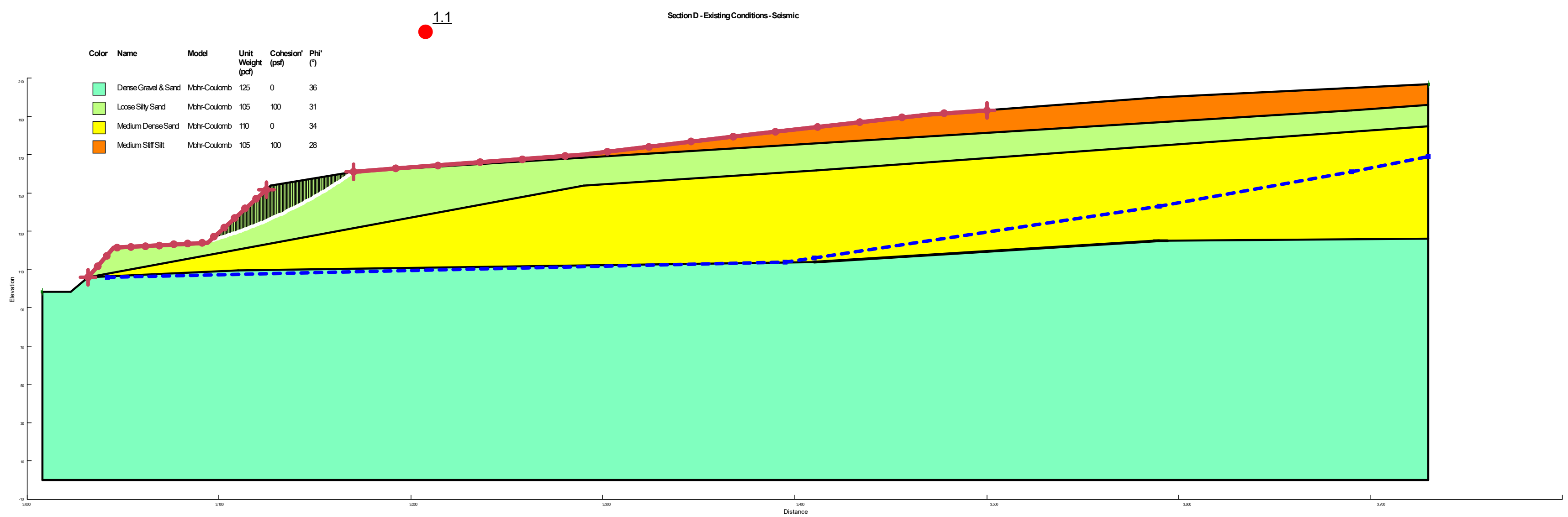
3,700
3,800
Distance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Section D - Existing Conditions - Seismic
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Model
Unit Weight (pcf)
Cohesion' (psf)
Phi' (°)
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125
0
36
Loose Silty Sand
Mohr-Coulomb
105
100
31
Medium Dense Sand
Mohr-Coulomb
110
0
34
Medium Stiff Silt
Mohr-Coulomb
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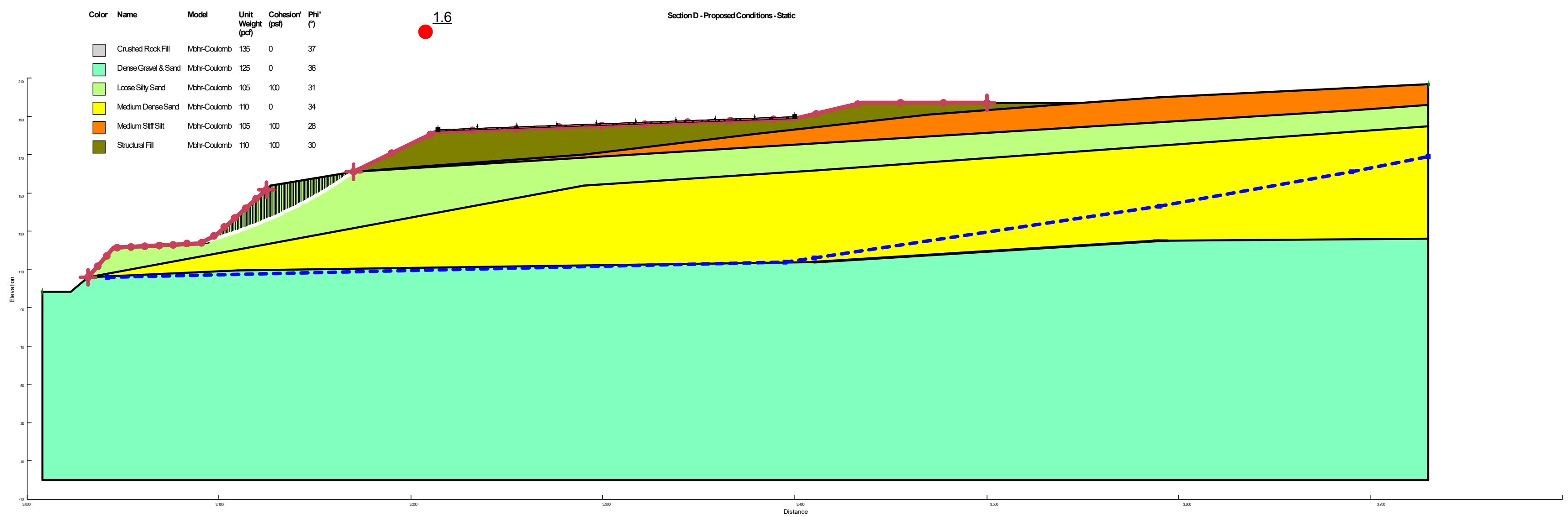
3,600
3,700
3,800



Section D - Proposed Conditions - Static
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Model
Unit Weight (pcf)
Cohesion' (psf)
Phi' (°)
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Dense Gravel & Sand Mohr-Coulomb 125 0 36
Loose Silty Sand Mohr-Coulomb 105 100 31
Medium Dense Sand Mohr-Coulomb 110 0 34
Medium Stiff Silt Mohr-Coulomb 105 100 28
Structural Fill Mohr-Coulomb 110 100 30
Elevation
Distance
3,000
3,100
3,200
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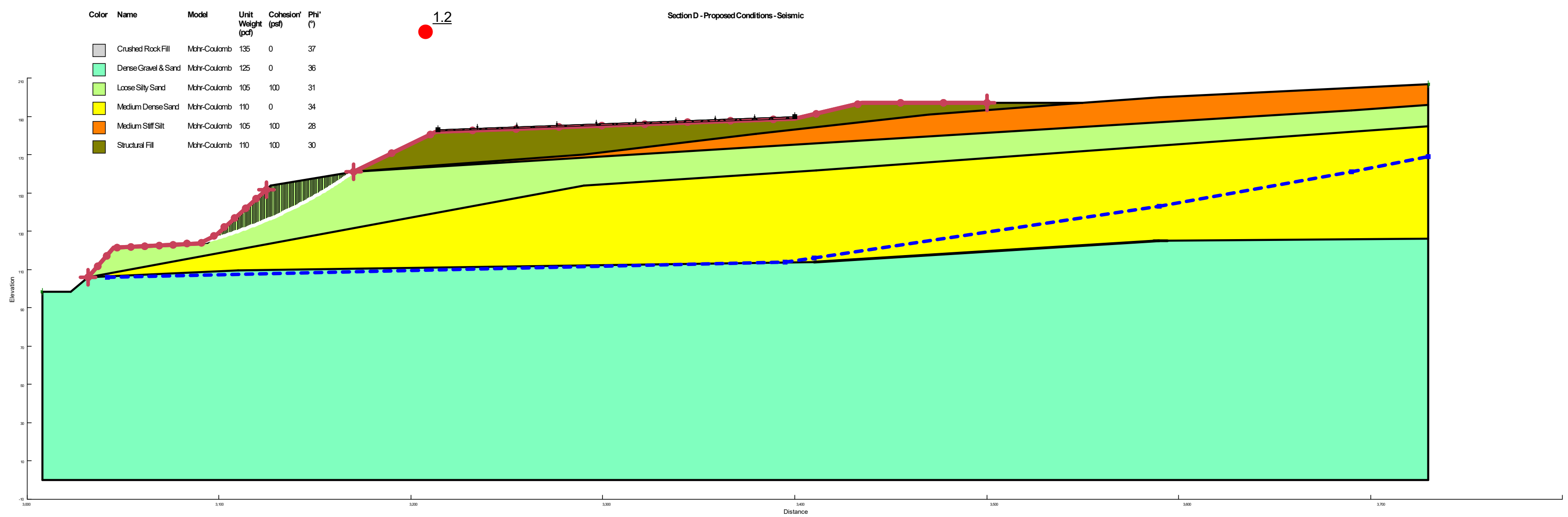
3,600
3,700
3,800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Model
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Cohesion' (psf)
Phi' (°)
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Loose Silty Sand Mohr-Coulomb 105 100 31
Medium Dense Sand Mohr-Coulomb 110 0 34
Medium Stiff Silt Mohr-Coulomb 105 100 28
Structural Fill Mohr-Coulomb 110 100 30
Elevation
Distance
3,000
3,100
3,200
3,300
3,400
3,500
3,600
3,700
3,800

APPENDIX F

APPENDIX F

SITE SPECIFIC SEISMIC HAZARD EVALUATION

INTRODUCTION

The appendix summarizes the results of the site-specific seismic hazard evaluation for the proposed Dollar Street middle school in West Linn, Oregon. The project consists of a one- and two-story, above-grade school. Based on experience, the fundamental period of the building will be less than approximately 0.5 second. This seismic hazard evaluation was performed in accordance with the requirements in the 2019 SOSSC and ASCE 7-16.

SITE CONDITIONS

REGIONAL GEOLOGY AND SUBSURFACE CONDITIONS

A detailed description of geologic and subsurface conditions at the site is presented in the main report.

SEISMIC SETTING

Earthquake Source Zones

Three scenario earthquakes are possible in the area. Two of the possible earthquake sources are associated with the CSZ, and the third event is a shallow, local crustal earthquake that could occur in the North American Plate. The three earthquake scenarios are discussed below.

Regional Events

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

1. An interface event earthquake on the seismogenic part of the interface between the Juan de Fuca Plate and the North American Plate on the CSZ. This source is reportedly capable of generating earthquakes with a moment magnitude of between 8.5 and 9.0.
2. A deep intraplate earthquake on the seismogenic part of the subducting Juan de Fuca Plate. These events typically occur at depths of between 30 and 60 km. This source is capable of generating an event with a moment magnitude of up to 7.5.

Local Events

A significant earthquake could occur on a local fault near the site within the design life of the facility. Such an event would cause ground shaking at the site that could be more intense than the CSZ events, although the duration would be shorter. Figure F-1 shows the locations of faults with potential Quaternary movement within a 40-km radius of the site (USGS, 2020). The most significant faults in the site vicinity are the Canby-Molalla fault, Oatfield fault, and Portland Hills fault. Figure F-2 shows the interpreted locations of seismic events that occurred between 1904 and 2020.

Canby-Molalla Fault

The nearest mapped Quaternary fault to the site is the Canby-Molalla fault. The mapped trace of the north-northwest-striking Canby-Molalla fault is based on a linear series of northeast-trending, discontinuous aeromagnetic anomalies that probably represent significant offset of Eocene basement and volcanic rocks of the Miocene CRBG beneath Neogene sediments that fill the northern Willamette River Basin. The fault has little geomorphic expression across the gently sloping floor of the Willamette Valley, but a small, laterally restricted berm associated with the fault may suggest young deformation. Deformation of probable Missoula flood deposits in a high-resolution seismic reflection survey conducted across the aeromagnetic anomaly east of Canby suggests possible Holocene deformation. Sense of displacement of the Canby-Molalla fault is poorly known, but the fault shows apparent right-lateral separation of several transverse magnetic anomalies, and down-west vertical displacement is also apparent in water well logs. The actual sense of displacement of the Canby-Molalla fault is poorly known. The fault shows apparent right-lateral separation of several transverse magnetic anomalies, and down-west vertical displacement is also apparent in water well logs (Blakely et al., 2001). Given the compressional setting of other faults in the area and lack of significant topographic expression (Blakely et al., 2001), the fault probably is a right-lateral, strike-slip fault with lesser amounts of reverse displacement.

Bolton Fault

The Bolton fault is mapped trending northwest to southeast along the base of the south Portland Hills. The Bolton fault forms a prominent northeast-facing topographic ridge trending parallel to the west side of the Willamette River between Lake Oswego and Oregon City. The location of the fault is based on reported vertical offset of approximately 500 to 650 feet of Miocene Age (20 million to 10 million years before present) basalt flows belonging to the CRBG (Personius, 2002a; Beeson et al., 1991; Madin, 1990). The general sense of movement is inferred to be a steeply dipping, southwest-facing, reverse fault with some strike-slip component. Fault offset of Quaternary deposits, or other unequivocal evidence of Quaternary displacement have not been conclusively documented with the Bolton fault. In our opinion, the published data on the Bolton fault is not a definitive indicator of a significant seismic source.

Oatfield Fault

The northwest-striking Oatfield fault forms northeast-facing escarpments in volcanic rocks of the Miocene CRBG in the Tualatin Mountains and northern Willamette Valley. The fault may be part of the Portland Hills-Clackamas River structural zone. The Oatfield fault is primarily mapped as a very high-angle, reverse fault with apparent down-to-the-southwest displacement, but a few kilometer-long reach of the fault with down-to-the-northeast displacement is mapped in the vicinity of the Willamette River. This apparent change in displacement direction along strike may reflect a discontinuity in the fault trace or could reflect the right-lateral, strike-slip displacement that characterizes other parts of the Portland Hills-Clackamas River structural zone. The fault has also been modeled as a 70-degree, east-dipping reverse fault. Reverse displacement with a right-lateral, strike-slip component is consistent with the tectonic setting, mapped geologic relations, and microseismicity in the area. Fault scarps on surficial deposits have not been described, but exposures in a light rail tunnel showing offset of approximately 1 M_a Boring Lava across the fault indicate Quaternary displacement (Personius, 2002b).

Portland Hills Fault

The northwest-striking Portland Hills fault forms the prominent linear northeast margin of the Tualatin Mountains (Portland Hills) and the southwest margin of the Portland Basin; this basin may be a right-lateral, pull-apart basin in the forearc of the CSZ or a piggyback synclinal basin formed between antiformal uplifts of the Portland fold belt. The fault is part of the Portland Hills-Clackamas River structural zone, which controlled the deposition of Miocene CRBG lavas in the region. The crest of the Portland Hills is defined by the northwest-striking Portland Hills anticline. Sense of displacement on the Portland Hills fault is poorly known and controversial. The fault was originally mapped as a down-to-the-northeast normal fault. The fault has also been mapped as part of a regional-scale zone of right-lateral oblique slip faults and as a steep escarpment caused by asymmetrical folding above a southwest-dipping blind thrust. Reverse displacement with a right-lateral, strike-slip component may be most consistent with the tectonic setting, mapped geologic relations, aeromagnetic data, and microseismicity in the area. Fault scarps on surficial Quaternary deposits have not been described along the fault trace, but some geomorphic (steep, linear escarpment, triangular facets, over-steepened, and knick-pointed tributaries) and geophysical (aeromagnetic, seismic reflection, and ground penetrating radar) evidence suggest Quaternary displacement (Personius, 2017).

Table F-1. Significant Crustal Faults

Source	Closest Mapped Distance[1] (km)	Mapped Length[1] (km)	Estimated Slip Rate[1] (mm/yr)
Canby-Molalla fault	2.0	50	<0.2
Oatfield fault	7.5	24	<0.2
Portland Hills fault	9.0	49	<0.2

1. Reported by USGS (USGS, 2020)
2. Slip rates of all faults are less than 1 mm/yr and the site is not considered near-fault per ASCE 7-16 – 11.4.1.

GROUND RESPONSE

GENERAL

Design levels of ground shaking were determined using site response analyses. The following sections explain determination of a target bedrock spectrum and the site response analysis.

TARGET BEDROCK SPECTRUM

The target bedrock spectrum was taken as the spectrum corresponding to a shear wave velocity of approximately 2,500 fps (Site Class B). It was determined using the USGS Unified Hazard tool (https://earthquake.usgs.gov/hazards/interactive/) and the latitude and longitude of the site (45.348100, -122.671839). The target bedrock spectrum is provided in Table F-2.

Table F-2. Target Spectrum

Period (seconds)	MCE Target Bedrock Spectral Acceleration (g)
0.01	0.404
0.1	0.870
0.2	0.896
0.3	0.735
0.5	0.528
0.75	0.400
1.0	0.319
2.0	0.175
3.0	0.115
4.0	0.086
5.0	0.065

GROUND MOTIONS

Six recorded base ground motions were selected to represent the local seismic setting. Based on deaggregation at the assumed fundamental period range of the building (approximately 0.25 to 0.5 second), the CSZ controls approximately 50 percent of the seismic hazard and the crustal events control the majority of the remaining hazard. Accordingly, three CSZ and three crustal ground motions were used for analysis. Table F-3 provides the ground motions selected for this study.

Table F-3. Selected Ground Motions

Ground Motion/Recording Station	Magnitude	Distance (km)	Component
CSZ Zone Records			
Tohoku – D2E	9.0	56.3	EW
Maule – LACH	8.8	81.9	NS
Tokachi-oki – HKD092	8.29	70.98	EW
Crustal Zone Records			
Chi-Chi, Taiwan – TCU076	7.62	2.74	E
Imperial Valley – El Centro Array #4	6.53	1.0	140
Chuetsu-oki, Japan - Joetsu Kakizakiku Kakizak	6.8	9.4	NS

Crustal records were obtained from earthquakes between M_w 6.5 and 7.7 and recording stations within 1 kilometer to 10 kilometers of the causative faults to match the seismic setting at the site. CSZ motions were selected from earthquakes with magnitudes greater than M_w 8.2.5 and distances between 50 and 85 km to simulate the M_w 9.0 CSZ interface event. Care was taken to select records with spectra similar to the target spectrum to reduce modification during spectral matching. Ground motions in Table F-3 were spectrally matched target spectrum using the RspMatch 2009 algorithm by Norm Abrahamson and Linda Al Atik (embedded in the EZ-FRISK 8.06 software application).

SITE RESPONSE ANALYSIS

The acceleration response spectrum for the site was determined by performing a site response analysis using the one-dimensional equivalent linear program Shake 91+ included in the EZ-FRISK 8.06 computer program. The input soil models used in analysis are based on explorations and our experience in the site vicinity. A detailed description of site subsurface conditions is provided in the main report. The soil parameters used in analysis are shown in Table F-4.

Table F-4. Input Soil Profile

Depth Interval (feet)	Subsurface Unit	Shear Wave Velocity (fps)	Modulus Reduction Curve	Damping Curve
0 to 7	Silt	500 to 600	Vucetic and Dobry, 1991	Vucetic and Dobry, 1991
7 to 25	Silty Sand	600 to 700	EPRI 0-20 EPRI 20-50	EPRI 0-20 EPRI 20-50
25 to 50	Silty Sand with Silt Zones	700 to 900	EPRI 20-50	EPRI 20-50
50 to 65	Silty Sand to Sand	900	EPRI 50-120	EPRI 50-120
65 to 145[1]	Gravel	1,100 to 1,300	Seed (et al. 1986)	Seed (et al. 1986)

Output spectrum at the ground surface
1. Input ground motion is at a depth of 145 feet BGS at the assumed contact with basalt.

Because the ground motion models used in the hazard calculation compute the average horizontal component of ground motions, scale factors were applied to adjust the site response results to the maximum rotated component as described in ASCE 7-16 (C21.2). According to ASCE 7-16 Supplement 1, a scale factor of 1.1 should be used for periods of 0.2 second and shorter, a scale factor of 1.3 should be used for periods of 1.0 second, and a scale factor of 1.5 was used for periods greater than 5 seconds. Linear interpolation was used to compute factors between 1 second and 5 seconds.

The results of the site response were also modified with risk coefficients using Method 2 outlined in ASCE 7-16 Section 21.2.1.2. A risk coefficient of C_{RS} = 0.890 was applied to the spectrum at periods of 0.2 second or less and a risk coefficient of C_{R1} = 0.865 was applied to the spectrum at periods greater than 1 second. Linear interpolation was used to compute risk coefficients between periods of 0.2 and 1.0 second. The intent of this is to achieve a 1 percent collapse of the structure in a 50-year period.

The acceleration response spectra produced from analysis is shown on Figure F-3.

PROBABILISTIC MCE$_R$ RESPONSE SPECTRUM

The PSHA MCE$_R$ is shown on Figure F-3.

DETERMINISTIC MCE$_R$ RESPONSE SPECTRUM

ASCE 7-16 Section 21.2.2 (Supplement 1) requires the DSHA spectral response period acceleration at each period is calculated as an 84th percentile, 5 percent damped spectral response acceleration in the direction of maximum horizontal response computed at each period. Per the exception in Section 21.2.2, the deterministic ground motion response spectrum need not be calculated when the largest spectral response acceleration of the probabilistic ground motion response spectrum in Section 21.2.1 (PSHA MCE$_R$) is less than 1.2F$_a$.

The largest spectral response acceleration of the PSHA MCE$_R$ is 0.87 g at 0.31 second. The project-specific F$_a$ from ASCE 7-16 (Site Class D) is 1.16, resulting in 1.2F$_a$ equal to 1.392 g. Because 1.2F$_a$ is larger than largest spectral response acceleration from the PSHA MCE$_R$, a deterministic ground motion response is not required for the project.

SITE-SPECIFIC MCE$_R$ RESPONSE SPECTRUM

As outlined in ASCE 7-16 Section 21.2.3, the site-specific MCE$_R$ shall be taken as the lesser of the probabilistic MCE$_R$ and the deterministic MCE$_R$. Because a deterministic ground motion response is not required, the site-specific MCE$_R$ is the PSHA MCE$_R$ on Figure F-3.

DESIGN RESPONSE SPECTRUM

In accordance with ASCE 7-16 Section 21.3, the design response spectrum is two-thirds of the MCE$_R$ at all periods. However, the lower bound of the design response spectrum is 80 percent of the generalized response spectrum as outlines in ASCE 7-16 Section 21.3.

Exception 1613.4.13 in SOSSC 1613.4.13 allows the value of F$_v$ for Site Classes D and E to be determined based on the relative hazard contribution from the CSZ. The relative contribution is determined using the USGS Unified Hazard deaggregation hazard tool at 1.0 second. Calculation of F$_v$ is determined using the equation below:

> *F$_v$ = (% Contribution of CSZ x F$_v$ from SOSSC Table 1613.2.3 (2)) + (% Contribution from non-CSZ sources x Fv per ASCE-7-16 - Section 21.3)*

The percent contribution from the CSZ at 1.0 second is 78 percent and the resulting F$_a$ and F$_v$ values used for the site were F$_a$ = 1.16 (from ASCE 7-16 Table 11.4-1) and F$_v$ = 2.023, respectively.

DESIGN ACCELERATION PARAMETERS

The parameter S$_{DS}$ is taken as 90 percent of the maximum spectral acceleration from the site-specific design response spectrum at any period within the range from 0.2 second to 5.0 seconds. The parameter S$_{D1}$ is taken as the maximum value of the product, TS$_a$, for periods from 1.0 second to 5.0 seconds for sites with Vs$_{30}$ less than 1,200 fps. Figure F-4 shows the design response spectrum for the site.

ASCE 7-16 requires the values of S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1} but not be less than 80 percent of the values determined in accordance with ASCE 7-16 and SOSSC described above. The resulting site-specific design parameters are as follows:

- S_{DS} = 0.527 g
- S_{D1} = 0.408 g
- S_{MS} = 0.791 g
- S_{M1} = 0.612 g

GEOLOGIC HAZARDS

FAULT SURFACE RUPTURE

The nearest conclusively active mapped fault is approximately 2 km from the site. Consequently, it is our opinion that the probability of fault surface rupture beneath the site is low.

LIQUEFACTION AND LATERAL SPREADING

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressure can dissipate. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Soil susceptible to liquefaction was not encountered in the explorations.

Based on the soil and groundwater conditions at the site, liquefaction and lateral spreading hazards are not design considerations at the site.

GROUND MOTION AMPLIFICATION

Soil capable of significantly amplifying ground motions beyond the levels determined by our site-specific seismic study were not encountered during the subsurface investigation program. The main report provides a detailed description of the subsurface conditions encountered.

LANDSLIDE

The main geotechnical report includes a summary of our slope stability analyses and provides grading, wall, and drainage recommendations to satisfy slope stability requirements for the site development. The existing steep slopes, which will remain undisturbed at the site, will continue to have some risk of shallow localized failures.

SETTLEMENT

Settlement due to earthquakes is most prevalent in relatively deep deposits of dry, clean sand. We do not anticipate that seismic-induced settlement in addition to liquefaction-induced settlement will occur during design levels of ground shaking.

SUBSIDENCE/UPLIFT

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

LURCHING

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

SEICHE AND TSUNAMI

The site is inland and elevated away from tsunami inundation zones and away from large bodies of water that may develop seiches. Seiches and tsunamis are not considered a hazard in the site vicinity.

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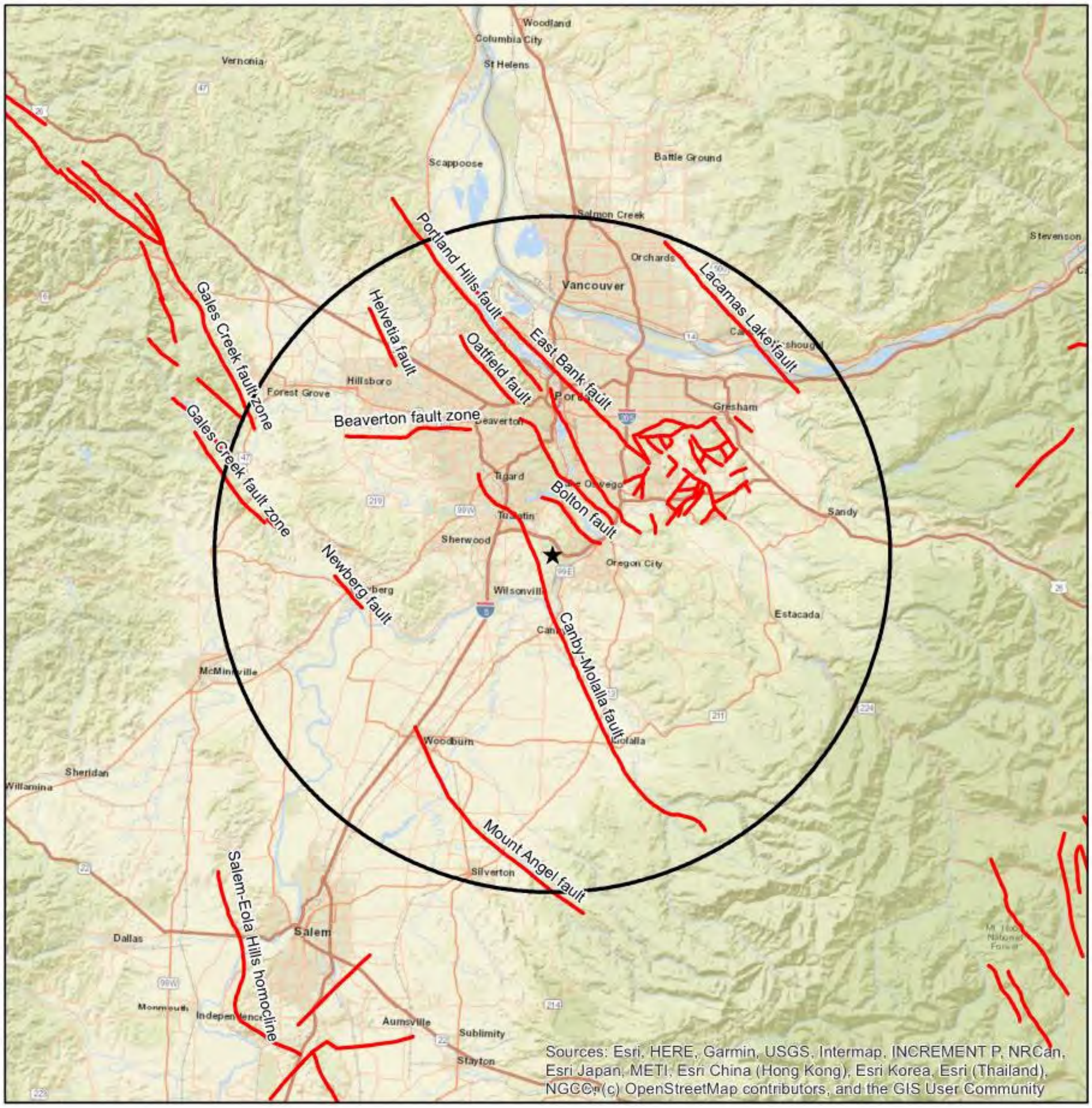
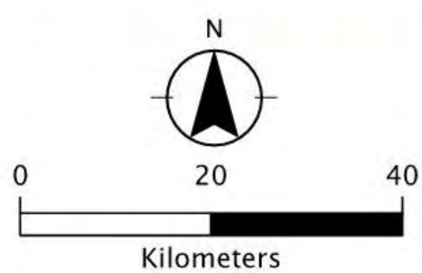


LEGEND

- RADIUS
- ★ SITE LOCATION
- USGS QUATERNARY FAULTS



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GEODESIGN INC AN NV5 COMPANY	WLWSCHDIST-1-01	QUATERNARY FAULT MAP	
	OCTOBER 2020	DOLLAR STREET MIDDLE SCHOOL WEST LINN, OR	**FIGURE F-1**

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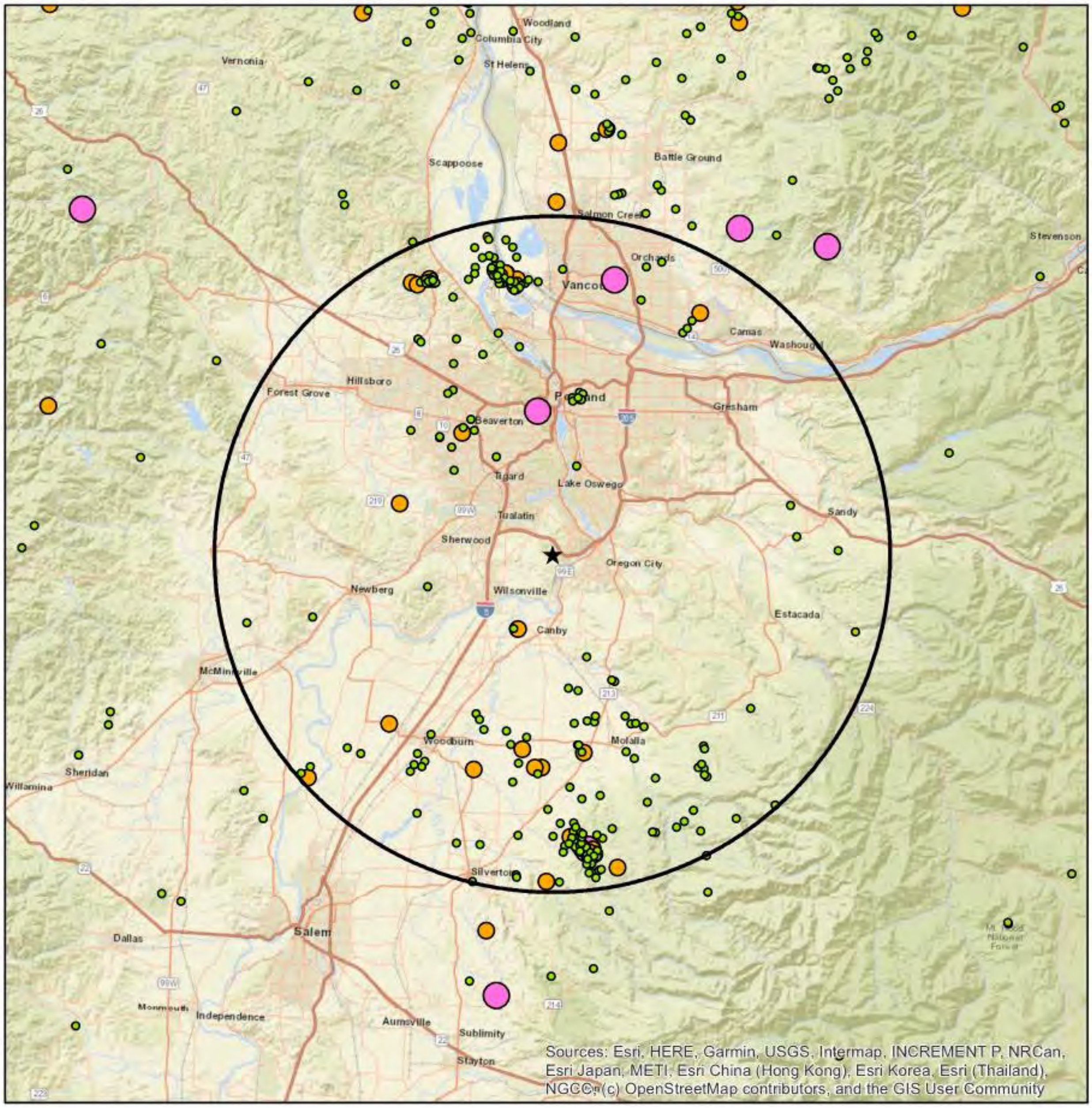


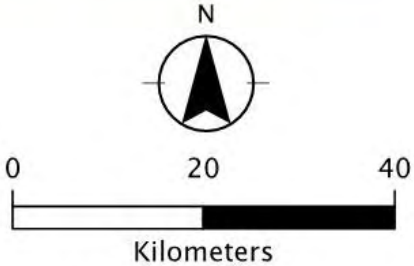
LEGEND

RADIUS

★ SITE LOCATION

INSTRUMENTAL EARTHQUAKE MAGNITUDE

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



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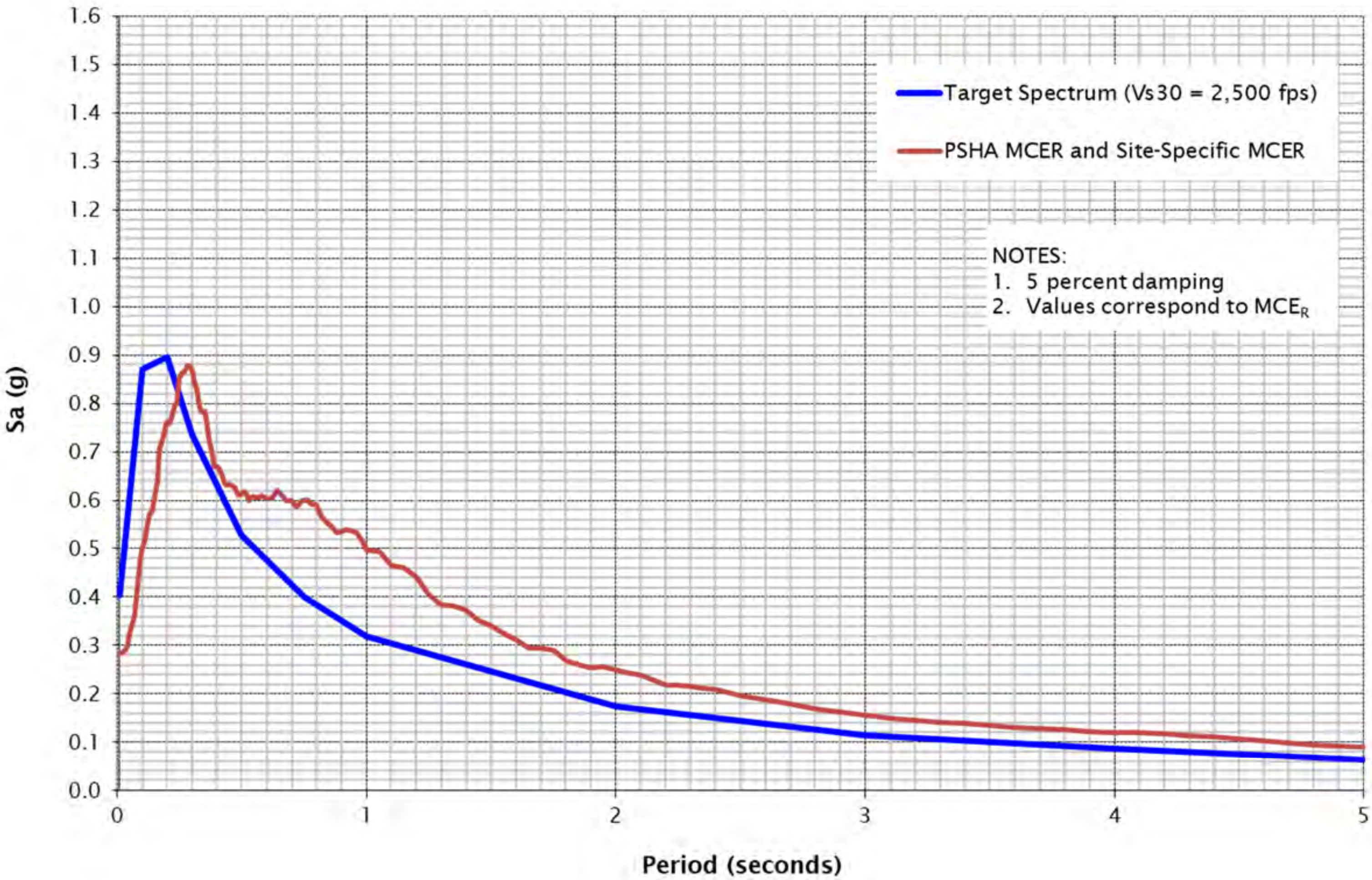
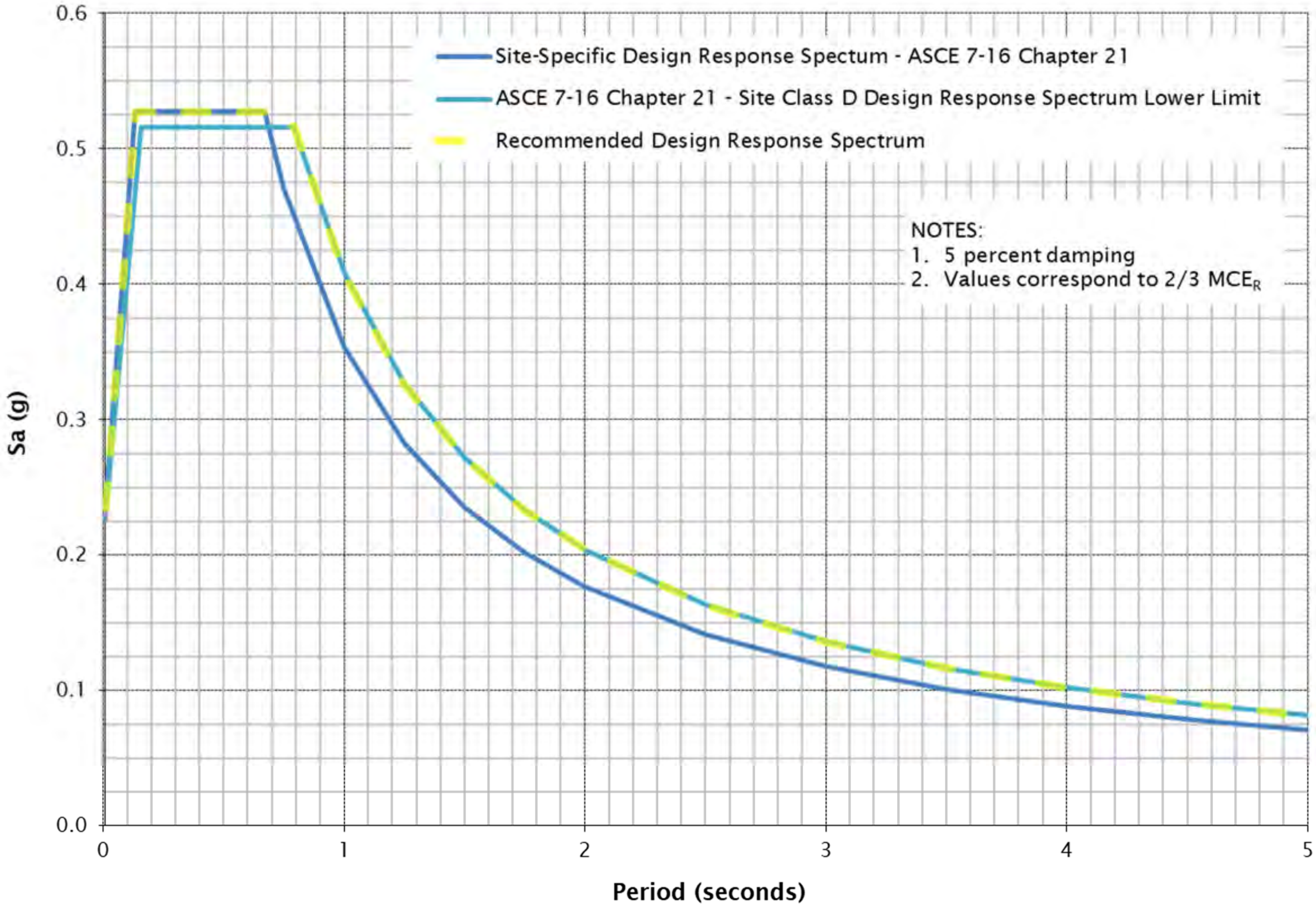
Earthquakes in figure are from 1904-2020.

GEODESIGN INC AN NV5 COMPANY	WLWSCHDIST-1-01	**HISTORICAL SEISMICITY MAP**	
	OCTOBER 2020	DOLLAR STREET MIDDLE SCHOOL WEST LINN, OR	**FIGURE F-2**

WLWSchDist-1-01-FF1_F2.docx Print Date: 10/19/20



GEODESIGN INC AN NV5 COMPANY	WLWSCHDIST-1-01	**SITE RESPONSE SPECTRA**
	OCTOBER 2020	DOLLAR STREET MIDDLE SCHOOL WEST LINN, OR — **FIGURE F-3**



GEODESIGN INC
AN NV5 COMPANY

	WLWSCHDIST-1-01	**DESIGN RESPONSE SPECTRUM**	
	OCTOBER 2020	DOLLAR STREET MIDDLE SCHOOL WEST LINN, OR	**FIGURE F-4**