REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Seaside School District Campus
Seaside, Oregon

For
Seaside School District
November 20, 2017

GeoDesign Project: SeasideSD-1-03-01

GEODESIGN INC

November 20, 2017

Seaside School District, Business Office
1801 South Franklin Street
Seaside, OR 97138

Attention: Justine Hill

Report of Geotechnical Engineering Services
Seaside School District Campus
Seaside, Oregon
GeoDesign Project: SeasideSD-1-03-01

GeoDesign, Inc. is pleased to submit this report of geotechnical engineering services for the planned Seaside School District Campus east of the Seaside Heights Elementary School expansion at 2000 Spruce Drive in Seaside, Oregon. The planned campus includes a new middle/high school building and associated parking, infrastructure, an access driveway, and an athletic field. Our services for this project were conducted in accordance with our scope and updated fee dated October 4, 2017.

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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1.0 INTRODUCTION

GeoDesign, Inc. is pleased to present this geotechnical engineering report for the planned Seaside School District Campus to be located east of the existing Seaside Heights Elementary School. The planned campus includes a new middle/high school building and associated parking, infrastructure, an access driveway, and an athletic field. Figure 1 shows the site location relative to existing topographic and physical features. The approximate locations of the planned new developments are shown on Figure 2.

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 include up to 20 feet for the parking area west of the new middle/high school building, up to 30 feet for the access road south of the track and field, and up to 25 feet for the west end of the track and field. The largest proposed cuts include up to 15 feet for the east end of the track and field, up to 15 feet for the proposed school building, 15 feet for the east end of the parking lot above the proposed school building, and up to 25 feet for the proposed detention pond near the west end of the campus area.

Acronyms and abbreviations used herein are defined at the end of this document.

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:

- Coordinated and managed the field evaluation, including utility checks, site access, and scheduling of subcontractor and GeoDesign field staff.
- Reviewed prior geotechnical and geological reports and information available for the site.
- Obtained an Oregon Forestry Department permit to complete explorations during the fire season, following fire season restrictions (including having a water trailer, pump, and 500 feet of fire hose at the site), and cleared paths through the ground debris for access to the exploration locations.
- Obtained readings on all the slope inclinometers at the site and installed three data loggers to record groundwater level measurements on existing piezometers from spring to summer of 2017.
- Completed the following subsurface explorations at the site:
  - Drilled five borings to the depths ranging between 36.3 and 81.5 feet BGS.
  - Advanced six CPT probes to depths ranging between 44.3 and 77.3 feet BGS (practical refusal) and obtained shear wave velocity measurements at 2-meter intervals in one of the CPT probes near the proposed building area.
  - Excavated 23 test pit explorations to depths ranging between 5.0 and 12.5 feet BGS.
  - Installed one new vibrating wire piezometer near the proposed new building location.
- Collected disturbed and undisturbed soil samples for laboratory testing at select depths from the explorations.

- Classified the materials encountered in, and maintained a detailed log of, each exploration.
- Completed the following laboratory tests on select soil samples:
    - Eighty-three moisture content determinations in general accordance with ASTM D 2216
    - Three dry density determinations in general accordance with ASTM D 7263
    - Five Atterberg limits tests in general accordance with ASTM D 4318
    - One grain-size analysis in general accordance with ASTM C 136 and ASTM C 117
    - One swell test and consolidation test in accordance with ASTM D 4546 and ASTM D 2435
    - Two sets of direct shear tests in general accordance with ASTM D 3080
    - One expansion index tests in general accordance with ASTM D 4829 with a swell test in accordance with ASTM D 4546.
    - One set of torsional residual shear tests in general accordance with ASTM D 6467
- 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.
- Provided recommendations for design and construction of a mat foundation.
- Conducted slope stability analyses for cut and fill slopes at the site, and evaluated the potential for massive slope movement at the contact with the underlying Astoria Formation from seismic loading.
- Provided recommendations for temporary and permanent cut and fill slopes.
- Provided recommendations for preparation of subgrades.
- Provided design criteria recommendations for retaining walls, including lateral earth pressures, backfill, compaction, and drainage.
- Provided recommendations for the management of identified groundwater conditions that may affect the performance of structures or pavements.
- Provided recommendations for construction of AC pavements for on-site access roads and parking areas, including subbase, base course, and AC paving thickness.
- Provided recommendations for subsurface drainage of foundations and roadways, as necessary.
- Provided recommendations for IBC seismic coefficients.
- Prepared this report summarizing the results of our geotechnical evaluation.
- Prepared a site-specific seismic hazard study for the site, which is presented in an appendix of this report.

3.0 BACKGROUND

GeoDesign previously prepared a geological assessment (GeoDesign, 2009) of potential development sites for the Seaside School District and Providence Health System. We also prepared an update to the geological assessment (GeoDesign, 2012) that focused on the current site above the Seaside Heights Elementary School, and updated the geologic mapping for the site using LiDAR imaging and data. The majority of the slopes above Seaside and the entire vicinity of the site are mapped as landslide topography. Through our geologic mapping and subsequent geotechnical reports for the site, we identified smaller slide scarps along many of the incised drainages at the site and recommended setbacks from the slide features.

We have also reviewed a prior 2012 preliminary geotechnical report (Geocon Northwest, Inc. [Geocon], 2012) and 2013 report update (Geotechnical Resources, Inc. [GRI], 2013) for the site.

The 2012 preliminary report included a series of explorations and slope inclinometers near the upper end of the site and a boring and slope inclinometer near the Seaside Heights Elementary School. The 2013 report update included another boring, slope inclinometer, and test pits on the slope above Seaside Heights. Piezometers were also installed at various depths in the four slope inclinometer installations. The report update indicates no evidence was observed to suggest the hillside has undergone significant movement in recent geologic past, which is in agreement with our observations. The 2013 report update also recommends installing a drainage trench or trenches to intercept and lower the groundwater surface with the eastern (higher elevation) portion of the property, which is consistent with the drainage recommendations provided in this report.

4.0 SITE CONDITIONS

4.1 GEOLOGIC CONDITIONS

The site is located on the eastern edge of the Northern Oregon Coastal Plain that resides on the western flank of the Coast Range physiographic province. The Northern Oregon Coastal Plain is composed of a series of marine terraces flanked by ocean beaches to the west and Coast Range uplands to the east. The marine terraces represent wave-cut platforms formed on Tertiary marine sedimentary and volcanic bedrock by Pleistocene sea level fluctuations. The terraces were subsequently covered by near-shore and terrestrial deposits and soils. The marine terraces have been tectonically uplifted and faulted to their present position and deeply weathered and incised by coastal streams.

The site covers several near-surface geologic units, including Tertiary marine sedimentary bedrock consisting of the Cannon Beach Member of the Astoria Formation, volcanic flows, and Quaternary terrace deposits and alluvium (Schlicker et al., 1973; Niem and Niem, 1985). During the early Miocene (15 million to 20 million years before present), the Astoria Formation was deposited in a marine sedimentary basin located near the mouth of the ancient Columbia River. The Astoria Formation consists of a thick assemblage of marine shelf deposits that include mudstones, siltstones, and sandstones. The Astoria Formation identified by Niem and Niem (1985) in the project area is dominated by siltstone and mudstone units.

During the middle Miocene (approximately 14.5 million years before present), basalt lava of the CRBG flowed down the ancient Columbia River drainage valley and entered the eastern edge of the marine sedimentary basin. The Frenchman Springs unit of the CRBG flowed onto and intruded into the soft, unconsolidated marine sediments of the Astoria Formation, forming subaerial lava flows and intrusive sills and dikes within the sediments. Contact of the ocean water and flowing lava formed breccias, pillow palagonite lava complexes, and hyaloclastites that overlie or intrude sedimentary rock of the Astoria Formation.

The Coast Range was uplifted and deeply eroded, forming an unconformity during the late Miocene to Pliocene time (approximately 11 million to 2 million years ago). Pleistocene (2 million to 10,000 years before present) sea level fluctuations coupled with slow Coast Range uplift formed multiple wave-cut terraces into the CRBG basalts and Astoria Formation. The terraces were subsequently covered by near-shore beach and terrestrial deposits, primarily colluvial soil. Westerly flowing streams have incised the terraces, forming isolated benches separated by deep ravines. Stream incision and erosion has resulted in active landslides and unstable slopes located on the steep side banks of these stream drainages. In addition, uplift and erosion has resulted in instability of steep slopes underlain by weak bedrock units and thick soil deposits. A massive ancient landslide scarp with a north-south trend is mapped roughly ,1500 to 2,500 feet east of the upper east edge of the site (Burns and Watzig, 2014). The ancient landslide likely resulted from different conditions than currently present, as indicated by subdued landslide features in the area.

4.2 SURFACE CONDITIONS

The site is located upslope to the east of the existing Seaside Heights Elementary School, which is at the east end of Spruce Drive. The site extends roughly 3,100 feet east of the elementary school site. Elevations range from roughly 70 feet at the level area of the elementary school up to 415 feet at the east end of the site. At the east end of the site a gravel road from the south forks into two roads that extend further north. A gravel logging access road extends down from the eastern road and branches out into the site. The site generally slopes gently to the west at an average slope of approximately 6 to 6.5 degrees with moderate to steep slopes along incised somewhat meandering drainage channels. A steeper cut slope of approximately 2.5H:1V to 3H:1V is also located to the northeast of the elementary school at the west end of the site. Several smaller drainages originate on the site and converge with the larger, more deeply incised drainage channels along the north and south borders of the site. The remnant of an older unimproved road, likely from prior logging operations, crosses the drainage along the southern edge of the site and traverses the lower portion of the site. Most of the site was heavily forested until last summer when it was logged prior to our field explorations. At the time of our field explorations the site was covered with slash and wood debris, including sticks, branches, logs, and stumps, after the recent logging operations.

Signs of recent or active deep-seated landslide features were not observed at the site or nearby areas during our studies or prior studies by others. Landslide scarps and potentially unstable slopes located along the drainage ravines, which were identified based on our 2009 site reconnaissance and 2012 LiDAR review are shown on Figure 3. These failures appear to result from stream incision and slope undercutting, which causes localized instability.

4.3 SUBSURFACE CONDITIONS

4.3.1 General

We explored subsurface conditions by drilling five borings (B-1 through B-5) to depths ranging between 36.3 and 81.5 feet BGS, excavating 23 test pits (TP-1 through TP-22 and TP-26) to depths ranging between 5.0 and 12.5 feet BGS, and advancing six CPT probes (CPT-2 through CPT-7a) to refusal depths ranging between 44.3 and 77.3 feet BGS. Other explorations completed near the east end of the site for the proposed water reservoir by GeoDesign included three borings (B-9 through B-11), three test pits (TP-23 through TP-25), and a CPT probe (CPT-1). Past explorations completed at or near the site by others included borings and test pits. The approximate exploration locations are shown on Figure 2. The boring and test pit logs and results of the laboratory testing completed at the site by GeoDesign are presented in Appendix A. The CPT log is presented in Appendix B. Expansion index testing and drained residual torsional direct shear testing results conducted on samples collected from our explorations are presented in Appendix C. Logs of GeoDesign's nearby explorations are presented in Appendix D. Logs of past explorations and associated laboratory testing completed by others are presented in Appendix E.

In general, subsurface conditions consist of colluvium and ancient landslide debris overlying weathered claystone of the Astoria Formation. Localized areas of fill were also encountered. The location of our geologic cross section through the site is shown on Figure 4 and our interpreted subsurface profile is shown in the geologic cross section on Figure 5.

The landslide debris is primarily comprised of decomposed to weathered Astoria Formation soils and some decomposed to weathered intruded pillow basalt. We encountered topsoil zones ranging up to 2.5 feet thick and surficial root zones ranging up to 10 inches thick. After removal of the wood debris on the surface we anticipate required stripping depths will typically vary from approximately 1 foot to 2 feet deep at the site. The following sections provide a more detailed description of the subsurface conditions encountered.

4.3.2 Undocumented Fill

Undocumented fill presumably from a remnant logging road was encountered from the ground surface to 9.0 feet BGS in boring B-4. The fill consists of soft, sandy silt. Undocumented fill from other past logging roads should be expected in isolated areas of the site.

4.3.3 Colluvium/Ancient Landslide Debris

Colluvium and ancient landslide debris comprise most of the soils encountered at the site. Colluvium was generally encountered at shallow depths and is predominantly indistinguishable from the ancient landslide debris. The ancient landslide debris likely includes Astoria Formation soils/sedimentary rock and some intruded decomposed pillow basalt. The colluvium and landslide debris soils generally consist of medium stiff to hard clay with variable fractions of silt, sand, and gravel. Both silt and clay were encountered in the test pit explorations.

Loose to very dense sand and gravel were also encountered at the site, particularly in boring B-2 where silty sand and silty gravel from decomposed pillow basalt was encountered to a depth of 74.0 feet BGS. Basalt sand, gravel, cobbles, and boulders, originating from intruded basalt lava flows, were also encountered in a clay or silt matrix resembling the Astoria Formation.

The stiffness and density of the soils generally increase with increasing depth. Based on our Atterberg limits testing, soils in the upper 20 feet of the site range from non-plastic to high plasticity; however, most of the shallower soils exhibit medium plasticity. The tested moisture content of the silt and clay at the site ranged from 27 to 70 percent at the time of our explorations. The tested moisture content of the silty sand and silty gravel at the site ranged from 27 to 61 percent at the time of our explorations. Consolidation testing indicates the soils are over-consolidated. Expansion index testing of a sample from TP-7 at 10.0 feet BGS and a swell testing on a sample from TP-7 at 5.0 feet BGS indicates the clay has low expansion potential.

4.3.4 Astoria Formation

The deeper soils at the site generally consist of decomposed to weathered claystone of the Astoria Formation. Decomposed to weathered siltstone/sandstone was also encountered from 74.0 feet BGS to the depth explored of 81.5 feet BGS in boring B-1. Other than an isolated zone of soft to medium stiff clay from 34.0 to 43.0 feet BGS in boring B-2, which could be a former failure zone, evidence of shear zones, slickensides, bedding planes, or other observations indicating slope movement were not encountered in the borings or CPT probes. The decomposed to weathered Astoria Formation is generally stiff to hard clay and silt or very dense, silty sand by soil classification methods as used on the exploration logs, but would be termed extremely soft to soft by rock classification methods. Based on prior Atterberg limits and grain-size testing by others and Atterberg limits testing by GeoDesign, the Astoria Formation generally exhibits medium to high plasticity with plasticity indices ranging from roughly 24 to 72 and clay fractions ranging from roughly 25 to 77 percent.

4.3.5 Groundwater

Slow groundwater seepage was observed in September 2017 in test pits TP-4, TP-9, and TP-12 at depths ranging between 10.0 and 11.0 feet BGS. In May/June 2017 we measured groundwater at depths ranging between 3.0 and 17.0 feet BGS in the following existing vibrating wire piezometers at the site: SI-1 at 25.0 and 55.0 feet BGS; SI-2 at 25.0 and 85.0 feet BGS; SI-3 at 25.0, 55.0, and 85.0 feet BGS; and SI-4 at 70 feet BGS. The piezometers in SI-1 at 25.0 and 55.0 feet BGS and SI-4 at 70.0 feet BGS were monitored until mid-September 2017 during which time the measured groundwater levels fell between 0.5 foot and 2.2 feet BGS. In mid-September 2017 we also measured groundwater at a depth of 10.1 feet BGS in a vibrating wire piezometer installed at 30.0 feet BGS in boring B-1. Pore pressure data from the CPT probes indicate deeper groundwater at depths of roughly 40 to 55 feet BGS, suggesting groundwater likely perches at the site and takes longer to influence deeper low-permeability soils at the site. Based on our observations and research, we anticipate water can be encountered at shallow depths below the ground surface, particularly near lower drainage areas and during the wet season. The depth to groundwater may fluctuate in response to seasonal changes, prolonged rainfall, changes in surface topography, and other factors not observed in this study.

4.4 SLOPE INCLINOMETER MONITORING

Four slope inclinometers were previously installed at the site (SI-1 through SI-3 by Geocon in 2012 and SI-4 by GRI in 2013). The inclinometer locations are shown on Figure 2. Slope inclinometers are used to monitor subsurface movements and deformation. The inclinometer casing has grooves to control the orientation of an inclinometer sensor, which is lowered down the casing to obtain subsurface measurements. Movement can be measured by comparing the results of successive sets of readings. GRI provided the prior slope inclinometer measurement data.

The accuracy of the equipment is shown on the plots as straight lines radiating out from the bottom of the readings. Data points located between the lines are considered within the error band of the equipment and should not be viewed as slope displacement.

We collected inclinometer readings in 2017 using a different probe and cable from the same equipment manufacturer. Plots of the inclinometer data are presented in Appendix F.

Comparing readings collected with different equipment added a level of potential error to the data. However, the 2017 readings appear generally consistent with the initial baseline readings taken in 2012 and 2013 and are within the error bands of the equipment. The exception is SI-2 where the surface monument had been damaged from truck traffic, resulting in compression on the casing. In addition, sediment at the bottom of the casing prevented our probe from being lowered to the same initial starting point as the 2012 baseline and 2013 readings. The damage to the casing and sediment resulted in a data error when comparing the 2017 reading to the 2012 reading.

In general, there was no significant measurable movement recorded by the inclinometer readings. There was some measured deformation in the secondary (B) axis direction of SI-1 at 40 to 50 feet deep. Considering the SI-1 movement was only in the secondary axis, had not changed significantly since 2013, and did not result in measurable movement in the overlying soils, it is not considered significant. There was some larger deformation that slightly exceeded the error band measured in the secondary (B) axis for SI-2; however, the surface monument had been damaged from truck traffic and was pushing down on the casing. The measured movements bowed at two depths with little movement at the surface, and the overall measured movement was generally in the northeastern (uphill) direction, all of which indicate the movement is from damage (compression) of the inclinometer casing.

5.0 SLOPE STABILITY ANALYSES

Slope stability analyses were performed using general limit equilibrium methods and the SLOPE/W software package. We used the Morgenstern-Price analysis method, which satisfies both moment and force equilibrium. Selected plots of our stability analyses results are presented in Appendix G.

Our initial slope stability analyses were conducted for the interpreted massive ancient or dormant landslide represented by the geologic cross section on Figure 5. Since evidence has not been observed by GeoDesign or recent studies by others to suggest there has been large-scale significant movement in the recent geologic past, the ancient dormant landslide was assumed to be relatively stable during the last CSZ event. The last CSZ event occurred over 300 years ago on January 26, 1700 and had an estimated magnitude of 8.7 to 9.2. A pseudostatic horizontal seismic load of 0.2 g was used based on an estimated PGA of approximately 0.46 g from the last subduction zone earthquake for our analyses. We estimated a dynamic effective stress friction angle of 22.5 to 28.5 degrees for the residual slide plane for groundwater levels ranging from approximately 5 to 20 feet BGS. We conservatively assumed a dynamic friction angle of 22.5 degrees for our back-calculation using the lower groundwater table elevation.

Dynamic friction angles of fine-grained residual slide planes have been shown by many studies to be 20 to 100 percent greater than for static loading with greater increases for higher rates of loading (Meehan, 2006; Vessely and Cornforth, 1998; and Yoshimine, 1999). The back-calculated dynamic friction angle was reduced by 50 percent for an estimated static residual shear strength of 15 degrees. This static residual shear strength is also roughly the average between the residual friction angle estimated using the average plasticity index and the average clay fraction (Lupini et al., 1981; Skempton, 1985). Residual friction angles are greater at lower stresses and experience strength gain over time; however, strength gain is limited at higher effective stresses (Stark and Hussain, 2010). Tested drained residual torsional (large strain) friction angles ranged from 9 to 19 degrees for clay from the site, and 15 degrees is also considered reasonable for the existing residual shear strength based on the depth of the assumed slide plane, lack of observed slide zones, and lack of evidence of significant movement in the recent geologic past.

We conducted static stability analysis for massive slope movement using variable groundwater depths. Our analysis indicates factors of safety range from approximately 1.2 for groundwater near the ground surface to 2.1 for groundwater approximately 30 feet BGS. Drainage is very important to the development of the site, and our report provides specific drainage recommendations, including a trench cutoff drain in the upper portion of the site.

Using the dynamic friction angle of 22.5 degrees for the residual slide plane, we estimate yield accelerations of 0.11 g to 0.20 g to obtain a factor of safety of 1.0 for groundwater depths ranging from approximately 5 to 20 feet BGS. Based on Newmark analyses using simplified empirical models (Rathje and Saygli, 2009; Jibson, 2007), we estimate maximum slope movements of 2 feet for a design level Magnitude 9.0 subduction zone earthquake, provided the average groundwater depth is maintained approximately 10 feet BGS or greater at the site. Estimated slope movements increase to approximately 4 feet for a groundwater depth of 5 feet BGS and decrease to approximately 1 foot for an average groundwater depth 20 feet BGS. Our dynamic analyses also indicate drainage is very important to maintaining slope stability for the development of the site.

We evaluated a localized cross section for the proposed parking lot fill area west of the proposed school building. The incised drainage channel below the proposed fill area results in a shallower depth to the interpreted ancient residual slide plane near the toe of the slope. Our analysis indicates a lower than typically recommended factor of safety of 1.3 for the existing slope, which is further reduced to 1.2 from the proposed fill under static loading conditions. Typically, a factor of safety of 1.5 is desired for static slope stability. We have recommended revising the proposed layout and/or grading of the site as further discussed in the "Conclusions and Recommendations" section. We have also recommended other revisions to the layout and grading based on geologic considerations of the site.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The site is located within a mapped massive ancient landslide area. Evidence has not been observed by GeoDesign or in prior studies by others to indicate there has been significant movement of the dormant landslide in the recent geologic past. In our opinion, the risk of reactivating the dormant landslide is low, provided the recommendations provided in this report are followed. We recommend changes to the proposed grading and site layout in this report, and GeoDesign should be contact to review and comment on revisions to the site layout and grading plans before they are finalized.

Our stability analyses indicate groundwater has a significant influence on the stability of slopes at the site and groundwater tends to perch on the medium to high plasticity, fine-grained soil at the site. We recommend installing a drainage cutoff trench with a minimum depth of 20 feet across the upper portion of the site to intercept as much water higher on the slope as possible. The trench should be excavated and backfilled in sections of less than 75 feet to reduce the potential for slope instabilities. We also recommend installing shallower French cutoff drains to minimum depths of 8 feet at the base of the proposed larger cut areas for the track and field and at the base of the parking lot east of the proposed school building. These trenches should also be excavated and backfilled in sections of less than 75 feet. All cutoff drains should be routed into the stormwater system or to the base of the larger drainages bordering the north and south edges of the site. All surface runoff should also be collected and routed into the stormwater system at the site. Further recommendations and details are provided in the "Drainage" section.

Provided drainage and grading recommendations provided in this report can be met, we recommend supporting the proposed middle/high school building on a thick, reinforced mat foundation for the following reasons. Based on the site conditions, geologic conditions, and our analyses, a design-level Magnitude 9.0 subduction zone earthquake may not result in significant differential movement within the building footprint, but could result in up to 2 feet of global movement of the ancient massive landslide. If a large subduction zone earthquake triggers global movement of the ancient massive landslide, deep foundations would be exposed to differential movement; additional shear stresses on the piling would result; and given the estimated potential movement, damage or fail the piling. We recommend supporting the proposed middle/high school building on a thick, reinforced mat foundation that would move together with any potential global mass movement of the hillside and limit the potential for differential movements. We recommend reinforcing the mat foundation on both the top and bottom to limit potential swell-induced deformations (see below) and potential earthquake-induced differential movement.

Shrink-swell damage has been documented as an ongoing issue for the existing elementary school building west of the site; however, soil near the elementary school building are generally higher plasticity clay than the shallow soil in the upper Seaside School Campus and proposed middle/high school area of the site. Based on the results of our swell test, expansion index test, and Atterberg limits testing, undisturbed soil at the foundation elevation of the proposed middle/high school building has low swell potential. Therefore, a thick, reinforced mat foundation with an underdrainage system and back-of-wall drains for embedded building walls is expected to sufficiently limit potential swell pressure-induced foundation movement. Further mat foundation recommendations are provided in the "Foundation Support Recommendations" section.

Our stability analysis for the proposed parking lot fill slope west of the building area indicates an insufficient factor of safety because the proposed fill extends the slope above an incised drainage channel. Therefore, the toe of the slope is expected to have shallower groundwater and extend closer to the interpreted residual slide plane for the ancient landslide. Accordingly, we recommend altering the proposed layout and/or grading for the parking area. Several potential options to address this area include the following:

- Fill the portion of the drainage below the proposed parking fill, essentially buttressing the toe of the fill slope.
- Lower the grade of the proposed parking area by installing a cut wall between the proposed school building and parking lot.
- Eliminate the proposed parking area west of the school building and increase the size of the proposed parking lot east of the school building.

Currently, a cut of up to approximately 25 feet is proposed for a stormwater detention pond below a proposed smaller cut for the roadway in the southwestern area of the site. The lower portion of the site is near the toe of the interpreted ancient landslide where we recommended limiting cuts. We recommend altering the proposed pond grading, moving the pond, and/or installing other stormwater detention facilities elsewhere at the site to prevent significant cuts in the lower portion of the site. We also recommend maximum cut slope grades of 3H:1V in the lower half of the site.

Proposed fills for the north end of the track and field area extend over two landslide scarps, which we mapped along the incised drainage at the north edge of the site as shown on Figure 2. We recommend offsetting any new fill a minimum of 25 feet from any structural features and a minimum of 75 feet from identified landslide scarps. The orientation and/or grade of the track and field could be adjusted to avoid filling near the landslide scarps. A cut wall could also be constructed instead of a cut slope at the uphill (east end) of the track and field to shift the proposed layout further away from the scarps and reduce fills required at the northwest end. Any significant size cut walls should have tiebacks, which are prestressed to substantially limit relaxation/movement of the soil and slopes above the walls.

Fills are proposed over various size drainage ravines or swales in multiple areas of the site. Drainage blankets should be placed at the base of any fills over drainage ravines or swales or any other area were seepage is observed at the ground surface. A minimum 6-inch-diameter drainage pipe should be placed at the bottom of the drainage blankets, which consist of a minimum 1-foot-thick and 20-foot-wide drain rock zone wrapped in a geotextile fabric. Additional details can be provided on a case-by-case basis when final grading plans are prepared.

The toe of proposed fill slopes will need to be keyed into firm, undisturbed native soil. We recommend assuming a minimum 10-foot-wide zone will need to be keyed a minimum of 5 feet deep at the base of structural fill slopes. Fills will also need to be benched into firm native soil as recommended in this report. Deeper keys may be required, particularly for fills over the isolated potentially unstable areas identified on Figure 3. The actual depth and width required for keys at the base of the fill slopes can be evaluated based on the final grading plan and site conditions observed at the time of construction.

Based on the results of our explorations and shear wave velocity testing, the soil profile in the planned new building area corresponds to IBC Site Class D. We have conducted seismic ground motion modeling to prepare a site-specific seismic hazard evaluation report for the site, which is presented in Appendix F. Based on our ground motion modeling, seismic parameters slightly exceeding those calculated for an IBC Site Class D are recommended for the site as detailed in our site-specific seismic hazard evaluation report.

The on-site soil can be used for structural fill. However, the on-site soil should only be used for structural fill for the building or other above-grade structures if testing indicates the soil to be used for fill exhibits low plasticity or if the soil is lime amended to limit the shrink-swell potential. Given the fine-grained nature of the soil at the site, the use of the on-site soil for structural fill 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. We anticipate that the moisture content of the soil currently will be above optimum and drying will be required for use as structural fill. Drying the soil will require an extended period of dry weather, typically experienced from early July to mid-October. Alternately, on-site soil can be amended for placement as structural fill without drying to the optimum moisture content for compaction.

The existing pavement sections at the site are not designed for the support of construction traffic. Construction traffic could cause significant distress to the pavements as well as disturbance to the underlying subgrade and should be planned carefully by the contractor. Moreover, the aggregate base thickness for new pavements is intended to support the design traffic and is not intended to support construction traffic. The on-site fine-grained subgrades should be protected from disturbance using granular haul roads and staging areas and/or amending the subgrades as described in this report.

Our recommendations for design and construction of the project are provided in the following sections.

7.0 DESIGN

7.1 PERMANENT SLOPES

Permanent cut slopes in the lower half of the site should not exceed a gradient of 3H:1V, and cut and fill slopes for the remainder of 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. 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 General

Groundwater has been measured at depths ranging from 3 to 17 feet BGS at the site and drainage is very important to maintaining stability of the ancient landslide and slopes at the site. The drain rock and drainage geotextile for subsurface drains should meet the requirements specified in the “Materials” section. All subsurface drains should discharge to the base of the larger drainages along the north and/or south edge of the site or be routed into the stormwater system for the site. Drainage should be provided behind walls as recommended in the “Retaining Structures” section. Other specific drainage recommendations are provided in the following sections.

7.2.2 Drainage Cutoff Trench

We recommend installing a drainage cutoff trench with a minimum depth of 20 feet across the upper portion of the site to intercept as much water higher on the slope as possible. The trench should be constructed as early as possible during construction to reduce groundwater downslope in the development area. We recommend installing several piezometers downslope of the cutoff trench and monitoring the existing piezometer installed in boring B-1 to evaluate the effectiveness of the cutoff trench. Additional drainage trenches may be necessary if the trench is not sufficiently lowering groundwater elevations below the trench. The trench should be excavated and backfilled in sections of less than 75 feet to reduce the potential for slope instabilities. The drainage trench excavation should be shored and/or flattened as necessary in accordance with the “Excavation” section. The cutoff drain should consist of minimum 12-inch-diameter, perforated drainpipe sloped to drain at the base of the trench backfilled with drain rock that is wrapped in a geotextile filter. The drain rock should extend up to 1 foot BGS.

Manholes should be installed along the alignment of the drainage trench as required to allow for inspection and maintenance of the pipe. The drainage pipes should have a regular inspection schedule to evaluate the system does not lose capacity from the accumulation of sediment or other material.

7.2.3 Shallower French Cutoff Drains

We recommend French drains be installed to intercept groundwater at the toe of any new cut slopes. We recommend extending the French drains to a minimum depth of 8 feet BGS at the base of the proposed larger cut areas for the track and field and at the base of the parking lot east of the proposed school building. The actual alignment and depth of the French drain should be based on the final grading plan. The French drains should be constructed at a minimum slope of approximately ½ percent and pumped or drained by gravity to a suitable discharge. The perforated drainpipe should not be tied to a stormwater drainage system without backflow provisions. The French drain should consist of 6-inch-diameter, perforated drainpipe embedded in drain rock that is wrapped in a geotextile filter.

7.2.4 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.

7.2.5 Surface Drainage

We recommend that all roof drains be connected 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 that ground surfaces adjacent to the building be sloped away from the building to facilitate drainage away from the building.

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 pumped or drained by gravity to a suitable discharge. The perforated drainpipe should not be tied to a stormwater drainage system without backflow provisions. The foundation drains should consist of 4-inch-diameter, perforated drainpipe embedded in a minimum 2-foot-wide zone of crushed drain rock that extends 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.

7.2.7 Subfloor Drainage

The subfloor drainage pipe should be connected to the perimeter foundation drainage pipe on minimum 30-foot centers in one direction across the building footprint. The crown of the perforated pipes should be at least 4 inches below the base of the mat foundation or floor slab and above the elevation of the perimeter drains. We recommend that the perimeter foundation and subfloor drainage pipes consist of minimum 4-inch-diameter, perforated PVC pipe that is surrounded on all sides by a 12-inch thickness of drain rock. The drain rock should be separated from the trench walls by a drainage geotextile. The perforated pipe should be installed on a minimum ½ percent slope draining to the storm sewer.

7.3 SEISMIC DESIGN CRITERIA

7.3.1 Seismic Design Parameters

Based on the results of our explorations and shear wave velocity testing, the soil profile in the planned new building area corresponds to IBC Site Class D. We have conducted seismic ground motion modeling to prepare a code-required site-specific seismic hazard evaluation report for the site, which is presented in Appendix F. Based on our ground motion modeling, seismic parameters slightly exceeding those calculated for an IBC Site Class D are recommended for the site as detailed in our site-specific seismic hazard evaluation report.

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 FOUNDATION SUPPORT RECOMMENDATIONS

7.4.1 General

Based on the site conditions, geologic conditions, and our analyses, a design-level Magnitude 9.0 subduction zone earthquake may not result in any significant movement, but could result in up to 2 feet of movement of the ancient massive landslide. We recommend supporting the proposed middle/high school building on a thick, reinforced mat foundation that would move together with any potential mass movement of the hillside and limit the potential for differential movement of the building. The mat foundation should be reinforced on both the top and bottom to limit potential swell-induced deformations and potential earthquake-induced differential movement.

Shrink-swell damage has been documented as an ongoing issue for the existing elementary school building at the site; however, soil near the elementary school building are generally higher plasticity clay than the shallow soil in the upper Seaside School Campus and proposed middle/high school area of the site. Based on the results of our swell test, expansion index test, and Atterberg limits testing, undisturbed soil at the foundation elevation of the proposed middle/high school building has low swell potential. Therefore, a thick, reinforced mat foundation with an underdrainage system as recommended in the "Drainage" section and back-of-wall drains for embedded building walls is expected to sufficiently limit potential swell pressure-induced foundation movement. Due to the variable plasticity and swell potential of the soil at the site, only engineer-approved, low-plasticity soil should be used for on-site fill below foundations. Alternately, the on-site fill soil can be lime-amended to reduce swell potential for placement below the building. In addition, native subgrades should be covered or backfilled to avoid excessive drying from exposure, which can increase the post-construction shrink-swell potential.

7.4.2 Mat Foundation

We recommend supporting the proposed new middle/high school building on a mat foundation bearing on firm, undisturbed native soil or structural fill consisting of imported granular material, low-plasticity on-site soil, or lime-amended on-site soil. The mat foundation will distribute the applied bearing pressures and limit differential movement from the shrink and swell potential of the subgrade soil. We recommend placing and compacting a minimum of 6 inches of imported granular fill over the mat foundation subgrade. Any zones of soft or loose soil, undocumented fill, or soil containing deleterious material should be removed and replaced with imported granular fill.

Design of the mat reinforcement should consider that it may be subjected to swell pressures at the base of the mat. We recommend a preliminary modulus of subgrade reaction of 100 pci for the mat foundation bearing on firm, undisturbed native soil or imported granular material underlain by firm, undisturbed native soil. The subgrade modulus value was estimated for the anticipated loads using correlations with existing blow count data, laboratory testing results, and subsurface information. The preliminary modulus of subgrade reaction can be doubled for the analyses of dynamic loads. We recommend the structural engineer provide GeoDesign with the bearing pressure distributions for the indicated modulus of subgrade reactions to evaluate if the values are suitable for final design or should be adjusted.

We anticipate the sustained contact pressure from dead and long-term live loads will not exceed a maximum value of 3,000 psf and an average contact pressure of 1,500 psf. We estimate total post-construction settlement associated with the indicated maximum and average contact pressures will be less than 1 inch. We anticipate the stiffness of the mat will limit differential settlement across the mat to less than ½ inch.

The installation of a vapor barrier may be warranted in order to reduce the potential for moisture transmission through and efflorescence growth on the floor slabs. In addition, flooring manufacturers often require vapor barriers to protect flooring and flooring adhesives and will warrant their product only if a vapor barrier is installed according to their recommendations. If the project includes highly moisture-sensitive flooring, we recommend that 10- or 15-mil Stego Wrap be considered for this project. The recommended procedures for installing Stego Wrap are to pour the floor slab concrete directly over the vapor barrier. We recommend that the structural engineer be contacted to determine if the mix design for the concrete should be modified assuming the above-referenced construction sequence. Actual selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team.

7.4.3 Spread Footings

Spread footings can be used to support the walls or other small ancillary structures. Spread footings should be supported by on firm, undisturbed native soil or structural fill consisting of imported granular material, low-plasticity on-site soil, or lime-amended on-site soil. We recommend placing and compacting a minimum of 6 inches of imported granular fill over the foundation subgrades. Any zones of soft or loose soil, undocumented fill, or soil containing deleterious material should be removed and replaced with imported granular fill.

Shallow footings should be proportioned on an allowable bearing pressure of 3,000 psf. The value above 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 can be doubled for short-term loads resulting from wind or seismic forces.

Continuous wall and isolated spread footings should be at least 16 and 20 inches wide, respectively. The bottom of exterior footings should be at least 18 inches below the lowest adjacent exterior grade. The bottom of interior footings should be established at least 12 inches below the base of the slab.

Total consolidation-induced settlement should be less than 1 inch, with differential settlement of up to ½ inch between lightly loaded and heavily loaded footings.

7.4.4 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 300 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 pressure of 250 pcf equivalent fluid pressure. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. For footings in contact with imported granular material, a coefficient of friction equal to 0.40 may be used when calculating resistance to sliding.

7.4.5 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, organics, unsuitable fill, prior topsoil zones, softened subgrades (if present) have been removed and native soil subgrades have not dried excessively. Localized deepening of footing excavations may be required to penetrate debris, fill, softened, dried, or 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 grades and preliminary site plan, we anticipate walls will be less than 15 feet in height. Cut walls larger than approximately 10 feet high should be evaluated on a case-by-case basis and will likely require tiebacks, which are prestressed to limit relaxation/movement of the soil and slopes above the walls.

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 15 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 pressure of 35 pcf equivalent fluid pressure for design. Cut retaining walls and retaining walls that will be restrained from rotation prior to being backfilled (such as basement stem walls) should be designed using a 55 pcf equivalent fluid pressure. For embedded building walls, a superimposed seismic lateral force should be calculated based on a dynamic force of $8.5H^2$ 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 1) when designing walls that retain sloping soil.

Table 1. 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

If other building foundations or other surcharges are located within a horizontal distance from the back of a wall equal to the height of the wall, additional pressures may need to be accounted for in the wall design. For alternate surcharge loadings, GeoDesign should be contacted to provide appropriate wall surcharges based on the actual magnitude and configuration of the applied loads.

7.5.5 Soldier Pile Walls

Cantilever soldier piles can likely be used for shorter cut walls of less than 15 feet. Larger cut walls should be evaluated on a case-by-case basis and will likely require tiebacks, which are prestressed to limit relaxation/movement of the soil and slopes above the walls. Cobbles and boulders, if encountered, will result in difficult installation of the soldier piles. Structural design of the soldier piles should consider the lateral earth pressures discussed above. The active pressure should be considered to act on 1 times the pile width below the excavated finish grade. A passive resistance of 300 pcf modeled as an equivalent fluid pressure acting over 2.5 times the pile width, including the grouted diameter of the piles, can be used to calculate the pile resistance. The passive resistance for the upper 2 feet of soil below the excavated finish grade should be neglected. Based on our experience, settlement on the order of 1 inch can be expected adjacent to the walls. We recommend a minimum soldier pile embedment of 5 feet. If soldier piles are drilled and groundwater is encountered, grout should be placed using tremie pipe methods.

7.5.6 Lagging

Lagging should consist of cross members between vertical supports capable of resisting horizontal earth pressures equal to one-half of the earth pressures used to design the shoring system. This one-half reduction is a rough approximation of the preferential redistribution of earth pressures on the stiff, tied-back soldier piles compared to the relatively flexible lagging between the piles. Soldier pile spacing greater than 4B, where B is the pile diameter, will greatly decrease the effect of arching between piles, resulting in higher lagging earth pressures. A more accurate earth pressure distribution for the lagging can be provided if the pile spacing is greater than 4B.

Soldier pile and lagging walls will likely consist of pressure-treated lumber or shotcrete. We recommend prompt and careful installation of lagging to maintain the integrity of the excavation, particularly in areas of perched seepage.

7.5.7 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 should be cut back at an inclination 1½H:1V or flatter or be shored, and they should be completed and backfilled in sections not exceeding 75 feet in length. The top of temporary slopes should be located at least 5 feet from pavements, 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.

7.5.8 Wall Foundations

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

7.5.9 Wall Backfill and Drains

The above design parameters have been provided assuming that 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.

7.5.10 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

Traffic at the proposed school campus will predominately consist of passenger cars and buses. At the time this report was prepared we had not been provided with anticipated traffic counts.

Pavements should be installed on firm, undisturbed native subgrade or new structural fill as described in the “Site Preparation” and “Materials” sections. If high-plasticity on-site soil will be used as structural fill for the upper 1 foot of pavement subgrades, it should be lime amended to reduce the shrink-swell potential.

Our pavement recommendations are based on the following assumptions:

- 20-year design life for AC and PCC.
- A resilient modulus of 20,000 psi was estimated for the aggregate base.
- Initial and terminal serviceability indices of 4.2 and 2.0, respectively, for AC and 4.5 and 2.5, respectively, for PCC pavement.
- Reliability and standard deviations of 85 percent and 0.45, respectively, for AC pavement and 85 percent and 0.40, respectively, for PCC pavement.
- Structural coefficient of 0.42 and 0.10 for the 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 native or fill subgrade prepared in accordance with the “Site Preparation” and “Materials” sections.

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 following AC pavement sections in Table 2.

Table 2. Recommended Standard Pavement Sections

Pavement Use	Busses per Day	Trucks per Day[1]	ESALs	AC Thickness (inches)	Aggregate Base Thickness (inches)
Automobile-Only Drive Aisles	0	0	50,000	3.0	10.0
Automobile Parking	0	0	10,000	2.5	9.0
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
	40	10	275,000	5.0	12.0

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

The AC should be Level 2, ½-inch, dense ACP according to OSSC 00745 (Asphalt Concrete Pavement) and compacted to 91 percent of the maximum specific gravity of the mix, as determined by AASHTO T 209. Asphalt binder should be performance graded and conform to PG 64-22 or better. The lift thicknesses should be 2.0 to 3.5 inches for ½-inch ACP. The AC should be compacted to 91 percent of the maximum specific gravity of the mix, as determined by ASTM D 2041. The aggregate base should meet the specifications for aggregate base provided in the "Materials" section. Recommended aggregate base thicknesses are not intended to support construction traffic. The fine-grained subgrades should be protected from disturbance as described in the "Subgrade Protection" 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, pavements, 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.

8.2.2 Stripping

The existing topsoil zone should be stripped and removed from all fill areas. Based on our explorations, the average depth of stripping will be approximately 3 inches, although greater stripping depths may be required to remove localized zones of loose or organic soil. Greater stripping depths 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 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-tired 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 which have dried excessively should be improved in accordance with subsequent sections of this report.

8.2.4 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 Excavation and Shoring

The soil conditions at the site consist primarily of stiff to hard clay. Most cuts should be readily completed with conventional excavation equipment; however, ripping equipment may be required for cuts in more competent claystone. 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 the 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.

8.4.2 Trench Dewatering

Shallow excavations 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 "Materials" section.

We note that these recommendations are for guidance only. The 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.

8.4.3 Safety

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

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 matter or other unsuitable material 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 is provided in this section.

In locations where fill is to be placed on slopes steeper than 5H:1V, level benches should be cut into the existing sloping surfaces to remove the surface loose material and should extend into the structural fill of the existing embankment. The benches should be a minimum of 10 feet wide or 1½ times the width of the compaction equipment, whichever is wider.

8.5.1.2 On-Site Soil

The material at the site should be suitable for use as general structural fill in some areas, provided it is properly moisture conditioned; free of debris, organic material, and particles over 6 inches in diameter; and meets the specifications provided in OSSC 00330.12 (Borrow Material). The on-site soil exhibits high plasticity and should be lime amended to reduce the swell potential for placement as structural fill below above-grade structures or a minimum of the upper 2 feet of finished subgrade for pavements or slabs.

Based on laboratory test results, the moisture content of the on-site soil at the time of our explorations is above the optimum for compaction. Moisture conditioning (drying) will be required to use on-site soil for structural fill. Accordingly, extended dry weather (typically experienced between early July and mid-October) 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 D 1557.

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, fairly well graded between coarse and fine material, have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and 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 D 1557. 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 as trench stabilization material, 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, less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and at least two mechanically fractured faces. The material should be free of organic matter and other deleterious material. 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 D 1557, 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 D 1557, 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 D 1557.

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 organics 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 D 1557, 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 matter, and other unsuitable material; have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis); and 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 pavements 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 base aggregate should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

8.5.2 Geotextile Fabric

8.5.2.1 Subgrade Geotextile

Subgrade geotextile should conform to OSSC Table 02320-1 and OSSC 00350 (Geosynthetic Installation). The geotextile should have a Level “B” certification. A minimum initial aggregate base lift of 6 inches is required over geotextiles.

8.5.2.2 Drainage Geotextile

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

8.5.3 Soil Amendment

8.5.3.1 General

In conjunction with an experienced contractor, the on-site soil can be amended to obtain suitable support properties without shrink-swell potential. Based on the predominantly high-plasticity soil at the site, amendment with quicklime or hydrated lime will be most suitable for most of the soil at the site. After treatment with lime, the soil can also be amended with cement if additional strength is desired for the support of construction equipment. Successful use of soil amendment depends on the use of correct mixing techniques, soil moisture content, and amendment quantities. Soil amending should be conducted in accordance with the specifications provided in OSSC 00344 (Treated Subgrade). The amount of lime or cement used during treatment should be based on an assumed soil dry unit weight of 100 pcf.

8.5.3.2 Subgrade 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 amended soils of 100 psi. The amount of lime and/or cement necessary will vary with moisture content, soil type, and desired strength. It is difficult to predict field performance of soil to lime and cement amendment due to variability in soil response, and we recommend laboratory testing to confirm expectations. Typically, 3 to 6 percent dried quicklime by weight or 4 to 8 percent cement by weight is required to stabilize soil. For preliminary design purposes, we recommend assuming 5 percent dry lime or 7 percent cement by weight will be necessary to amend the on-site soil for placement as structural fill at the current moisture contents. The recommended amount of lime can be estimated on samples tested in accordance with ASTM D 6276 prior to treatment. The amount of lime and 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.

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

The crushed rock placed over treated subgrades 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, as well as the contractor's means and methods and, accordingly, should be the contractor's responsibility.

8.5.3.3 Other Considerations

On-site soil that because of elevated moisture contents would not otherwise be suitable for structural fill may be amended and placed as fill over a subgrade prepared in conformance with the "Site Preparation" section. Typically, a minimum curing of four days is required between treatment and construction traffic access. Consecutive lifts of fill may be treated 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, then the four-day wait period is in effect.

Portland cement- and lime-amended soils are hard and have low permeability. These soils do not drain well, nor are they suitable for planting. Future planted areas should not be amended, if practical, or accommodations should be made for drainage and planting. Moreover, 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. Amendment should not be used if runoff during construction cannot be directed away from adjacent wetlands.

In addition, we recommend that the following comments be included in the specifications for the project:

- 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 a No. 4 sieve, exclusive of gravel or stone retained on these sieves. 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/treatment depth shall extend the full depth of the disturbance.
  - Multiple "passes" of the tiller will likely be required to adequately blend the lime 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/treatment 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.
  - Use a vibratory, smooth-drum roller with a minimum applied lineal force of 600 pounds per inch for final compaction, or an alternate approved by the geotechnical engineer.

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, performing laboratory compaction and field moisture-density tests, observing final proof rolling of the pavement subgrade and base rock, and AC placement and compaction.

10.0 LIMITATIONS

We have prepared this report for use by Seaside 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 for the buildings and walls, 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 our 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 the 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.

Charles M. Clough, C.E.G.
Project Engineering Geologist

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

REGISTERED PROFESSIONAL
ENGINEER
63114PE
OREGON
March 14, 2006
SHAWN DIMKE
EXPIRES: 12/31/17

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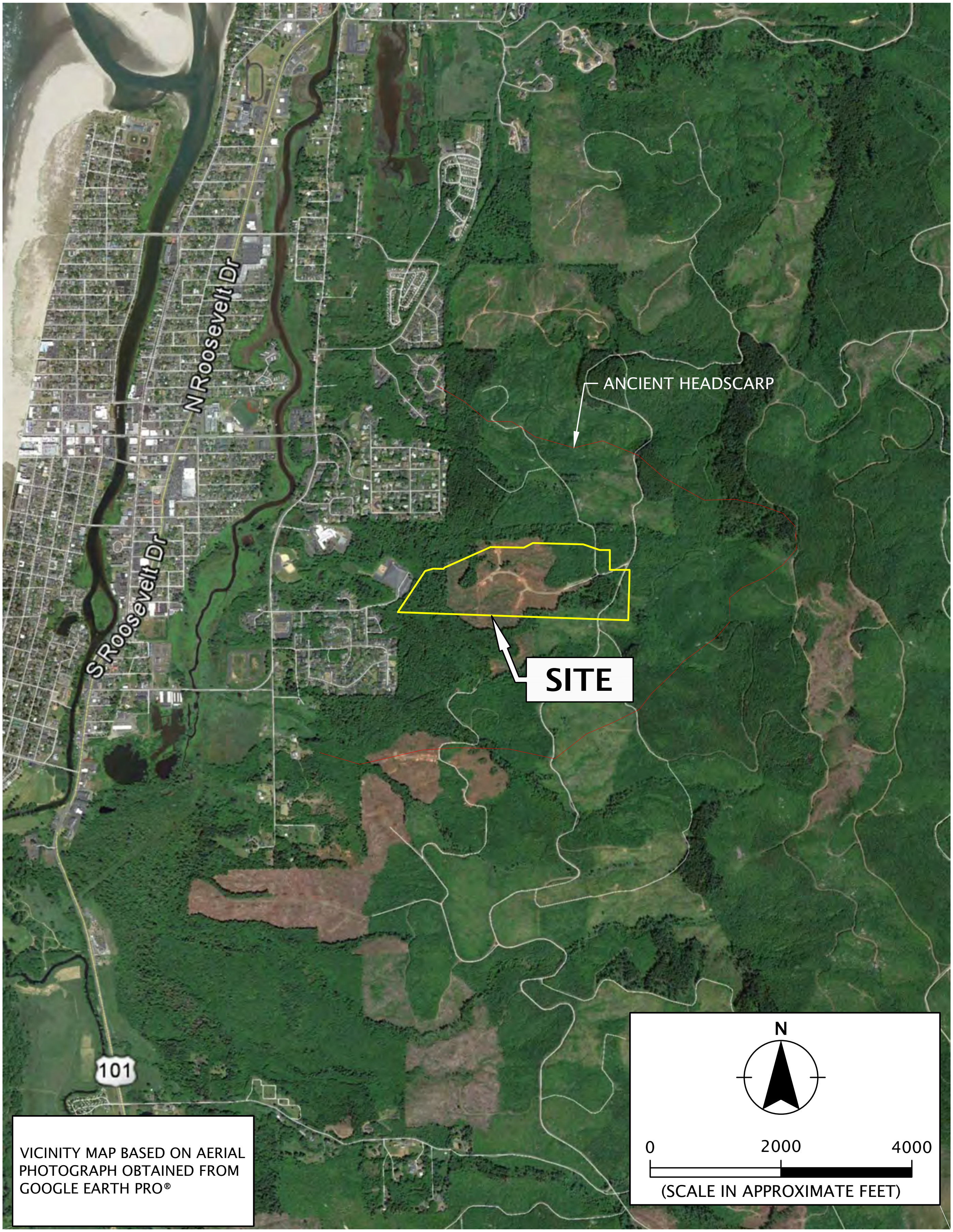
FIGURES



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

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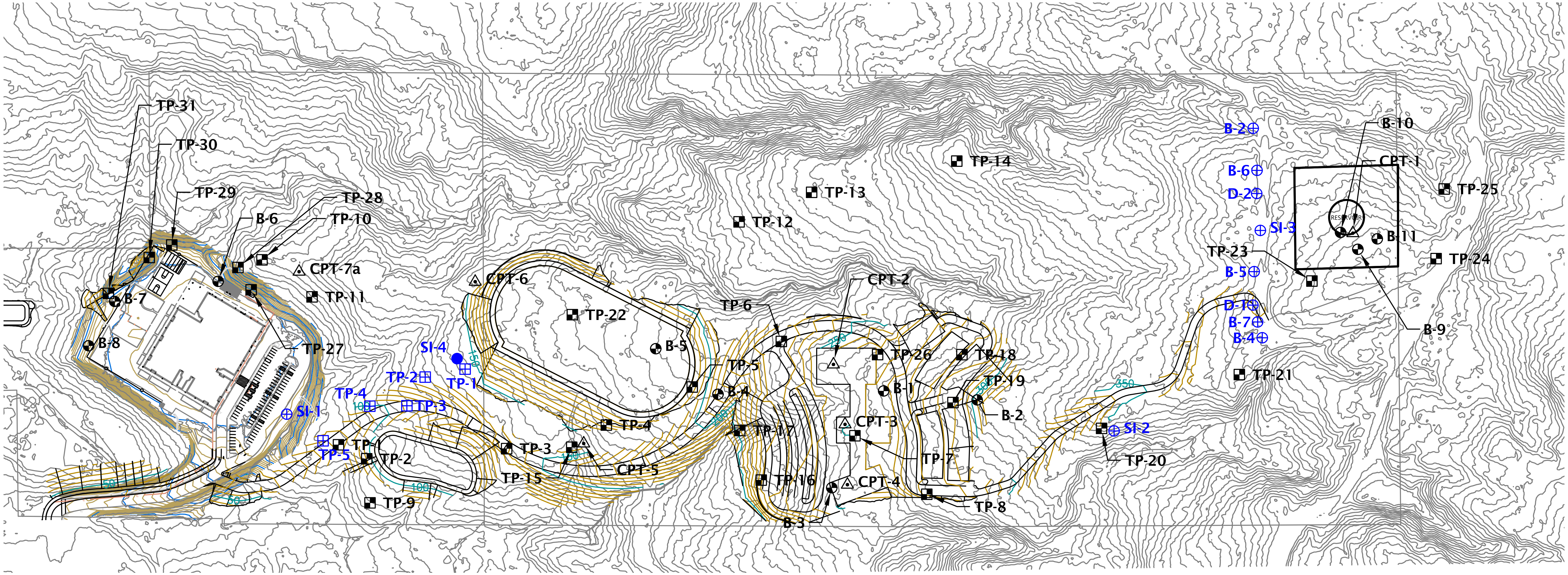
NOVEMBER 2017

VICINITY MAP

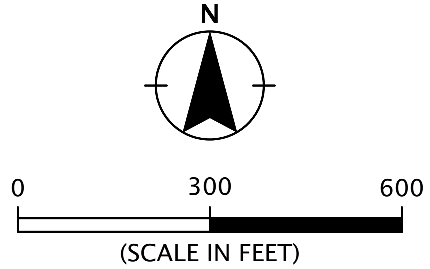
SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE 1



LEGEND:

B-1⊕	BORING (GEOCON, NOVEMBER 2012)
SI-1⊕	BORING (GEOCON, NOVEMBER 2012)
SI-4●	BORING (GRI, MAY 2013)
TP-1⊞	TEST PIT (GRI, MAY 2013)
B-1◐	BORING (GEODESIGN, SEPTEMBER 2017)
TP-1▚	TEST PIT (GEODESIGN, SEPTEMBER 2017)
CPT-1△	CPT PROBE (GEODESIGN, SEPTEMBER 2017)



SITE PLAN BASED ON DRAWING PROVIDED BY
IBI GROUP ARCHITECTS INC. ON SEPTEMBER 29, 2017

SITE PLAN

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE 2

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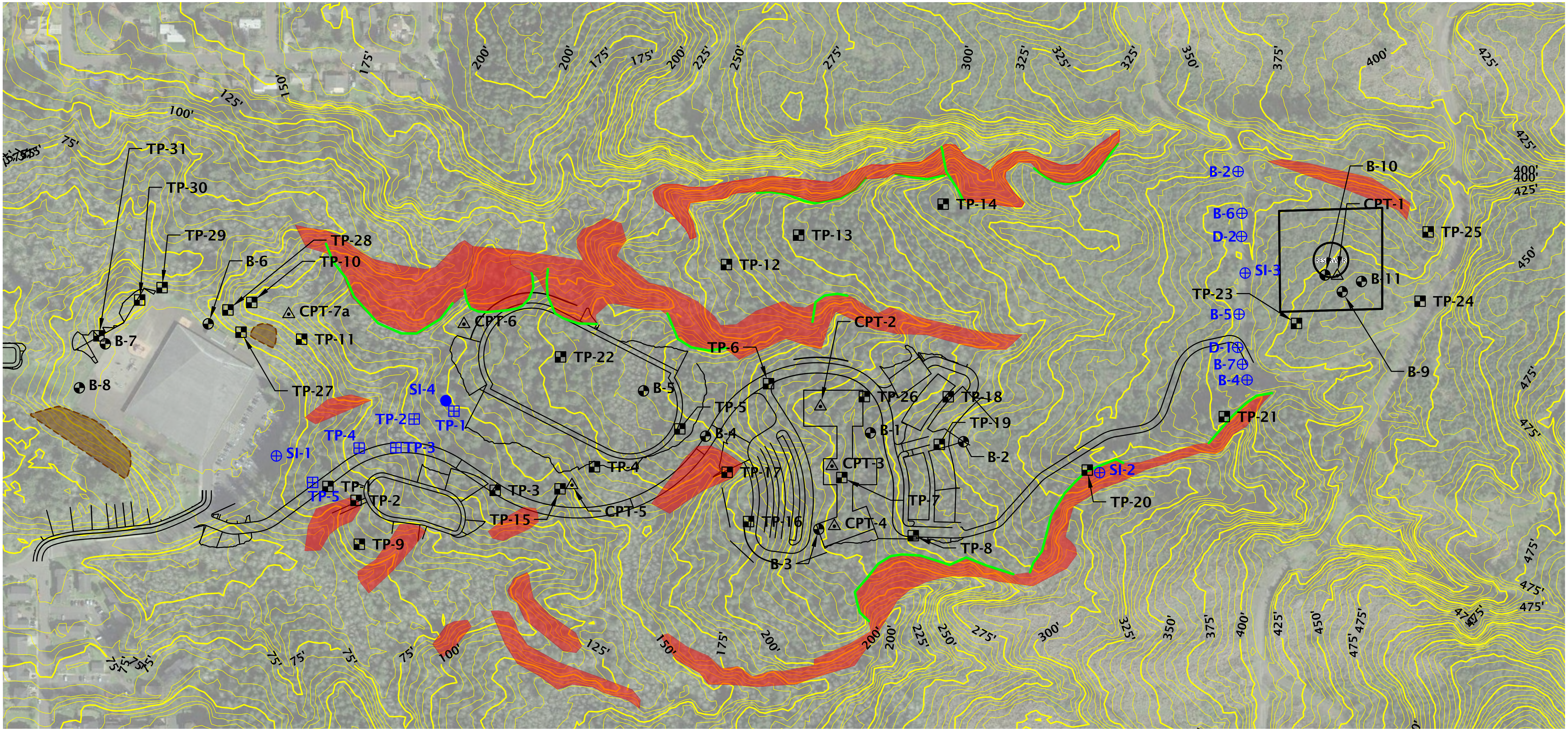

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B-1◒	BORING (GEODESIGN, SEPTEMBER 2017)
TP-1▚	TEST PIT (GEODESIGN, SEPTEMBER 2017)
CPT-1△	CPT PROBE (GEODESIGN, SEPTEMBER 2017)
—	HEAD SCARP
🟥	POTENTIALLY UNSTABLE SLOPE
▨	APPROXIMATE LANDSLIDE OR SLOPE MOVEMENT AREAS NEAR ELEMENTARY SCHOOL PER PRIOR REPORTS

N

0 300 600

(SCALE IN FEET)

HEADSCARPS AND POTENTIALLY UNSTABLE SLOPES BASED ON REVIEW OF SLOPES USING LIDAR DATA AND 2012 GEOLOGIC RECONNAISSANCE

LANDSLIDES AND POTENTIALLY UNSTABLE SLOPES

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE 3

SEASIDESD-1-03-01

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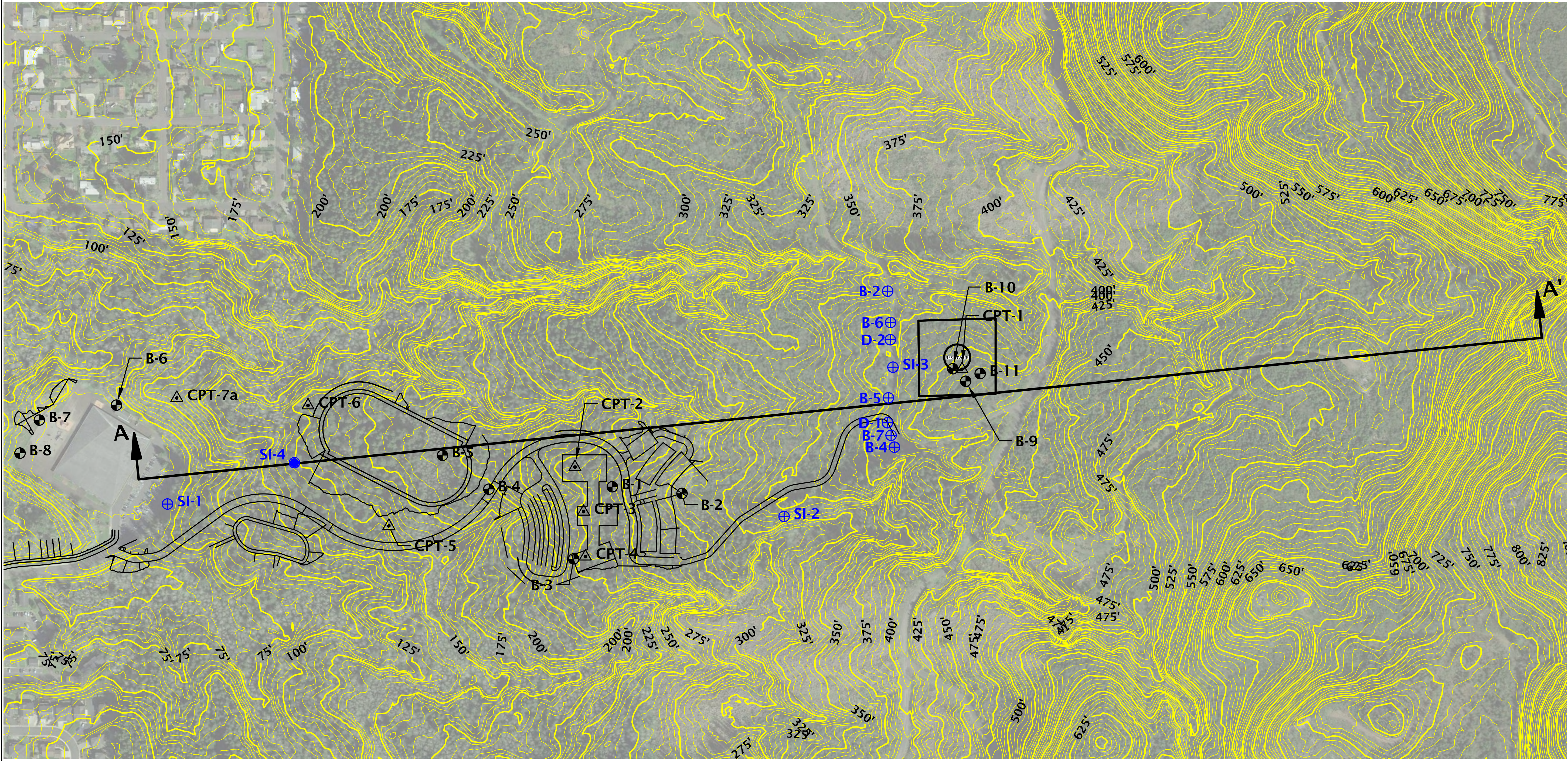
Printed By: mmiller | Print Date: 11/20/2017 8:42:37 AM
File Name: J:\S-Z\SeasideSD\SeasideSD-1\SeasideSD-1-03\SeasideSD-1-03-01\Figures\CAD\SeasideSD-1-03-01-SP-CS-LIDAR.dwg | Layout: FIGURE 3



LEGEND:

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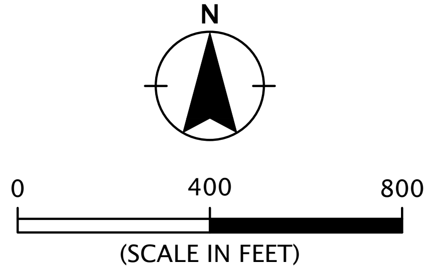
SI-1⊕	BORING (GEOCON, NOVEMBER 2012)
SI-4⦿	BORING (GRI, MAY 2013)
TP-1⊞	TEST PIT (GRI, MAY 2013)
B-1◐	BORING (GEODESIGN, SEPTEMBER 2017)
CPT-1△	CPT PROBE (GEODESIGN, SEPTEMBER 2017)



SLOPE CONTOURS DERIVED FROM BARE
EARTH LIDAR FROM DOGAMI (2009)

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GEOLOGIC CROSS SECTION LOCATION

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE 4

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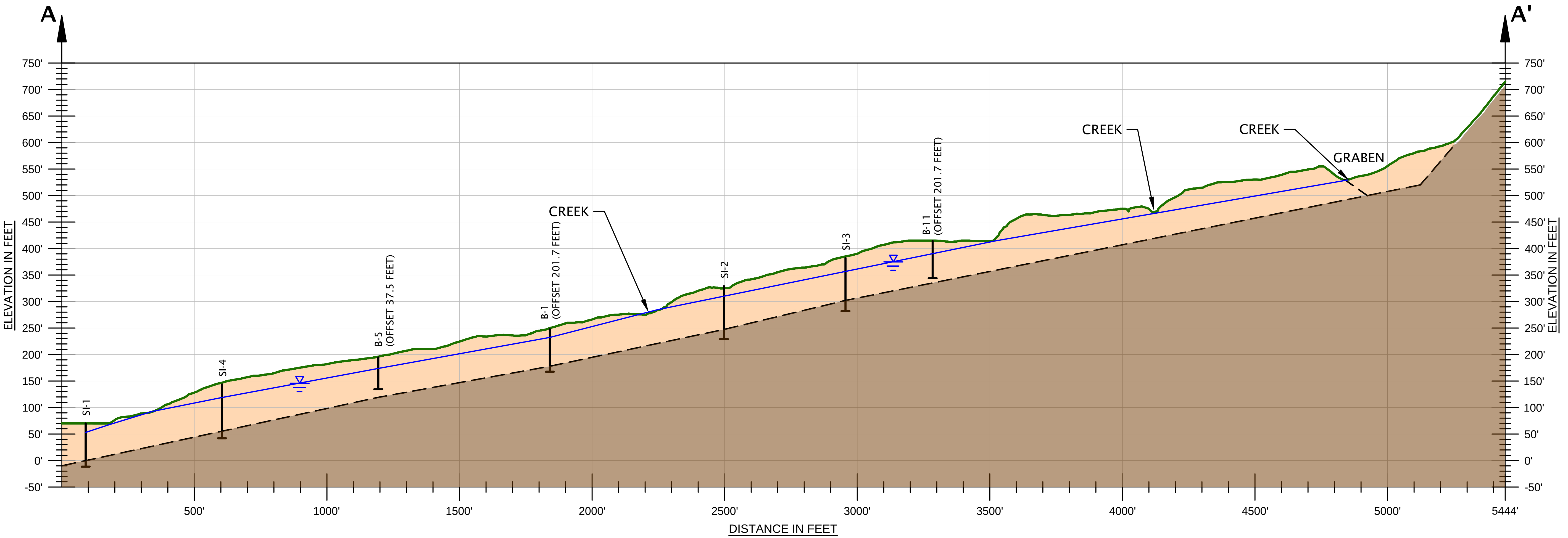

LEGEND:

EXISTING TOPOGRAPHY

BORING

CPT

INTERPRETED SLIDE PLANE

GROUNDWATER

▨ ASTORIA FORMATION

▨ ANCIENT LANDSLIDE DEBRIS

0 200 400

(VERTICAL SCALE IN FEET)

0 400 800

(HORIZONTAL SCALE IN FEET)

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FIGURE 5

CROSS SECTION A-A'

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

SEASIDESD-1-03-01

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APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

Our subsurface exploration program included drilling five borings (B-1 through B-5) to depths ranging between 36.3 and 81.5 feet BGS and excavating 23 test pits (TP-1 through TP-22 and TP-26) to depths ranging between 5.0 and 12.5 feet BGS at the approximate locations shown on Figure 2. The borings were drilled using a track-mounted drill rig and mud rotary drilling methods by Western States Soil Conservation, Inc. of Hubbard, Oregon, on September 11 through 15, 2017. The test pits were excavated using a Komatsu PC60 tracked excavator by Dan J. Fischer Excavating, Inc. of Forest Grove, Oregon, on September 6 through 13, 2017. The exploration logs are presented in this appendix. The explorations were observed by members of our geotechnical and geology staff.

Approximate locations of the explorations are shown on Figure 2. 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:

- SPTs were performed in general conformance with ASTM D 1586. The sampler was driven with a 140-pound hammer free-falling 30 inches. The number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soil is shown adjacent to the sample symbols on the exploration logs. Disturbed sand samples were collected from the split barrel for subsequent classification and index testing.
- Relatively undisturbed samples were obtained at selected intervals by pushing a Shelby tube sampler 24 inches ahead of the boring front. Shelby tube samples are preferred for consolidation and strength testing due to the lower level of disturbance.

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 88.3 percent. The calibration testing results are presented at the end of this appendix.

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 were 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 D 2216. 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

The Atterberg limits (plastic and liquid limits) testing was performed on select soil samples in general accordance with ASTM D 4318. 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 AND SWELL TESTING

We performed a swell test and one-dimensional consolidation test on a relatively undisturbed soil sample in general accordance with ASTM D 4546 and ASTM D 2435. The tests measure the volume change of a soil sample when inundated after a seating load is applied and under predetermined load increases. No appreciable swell was measured from inundating the sample with the seating load applied. The consolidation test results are presented in this appendix.

GRAIN-SIZE TESTING

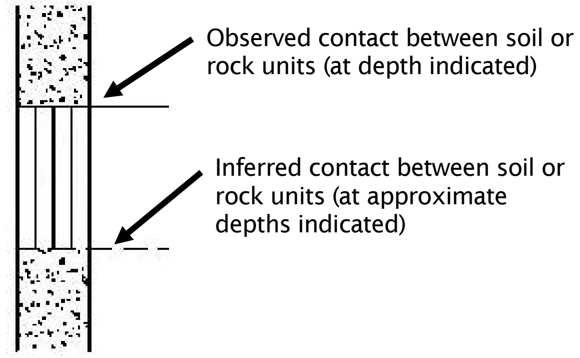
Grain-size testing was performed on a select soil sample to determine the distribution of soil particle sizes. The testing consisted of a sieve analyses and percent fines determination (percent passing the U.S. Standard No. 200 sieve) completed in general accordance with ASTM C 136 and ASTM C 117. The test results are presented in this appendix.

DIRECT SHEAR TESTING

Direct shear testing was performed on select soil samples in general accordance with ASTM D 3080. 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 obtained in general accordance with ASTM D 1586 Standard Penetration Test with recovery
▮	Location of sample obtained using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D 1587 with recovery
▤	Location of sample obtained using Dames & Moore sampler and 300-pound hammer or pushed with recovery
▥	Location of sample obtained using Dames & Moore and 140-pound hammer or pushed with recovery
⊠	Location of sample obtained using 3-inch-O.D. California split-spoon sampler and 140-pound hammer
☒	Location of grab sample
▮	Rock coring interval
▽	Water level during drilling
▼	Water level taken on date shown

Graphic Log of Soil and Rock Types



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

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

GEOTECHNICAL TESTING EXPLANATIONS

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

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EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOILS

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 SOILS

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 SOILS (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVELS (< 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
		GRAVELS 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 SANDS (<5% fines)	SW or SP	SAND
		SANDS WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SANDS WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS (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 SOILS			PT	PEAT

MOISTURE CLASSIFICATION

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

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.					
Percent	Silt and Clay In:		Percent	Sand and Gravel In:	
	Fine-Grained Soils	Coarse-Grained Soils		Fine-Grained Soils	Coarse-Grained Soils
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

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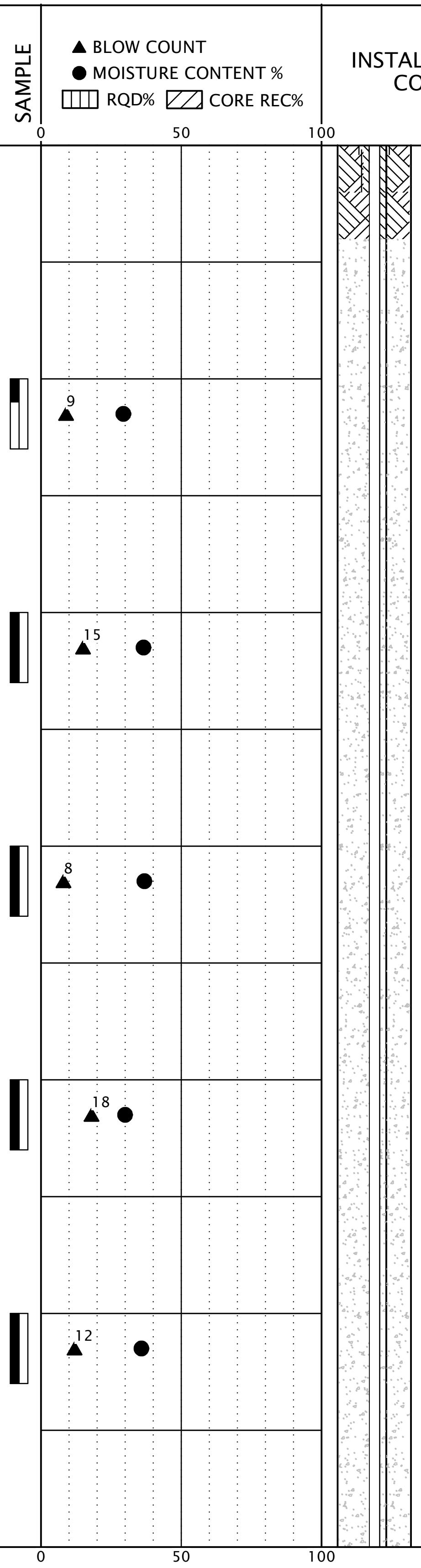
SOIL CLASSIFICATION SYSTEM

TABLE A-2



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	AGGREGATE BASE (~8.0 inches).				Flush-mount monument with 2 feet concrete backfill.
	Stiff, light gray-brown with orange mottled SILT with sand and gravel (ML); moist (landslide debris).	0.7			
2.5					Cement-bentonite grout 1/2-inch tremie pipe
5.0			PP	9	PP = 0.5 tsf
7.5					
10.0					▼ 10.1 feet, 9/15/17
	Stiff to very stiff, light brown with orange mottled, sandy CLAY (CH), some silt; moist, medium plasticity, sand is fine, weakly indurated, relict rock structure (landslide debris).	10.5	PP	15	PP = >4.5 tsf
12.5					
15.0	dark gray at 15.0 feet		ATT PP	8	PP = 1.5 tsf LL = 56% PL = 24%
17.5					
20.0	very stiff, with gravel; gravel is subangular at 20.0 feet		PP	18	Firm, smooth drilling from 20.0 to 25.0 feet. PP = 2.5 tsf
22.5					
25.0	stiff, dark gray with brown streaks and mottles, minor clay, without gravel at 25.0 feet		PP	12	PP = >4.5 tsf
27.5					
30.0					

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: CMC

COMPLETED: 09/11/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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SEASIDESD-1-03-01

NOVEMBER 2017

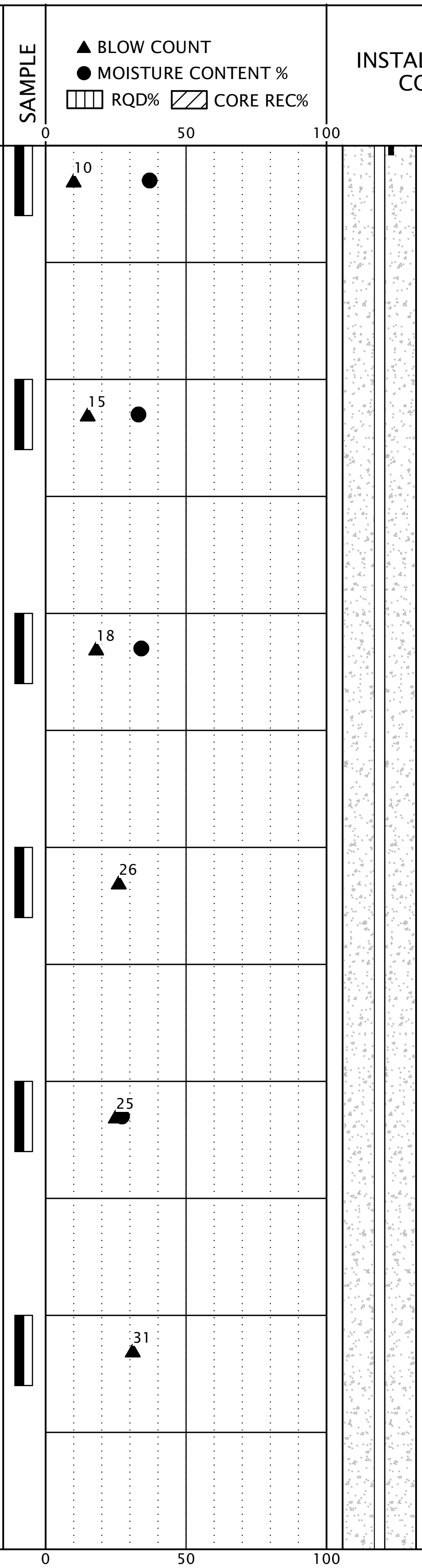
BORING B-1

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-1

BORING LOG SEASIDESD-1-03-01-B1_5-TP1_22,26.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:RC:KT

DEPTH FEET	MATERIAL DESCRIPTION	TESTING	▲ BLOW COUNT	INSTALLATION AND COMMENTS
30.0	(continued from previous page)		10	Vibrating wire piezometer #1600139 set at 30.0 feet
35.0	trace gravel at 35.0 feet		15	Gravel clasts are angular and consist of more indurated pieces of siltstone at 35.0 feet.
40.0	very stiff, dark gray with orange mottles, with gravel; gravel is fine to medium (basalt and sandstone clasts) at 40.0 feet	PP	18	PP = 3.0 tsf
42.5				Driller Comment: harder at 42.0 feet.
45.0		PP	26	PP = 4.0 tsf
50.0			25	
55.0	very stiff to hard at 55.0 feet		31	
60.0				



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: CMC

COMPLETED: 09/11/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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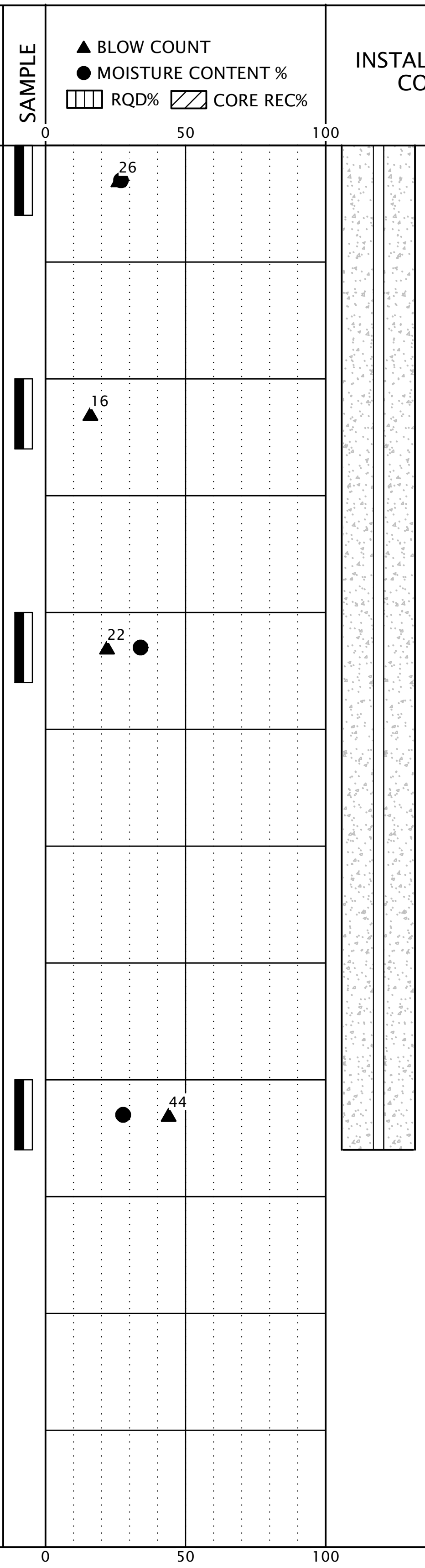
NOVEMBER 2017

BORING B-1
(continued)

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
60.0	with gravel at 60.0 feet		
64.0	Very stiff, dark gray with green and black patched CLAY with gravel (CH), minor sand; moist, medium to high plasticity, blocky and disturbed structure (landslide debris).	64.0	Driller Comment: chatter from 63.0 to 64.0 feet.
70.0	dark gray-brown, trace gravel; blocky and undisturbed structure at 70.0 feet		
74.0	Dense, gray-green, silty SAND (SM); moist, fine, weakly indurated (decomposed to weathered siltstone/sandstone; Astoria Formation).	74.0	Chatter at 74.0 feet.
81.5	layer of dark brown to dark green-gray SAND; fine to coarse at 81.0 feet Exploration completed at a depth of 81.5 feet. Hammer efficiency factor is 88.3 percent.	81.5	Surface elevation was not measured at the time of exploration.



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: CMC
COMPLETED: 09/11/17

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

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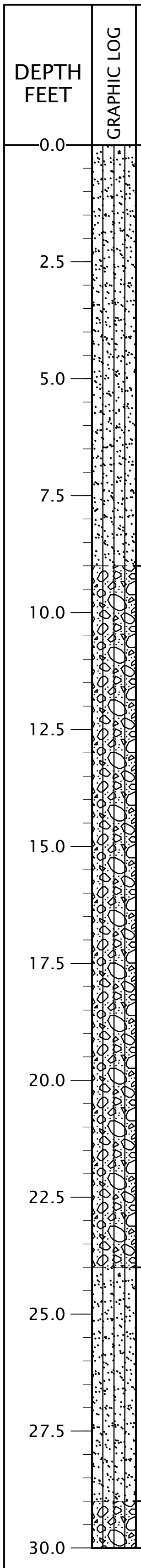
NOVEMBER 2017

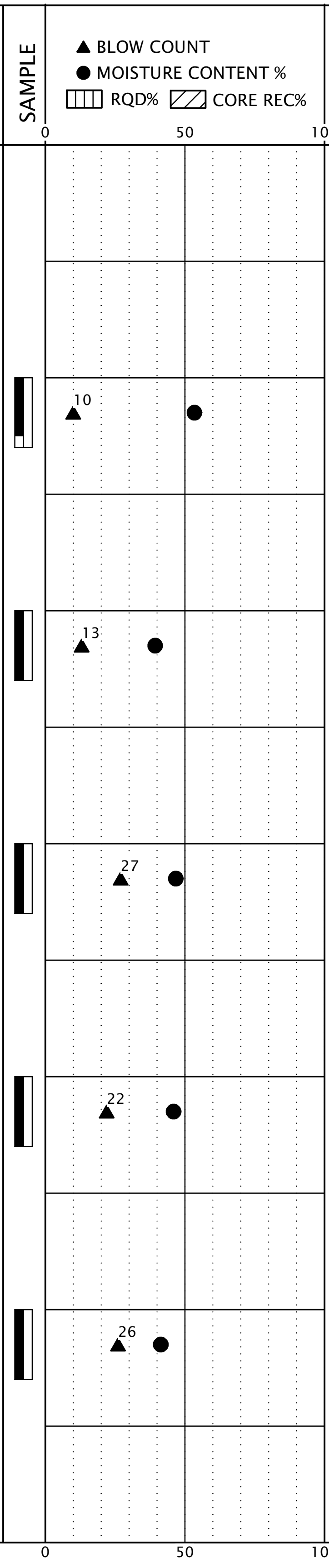
BORING B-1
(continued)

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-1



MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
Medium dense, dark red-brown, silty SAND with gravel (SM); moist, fine to coarse and subangular (landslide debris).			
Medium dense, red-brown and black, silty GRAVEL with sand (GM); moist, fine to coarse and subangular to angular (landslide debris).	9.0		Heavy chatter. Smoother drilling at 18.0 feet.
Medium dense, red-brown and dark gray-brown, silty SAND with gravel (SM); moist, fine to coarse and subangular to subrounded, gravel is fine (landslide debris).	24.0		Smoother drilling.
Very dense, dark gray-brown, silty GRAVEL with sand (GM); moist, fine to	29.0		



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: CMC
COMPLETED: 09/12/17

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

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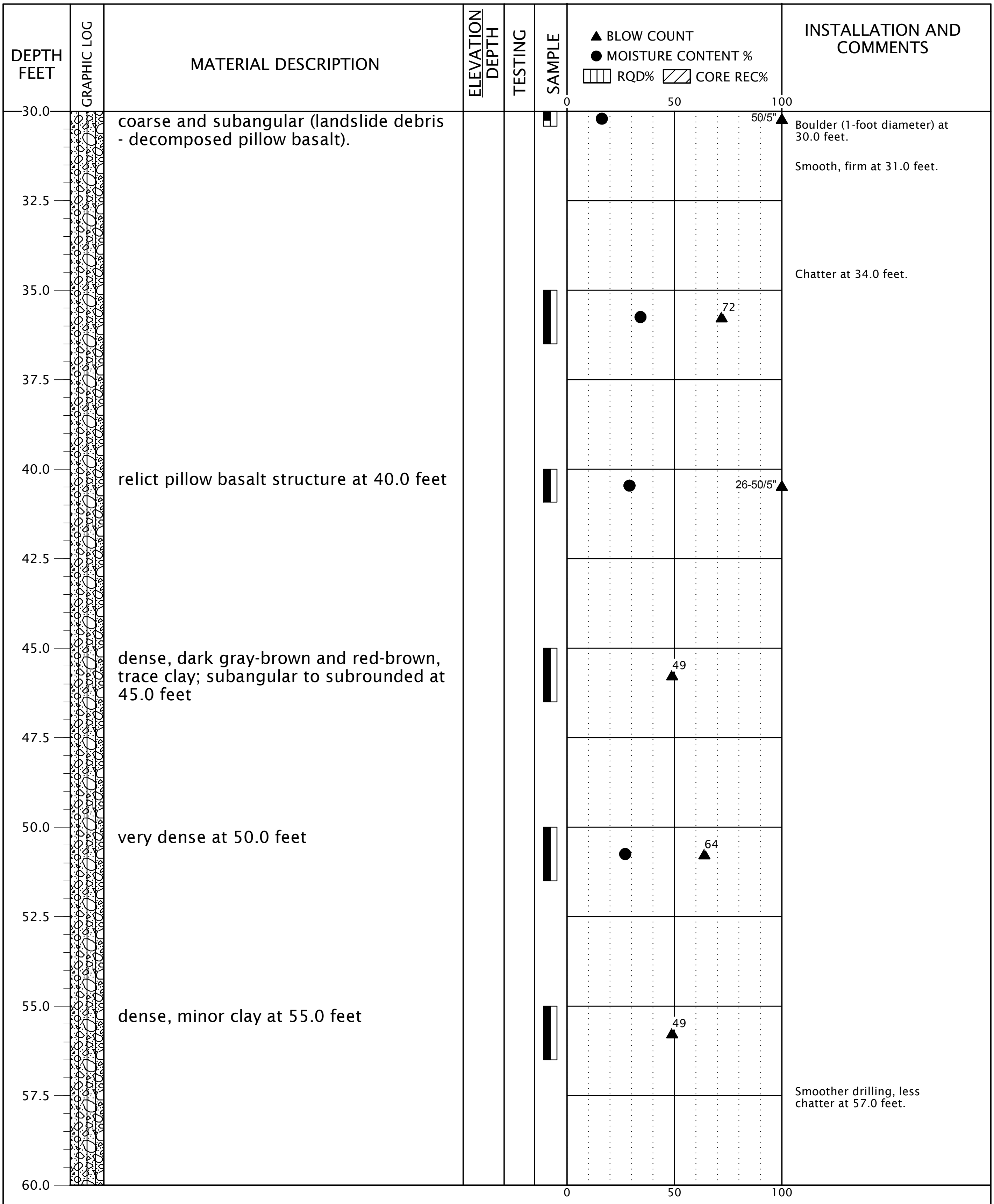
BORING B-2

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-2



coarse and subangular (landslide debris - decomposed pillow basalt).

relict pillow basalt structure at 40.0 feet

dense, dark gray-brown and red-brown, trace clay; subangular to subrounded at 45.0 feet

very dense at 50.0 feet

dense, minor clay at 55.0 feet

Boulder (1-foot diameter) at 30.0 feet.

Smooth, firm at 31.0 feet.

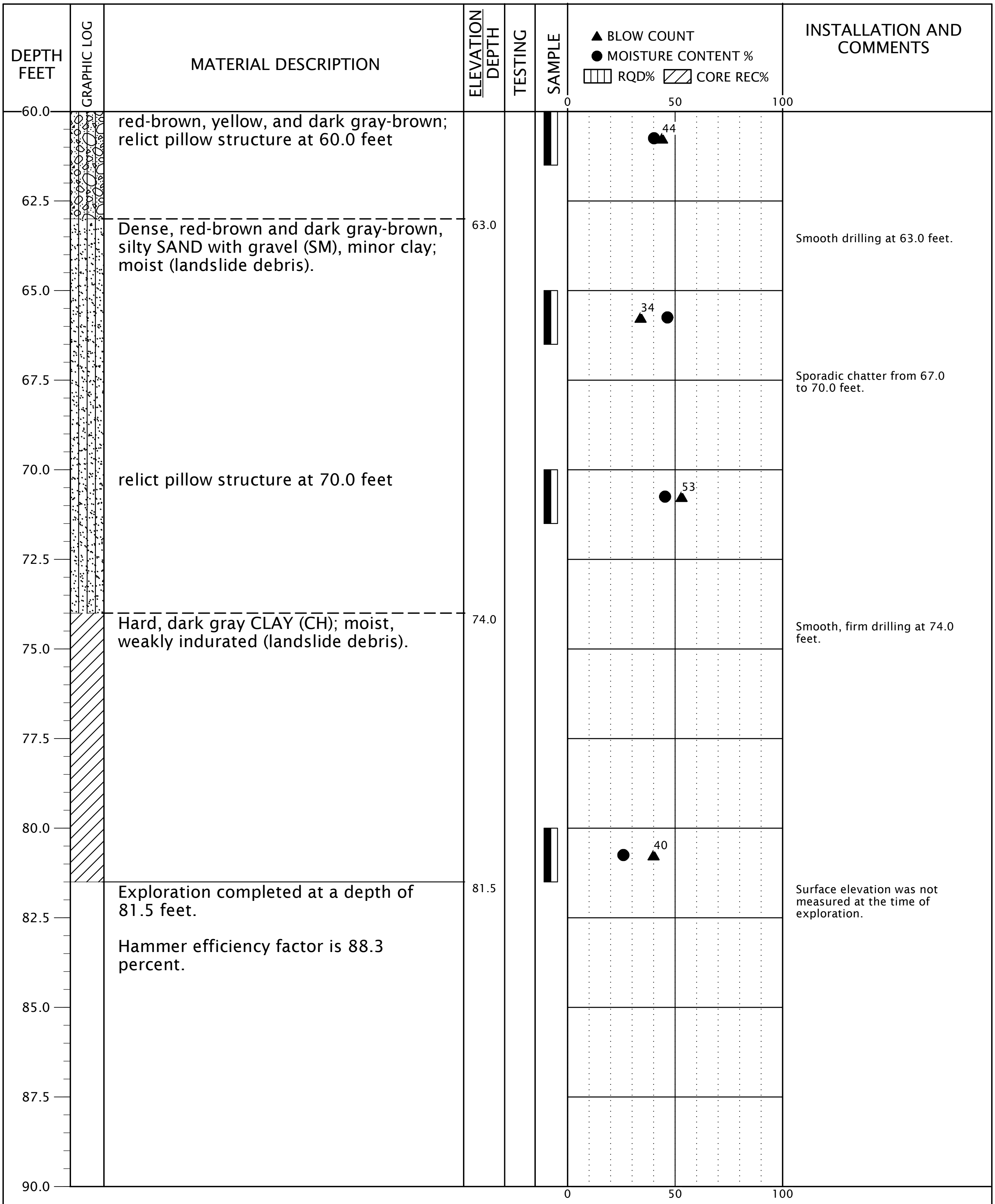
Chatter at 34.0 feet.

Smoother drilling, less chatter at 57.0 feet.

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: CMC COMPLETED: 09/12/17

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

GEODESIGN INC 9450 SW Commerce Circle - Suite 300 Wilsonville OR 97070 503.968.8787 www.geodesigninc.com	SEASIDESD-1-03-01	**BORING B-2** (continued)
	NOVEMBER 2017	SEASIDE SCHOOL DISTRICT CAMPUS SEASIDE, OR
		FIGURE A-2



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
60.0	red-brown, yellow, and dark gray-brown; relict pillow structure at 60.0 feet		
63.0	Dense, red-brown and dark gray-brown, silty SAND with gravel (SM), minor clay; moist (landslide debris).	63.0	Smooth drilling at 63.0 feet.
67.0			Sporadic chatter from 67.0 to 70.0 feet.
70.0	relict pillow structure at 70.0 feet		
74.0	Hard, dark gray CLAY (CH); moist, weakly indurated (landslide debris).	74.0	Smooth, firm drilling at 74.0 feet.
81.5	Exploration completed at a depth of 81.5 feet. Hammer efficiency factor is 88.3 percent.	81.5	Surface elevation was not measured at the time of exploration.

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: CMC

COMPLETED: 09/12/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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SEASIDESD-1-03-01

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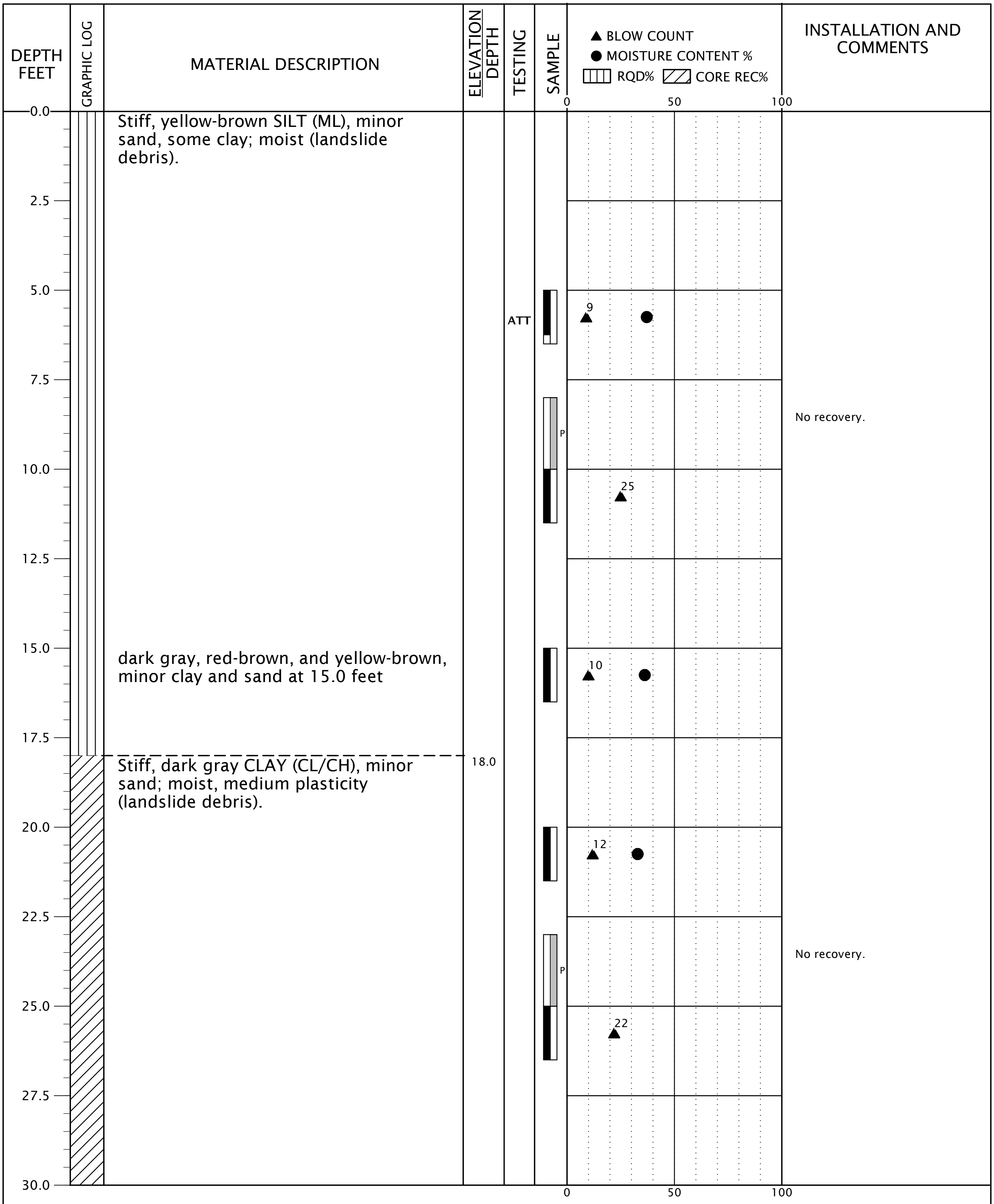
BORING B-2
(continued)

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-2



Stiff, yellow-brown SILT (ML), minor sand, some clay; moist (landslide debris).

dark gray, red-brown, and yellow-brown, minor clay and sand at 15.0 feet

Stiff, dark gray CLAY (CL/CH), minor sand; moist, medium plasticity (landslide debris).

No recovery.

No recovery.

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: CMC

COMPLETED: 09/13/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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SEASIDESD-1-03-01

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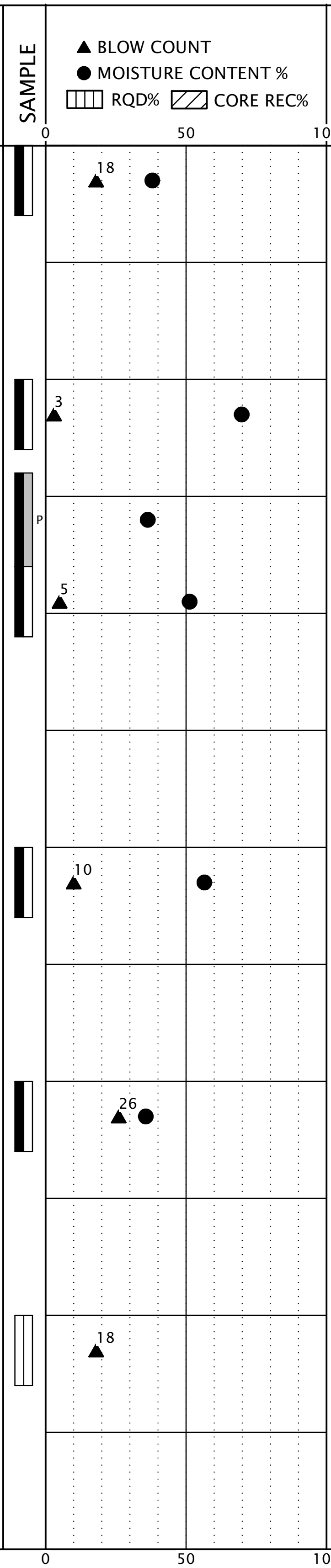
BORING B-3

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-3

BORING LOG SEASIDESD-1-03-01-B1_5-TP1_22,26.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:RC:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
30.0	very stiff, with white siltstone clast at 30.0 feet			
34.0	Soft, dark gray CLAY with organics and charcoal wood fragments (CH); moist to wet (landslide debris).	34.0		
37.5			DD DS	DD = 83 pcf
39.0	medium stiff, trace to minor sand; wet, sand is fine to medium at 39.0 feet			
43.0	Stiff, brown to light blue-gray CLAY (CL/CH), minor sand, trace siltstone fragments; moist, sand is fine (landslide debris).	43.0		
49.0	Medium dense, gray, silty SAND (SM); moist, fine, weakly indurated (landslide debris).	49.0		
55.0	with clay at 55.0 feet			
56.5	Exploration completed at a depth of 56.5 feet. Hammer efficiency factor is 88.3 percent.	56.5		Surface elevation was not measured at the time of exploration.



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: CMC
COMPLETED: 09/13/17

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

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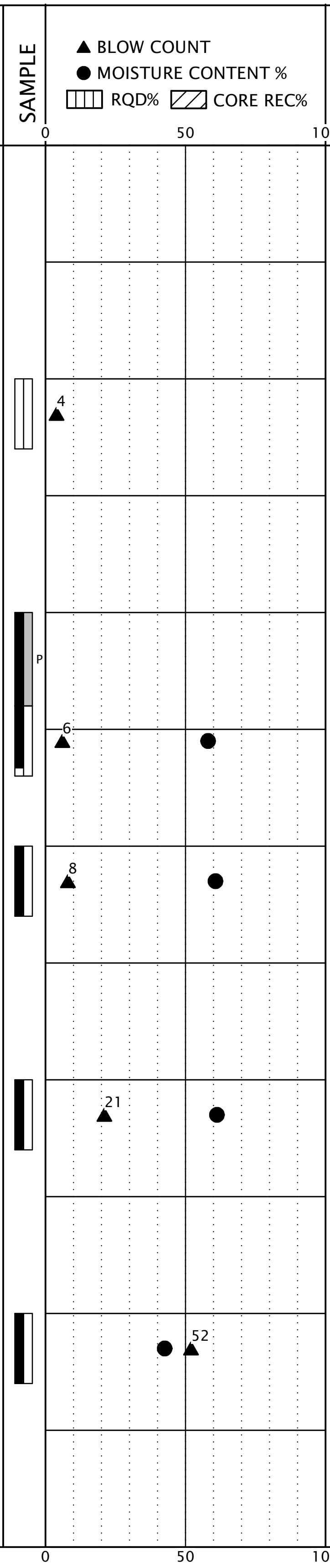
BORING B-3
(continued)

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
0.0	Soft, brown, sandy SILT with wood debris (ML); moist - **FILL**.		
9.0	Medium stiff, gray CLAY with sand (CH), minor gravel, trace wood; moist to wet, medium to high plasticity (landslide debris).	9.0	Loss of circulation at 8.0 feet. Circulation returns at 9.0 feet.
14.0	Loose, black, silty SAND (SM), minor gravel; wet, fine to coarse and subangular to angular (landslide debris). dark gray, with clay at 16.0 feet	14.0	
20.0	medium dense; moist at 20.0 feet		
23.0	Very dense, dark gray, silty SAND (SM), minor gravel, trace clay; moist, fine to coarse and subangular to angular, gravel is fine, weakly indurated, brecciated structure (landslide debris).	23.0	Harder drilling at 23.0 feet. Brecciated siltstone is healed.



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: CMC

COMPLETED: 09/14/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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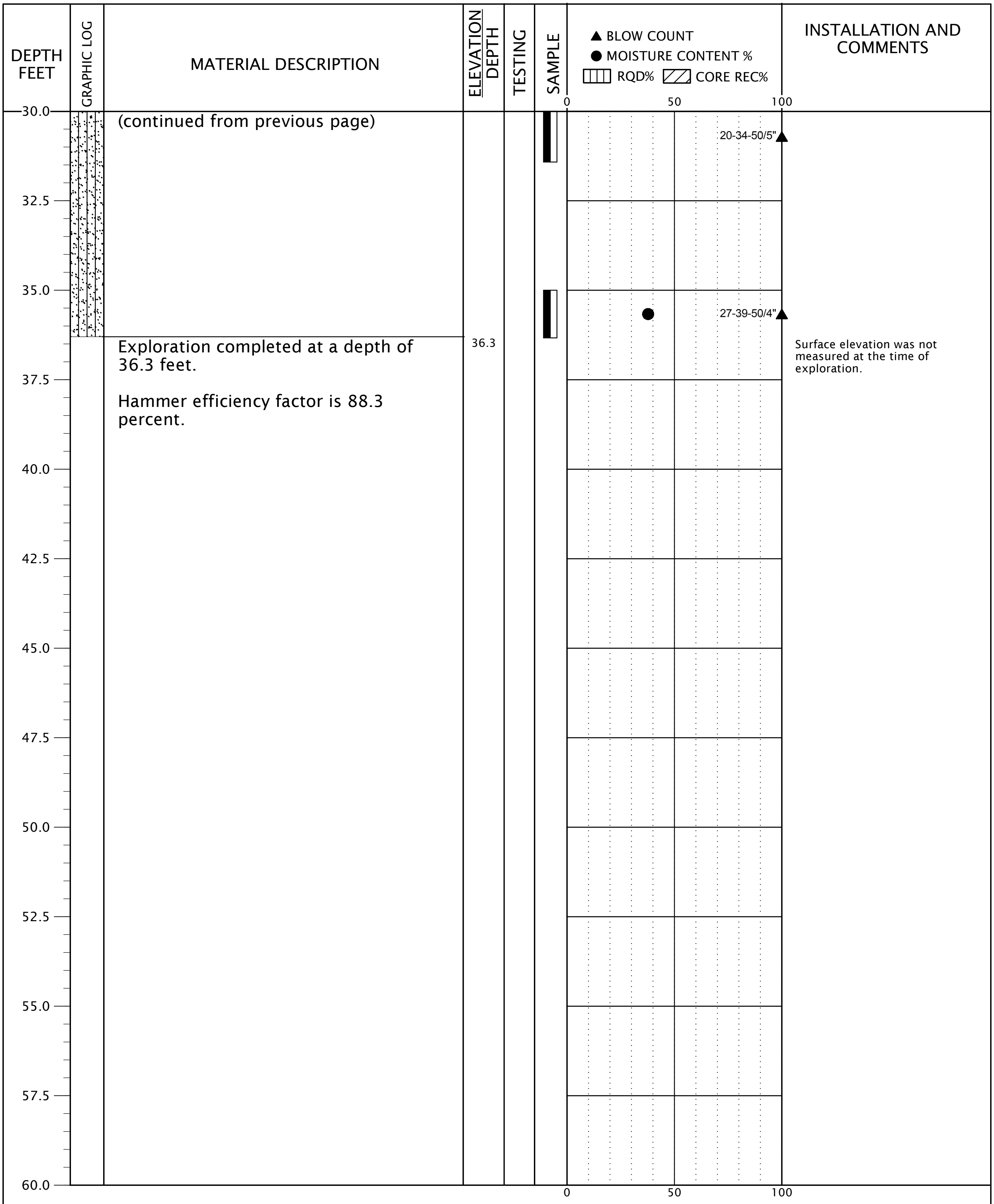
BORING B-4

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-4

BORING LOG SEASIDESD-1-03-01-B1_5-TP1_22,26.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:RC:KT



(continued from previous page)

Exploration completed at a depth of 36.3 feet.

Hammer efficiency factor is 88.3 percent.

Surface elevation was not measured at the time of exploration.

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: CMC

COMPLETED: 09/14/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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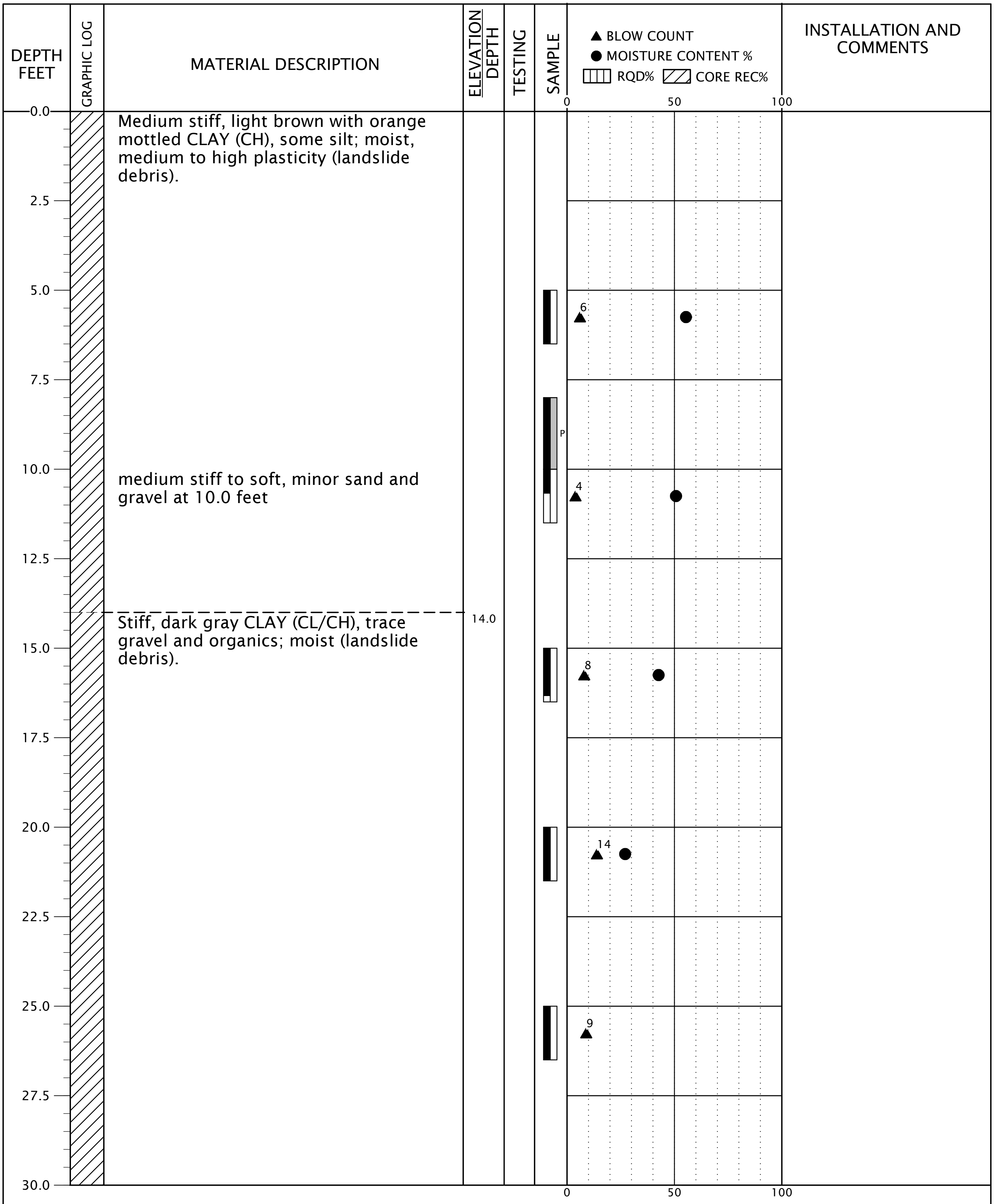
BORING B-4
(continued)

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-4



Medium stiff, light brown with orange mottled CLAY (CH), some silt; moist, medium to high plasticity (landslide debris).

medium stiff to soft, minor sand and gravel at 10.0 feet

Stiff, dark gray CLAY (CL/CH), trace gravel and organics; moist (landslide debris).

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: CMC

COMPLETED: 09/15/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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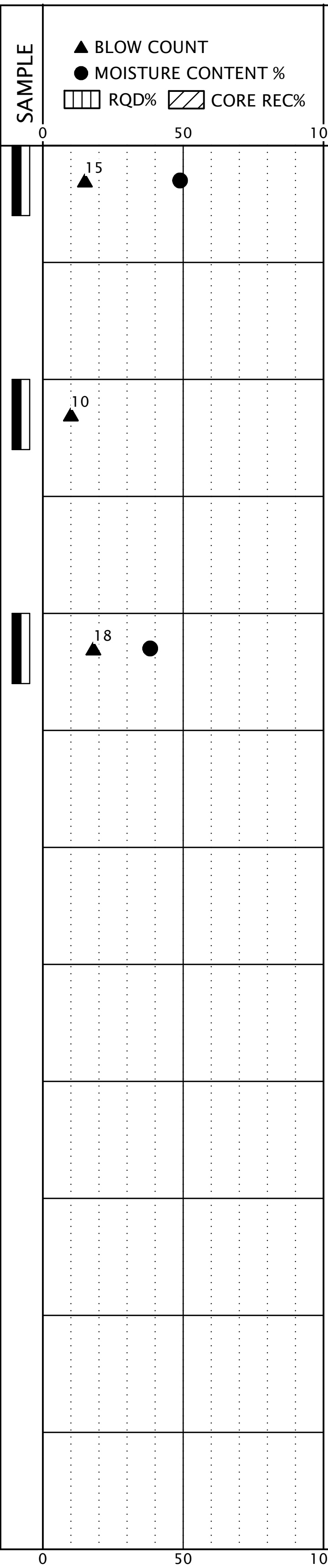
BORING B-5

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-5

BORING LOG SEASIDESD-1-03-01-B1_5-TP1_22,26.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:RC:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
30.0	stiff to very stiff, with wood and carbonized organics at 30.0 feet		
35.0	stiff, light gray at 35.0 feet		
40.0	very stiff, dark gray at 40.0 feet		
41.5	Exploration completed at a depth of 41.5 feet. Hammer efficiency factor is 88.3 percent.	41.5	Surface elevation was not measured at the time of exploration.



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: CMC
COMPLETED: 09/15/17

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

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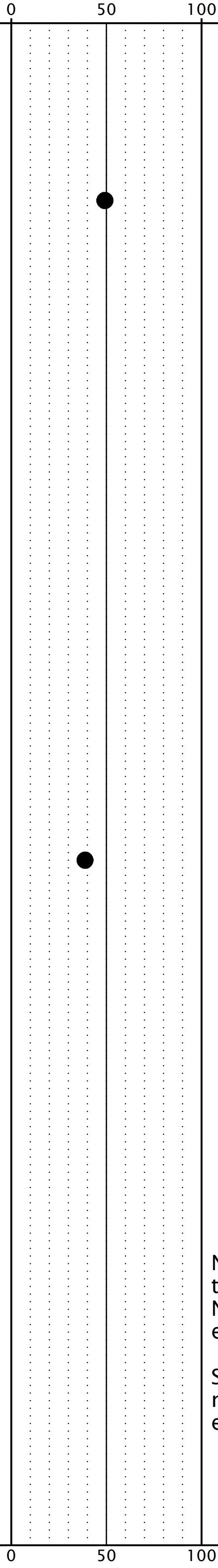
BORING B-5
(continued)

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-5

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium stiff, brown SILT (ML), trace organics and sand; moist (topsoil, 8-inch-thick root zone).				
	Medium stiff to stiff, brown with orange mottled CLAY (CL/CH), some silt, trace sand; moist.	1.0		☒	
2.5					
	brown with light gray-brown streaks at 3.5 feet			☒	
5.0					
	stiff to very stiff at 6.0 feet				
	Very stiff, light brown with orange streaked CLAY (CL/CH), minor silt, trace sand; moist.	7.0			
7.5					
				☒	
10.0	dark gray with orange streaks at 10.0 feet				
	very stiff to hard at 11.0 feet				
	Exploration completed at a depth of 12.0 feet.	12.0			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					
15.0					

● MOISTURE CONTENT %



EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: RSK

COMPLETED: 09/06/17

EXCAVATION METHOD: trackhoe (see document text)

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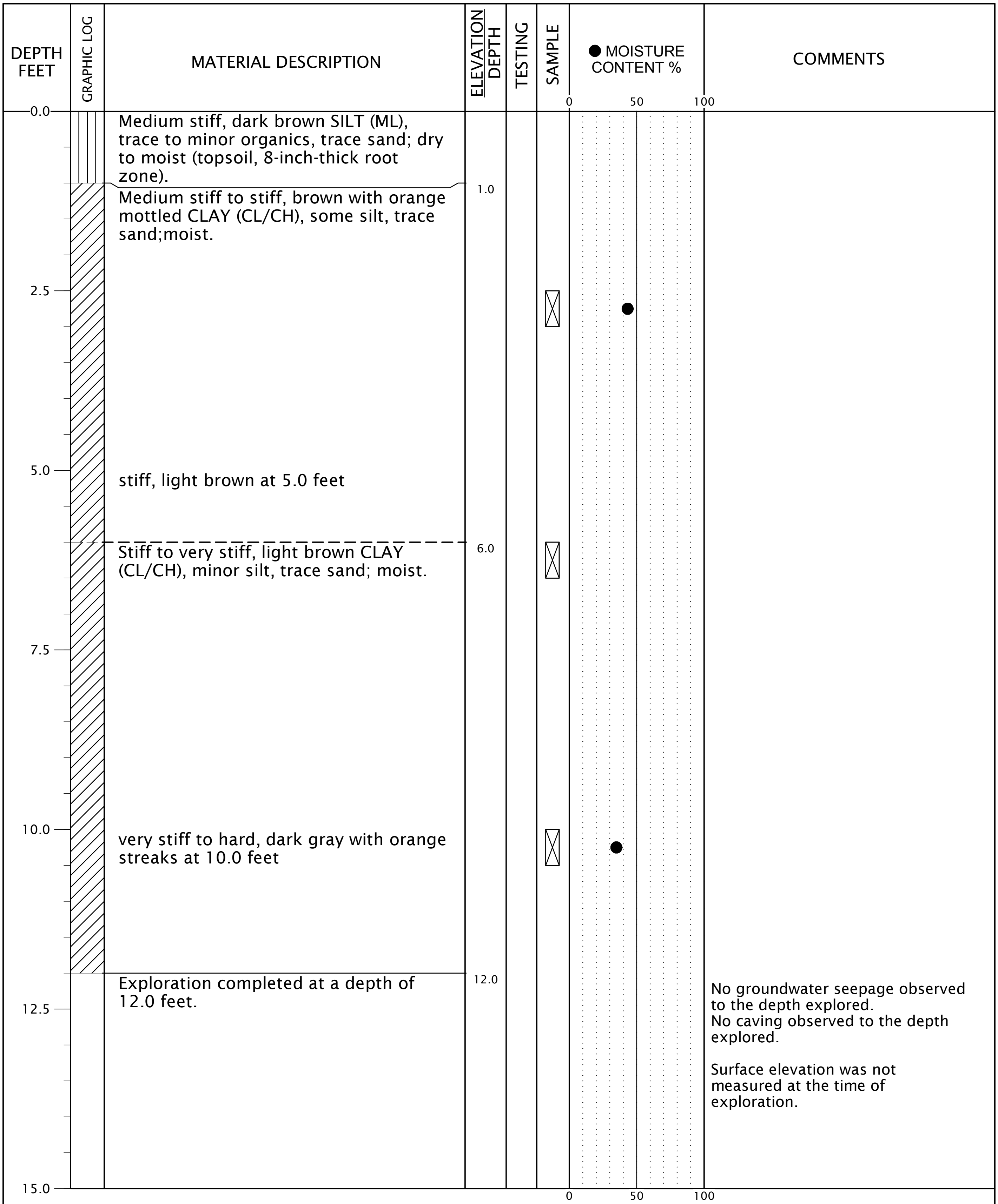
TEST PIT TP-1

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-6

TEST PIT LOG - 1 PER PAGE SEASIDESD-1-03-01-B1_5-TP1_22,26.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:RC:KT



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0	Medium stiff, dark brown SILT (ML), trace to minor organics, trace sand; dry to moist (topsoil, 8-inch-thick root zone).					
	Medium stiff to stiff, brown with orange mottled CLAY (CL/CH), some silt, trace sand;moist.	1.0				
2.5				☒	●	
5.0	stiff, light brown at 5.0 feet					
	Stiff to very stiff, light brown CLAY (CL/CH), minor silt, trace sand; moist.	6.0		☒		
7.5						
10.0	very stiff to hard, dark gray with orange streaks at 10.0 feet			☒	●	
	Exploration completed at a depth of 12.0 feet.	12.0				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						
15.0						

EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: RSK

COMPLETED: 09/06/17

EXCAVATION METHOD: trackhoe (see document text)

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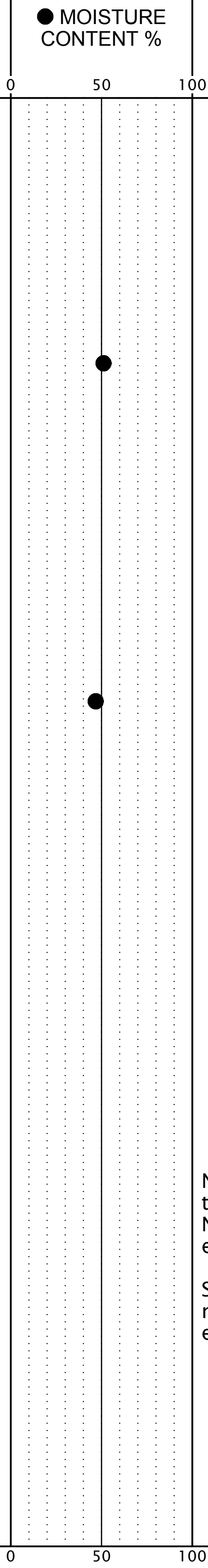
TEST PIT TP-2

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-7

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	COMMENTS
0.0	▥	Medium stiff, dark brown SILT (ML), trace to minor organics, trace sand; dry to moist (topsoil, 10-inch-thick root zone).				
	▨	Stiff, brown CLAY (CL/CH), some silt, trace sand; moist.	1.0			
2.5	▨				☒	
5.0	▨	brown with orange mottles at 5.0 feet				
	▨	stiff to very stiff, brown with orange mottles at 6.0 feet			☒	
7.5	▨					
	▨	dark gray at 8.0 feet				
10.0	▨	hard at 10.0 feet				
		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						
15.0						



EXCAVATED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: RSK
COMPLETED: 09/06/17

EXCAVATION METHOD: trackhoe (see document text)

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TEST PIT TP-3

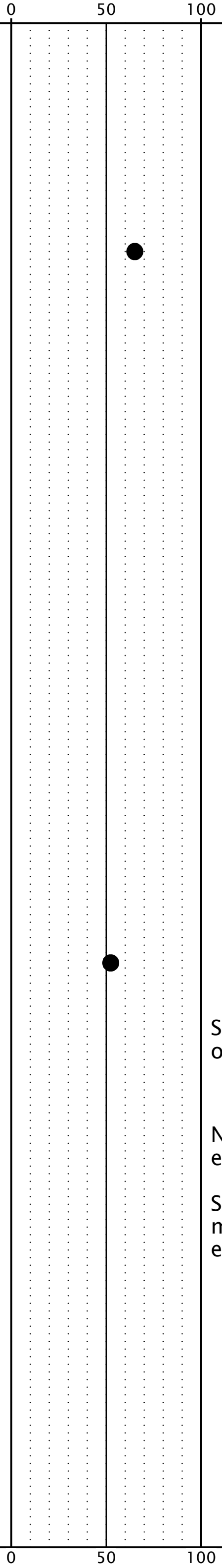
SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-8

TEST PIT LOG - 1 PER PAGE SEASIDESD-1-03-01-B1_5-TP1_22,26.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:RC:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium stiff, dark brown SILT (ML), trace to minor organics, trace sand; dry to moist (topsoil, 6-inch-thick root zone).				
	Medium stiff to stiff, brown with orange mottled CLAY (CL/CH), some silt, trace sand; moist.	1.0		☒	
2.5					
5.0					
	boulder at 6.0 feet				
7.5	Very stiff, dark brown with yellow spotted CLAY (CL/CH), minor sand, trace gravel; moist.	7.5		☒	
10.0	hard at 10.0 feet				Slow groundwater seepage observed at 10.0 feet
	Exploration completed at a depth of 11.0 feet.	11.0			No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
12.5					
15.0					

● MOISTURE CONTENT %



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LOGGED BY: RSK

COMPLETED: 09/06/17

EXCAVATION METHOD: trackhoe (see document text)

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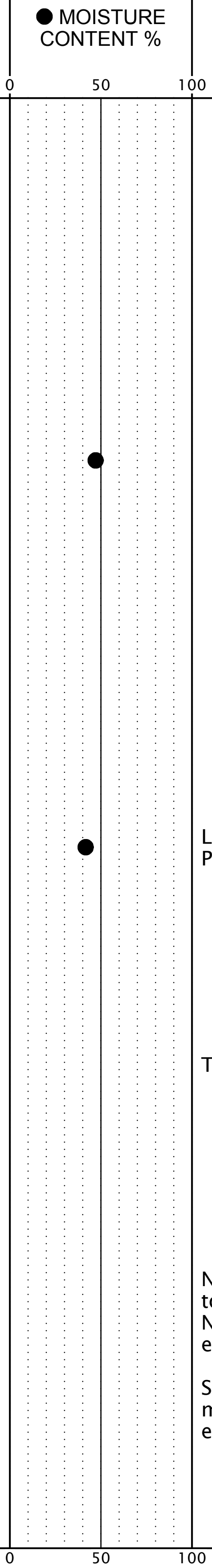
TEST PIT TP-4

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-9

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium stiff, brown CLAY (CL/CH), minor silt, trace sand; moist.				
2.5	brown with orange mottles at 2.5 feet				
	stiff, light gray at 3.5 feet			☒	
5.0					
	Very stiff, dark gray CLAY (CH), trace sand; moist, medium to high plasticity.	7.0			
7.5			ATT	☒	LL = 85% PL = 31%
10.0					Tree limb at 10.0 feet.
	Exploration completed at a depth of 12.0 feet.	12.0			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					
15.0					



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TEST PIT TP-5

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-10

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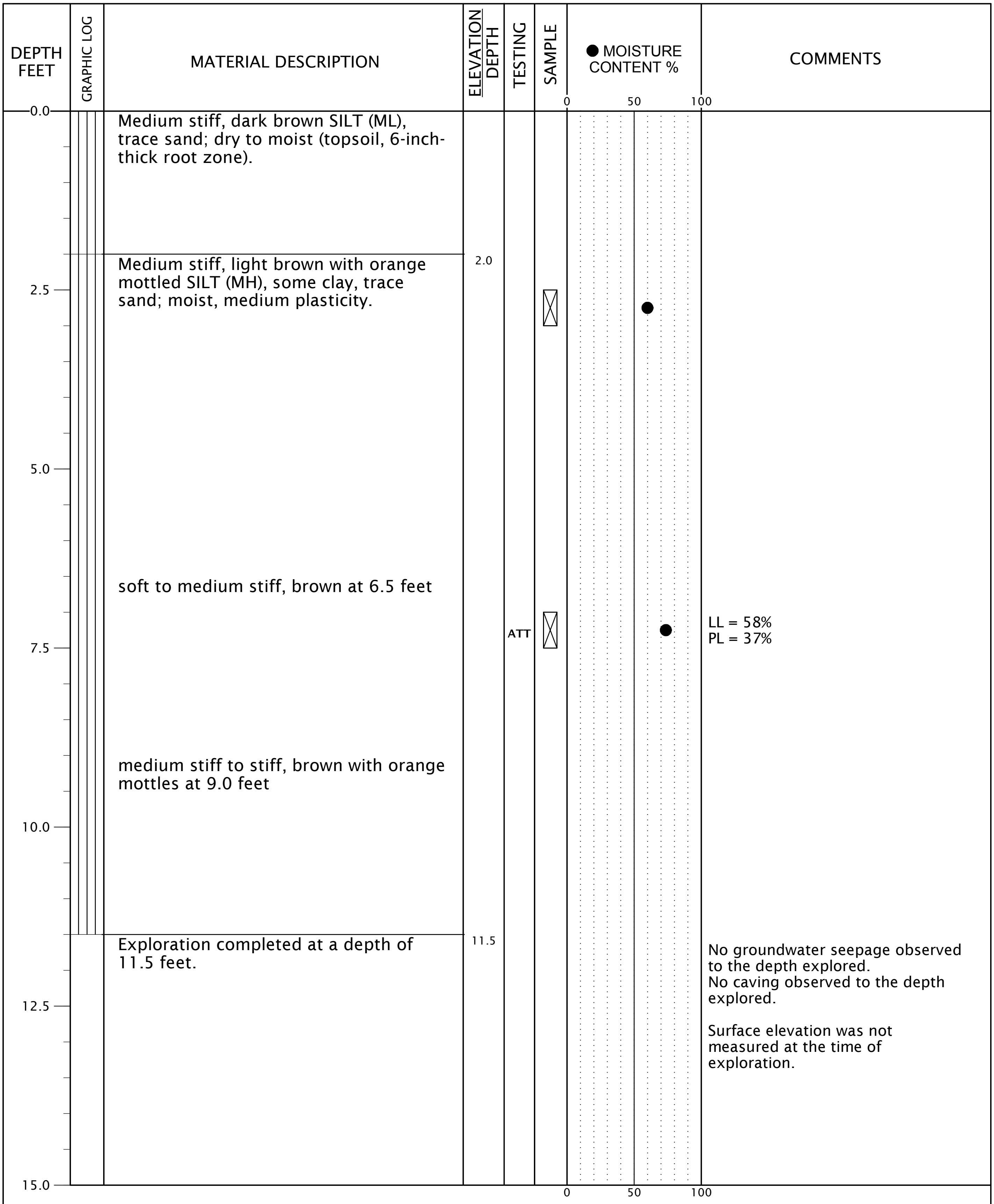

MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	COMMENTS
Medium stiff, dark brown SILT (ML), trace sand; dry to moist (topsoil, 6-inch-thick root zone).				
Medium stiff, light brown with orange mottled SILT (MH), some clay, trace sand; moist, medium plasticity.	2.0		☒	
soft to medium stiff, brown at 6.5 feet		ATT	☒	LL = 58% PL = 37%
medium stiff to stiff, brown with orange mottles at 9.0 feet				
Exploration completed at a depth of 11.5 feet.	11.5			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.

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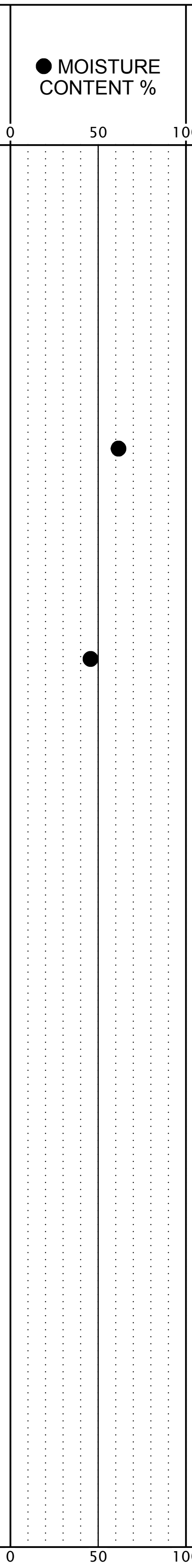
SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-11

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % 0 50 100	COMMENTS
0.0		WOOD DEBRIS (sticks, needles).					
2.5		Medium stiff, brown SILT (ML), trace to minor organics, trace sand; moist (topsoil).	2.0		☒	●	
		Medium stiff, brown with orange mottled CLAY (CL/CH), some silt, trace sand; moist.	3.5				
5.0				DD CON	▮	●	DD = 73 pcf
		stiff at 6.0 feet					
7.5		very stiff at 8.0 feet					
10.0					▮		
12.5		Exploration completed at a depth of 12.0 feet.	12.0				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.
15.0							



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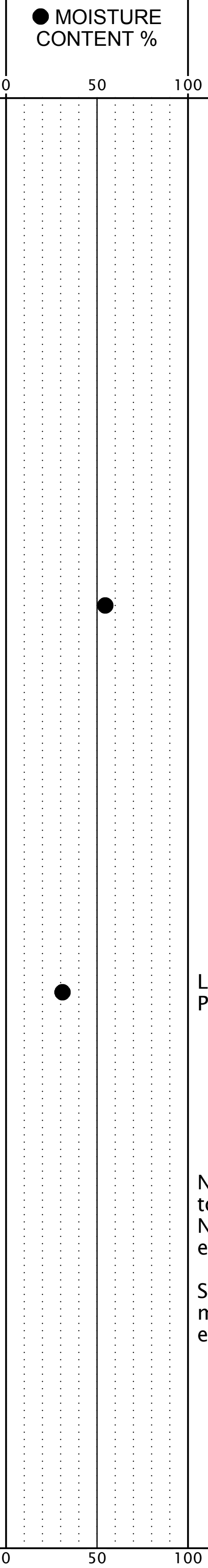
TEST PIT TP-7

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-12

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT %	COMMENTS
0.0		Medium stiff, brown SILT (ML), minor gravel, trace sand; moist (8-inch-thick root zone).					
					☒		
2.5							
		stiff at 3.0 feet					
5.0					☒	●	
7.5		stiff to very stiff, light brown with orange mottles, some clay at 7.5 feet					
				ATT	☒	●	LL = NP PL = NP
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							
15.0							



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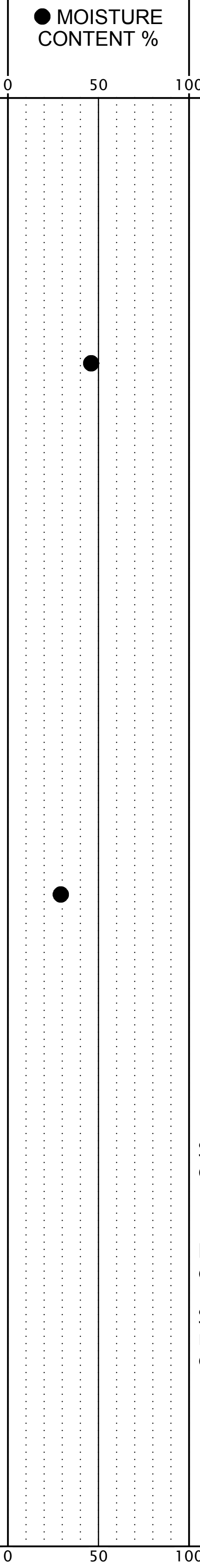
SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-13

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium stiff, dark brown SILT (ML), trace to minor organics, trace sand; moist (topsoil, 10-inch-thick root zone).				
	Medium stiff to stiff, gray with orange mottled CLAY (CL/CH), some silt, trace sand; moist.	1.5			
2.5				☒	
5.0					
7.5	very stiff to hard, dark gray at 7.5 feet				
				☒	
10.0					
					Slow groundwater seepage observed at 11.0 feet.
	Exploration completed at a depth of 12.0 feet.	12.0			No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
12.5					
15.0					



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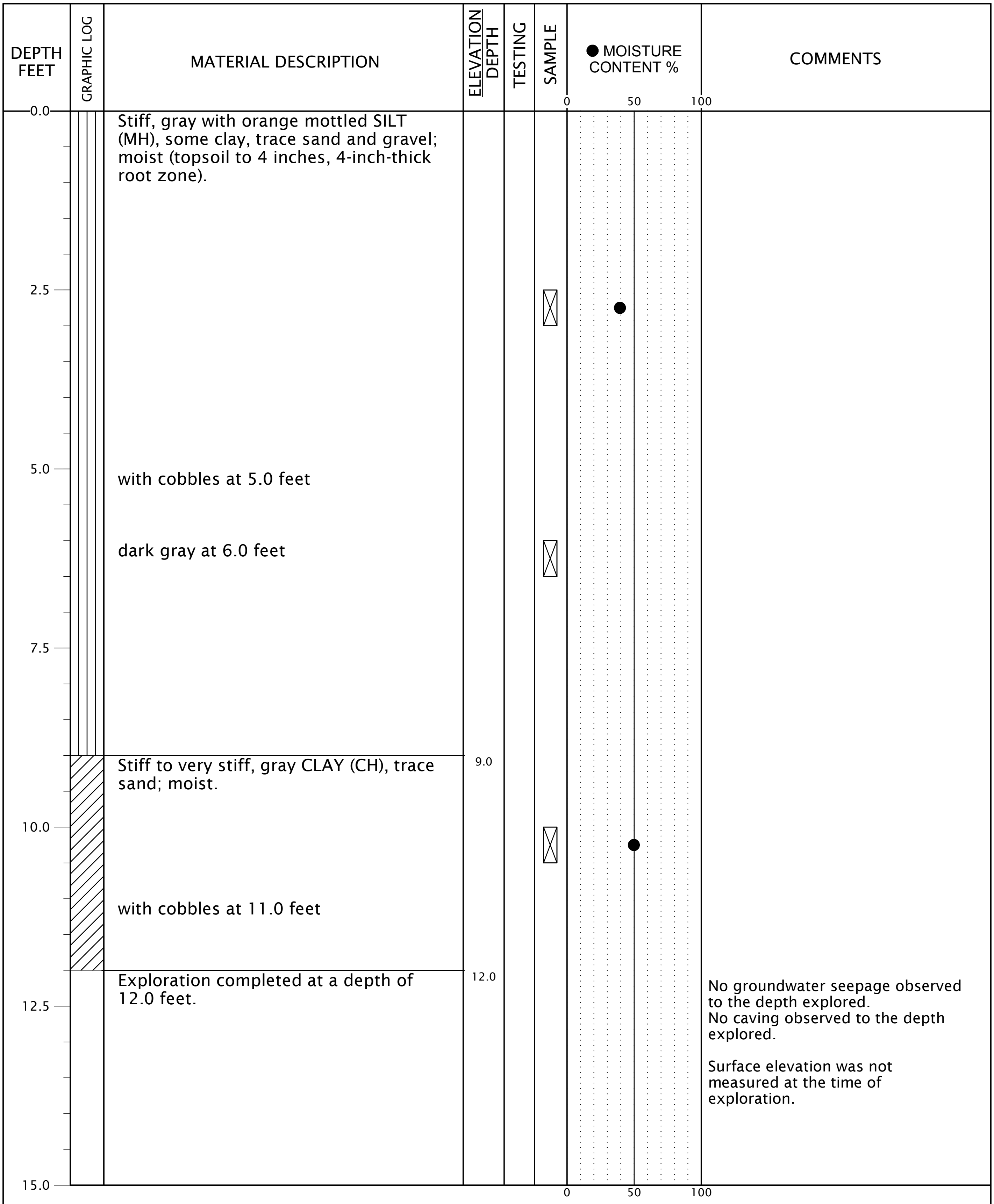
TEST PIT TP-9

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-14



Stiff, gray with orange mottled SILT (MH), some clay, trace sand and gravel; moist (topsoil to 4 inches, 4-inch-thick root zone).

with cobbles at 5.0 feet

dark gray at 6.0 feet

Stiff to very stiff, gray CLAY (CH), trace sand; moist.

with cobbles at 11.0 feet

Exploration completed at a depth of 12.0 feet.

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.

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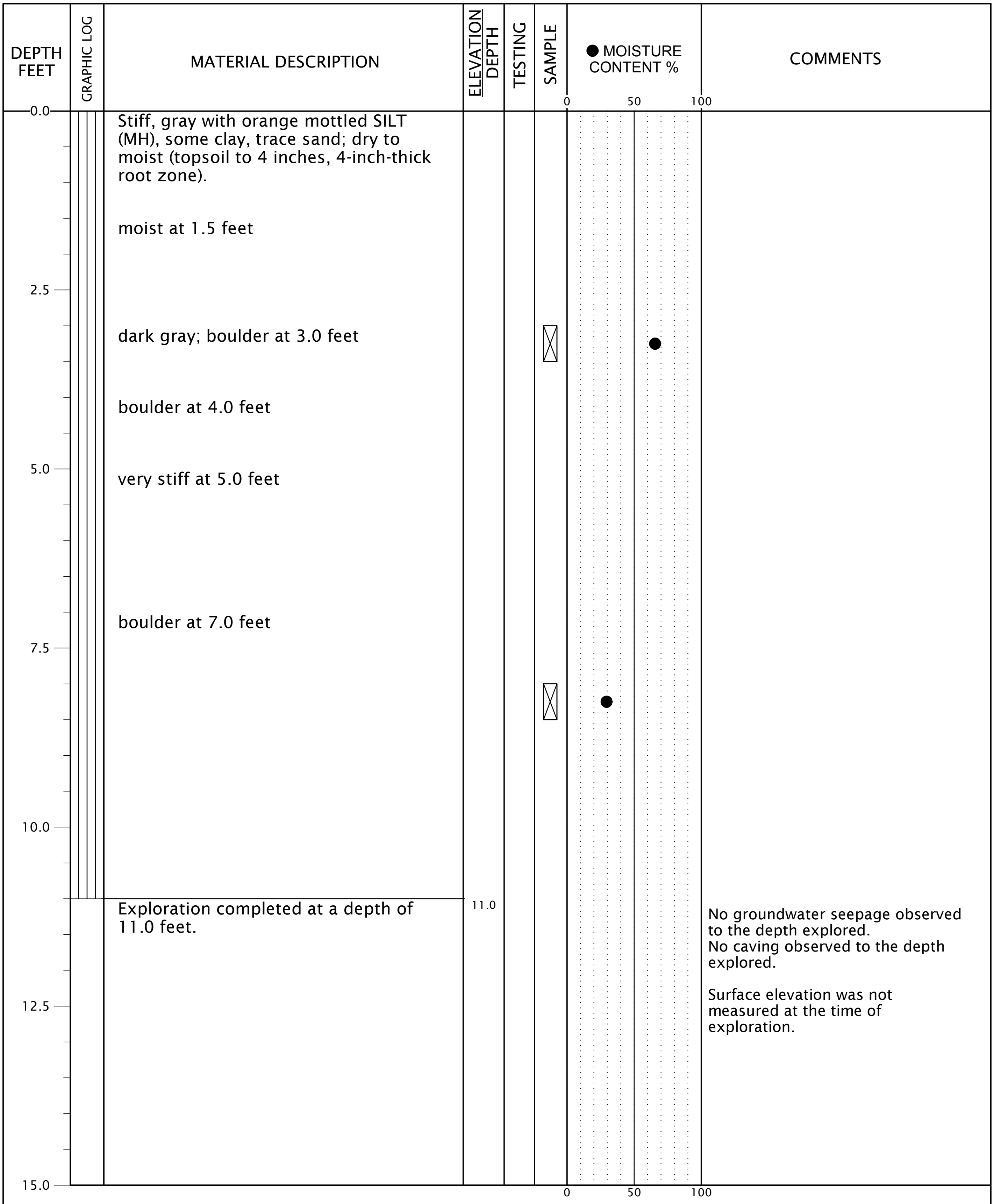
TEST PIT TP-10

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-15



Stiff, gray with orange mottled SILT (MH), some clay, trace sand; dry to moist (topsoil to 4 inches, 4-inch-thick root zone).

moist at 1.5 feet

dark gray; boulder at 3.0 feet

boulder at 4.0 feet

very stiff at 5.0 feet

boulder at 7.0 feet

Exploration completed at a depth of 11.0 feet.

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.

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TEST PIT TP-11

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-16

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		WOOD DEBRIS.					
		Soft to medium stiff, dark brown SILT (ML), trace to minor organics, trace sand; moist (topsoil).	2.0				
2.5							
	▨	Medium stiff to stiff, brown CLAY (CL/CH), some silt, trace sand; moist.	3.0				
	▨				☒		
5.0	▨						
	▨	stiff, light gray with orange mottles at 6.0 feet					
	▨				☒		
7.5	▨						
	▨	with cobbles at 9.0 feet					
10.0	▨				☒	●	Slow groundwater seepage observed at 10.0 feet.
		Exploration completed at a depth of 12.0 feet.	12.0				No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
12.5							
15.0							

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SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-17

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % 0 50 100	COMMENTS
0.0	▥	Medium stiff to stiff, brown SILT (MH), some clay, minor gravel, trace sand; moist (topsoil to 8 inches, 6-inch-thick root zone).					
2.5	▥						
	▥	with cobbles at 3.0 feet			☒		
5.0	▨	Very stiff, brown with yellow spotted CLAY with gravel and cobbles (CL/CH), some silt, minor sand; moist.	5.0				
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							
15.0						0 50 100	

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SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-18

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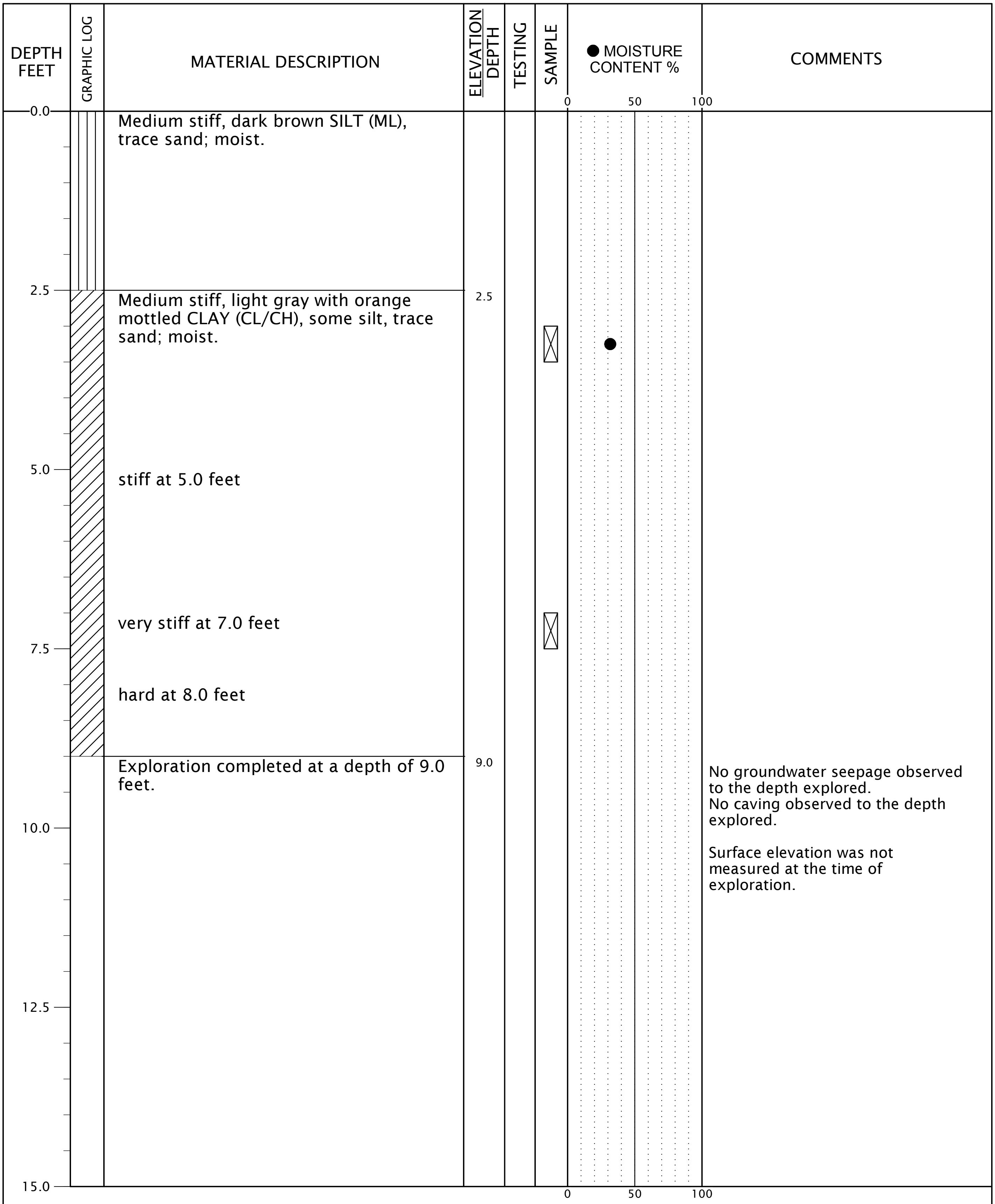

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	COMMENTS
0.0	Medium stiff, dark brown SILT (ML), trace sand; moist.		
2.5	Medium stiff, light gray with orange mottled CLAY (CL/CH), some silt, trace sand; moist.	2.5	
5.0	stiff at 5.0 feet		
	very stiff at 7.0 feet		
	hard at 8.0 feet		
	Exploration completed at a depth of 9.0 feet.	9.0	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.

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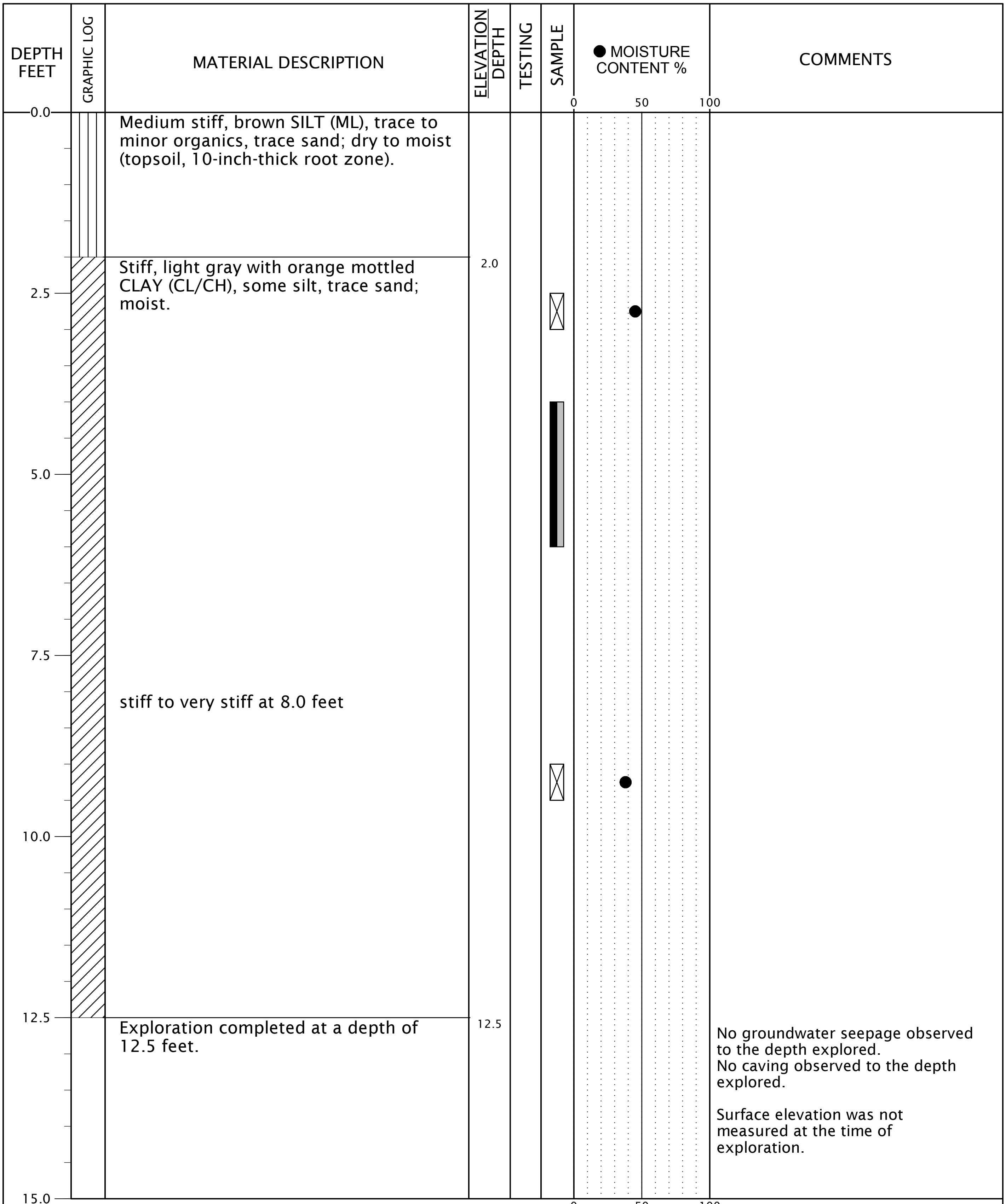
TEST PIT TP-14

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-19



Medium stiff, brown SILT (ML), trace to minor organics, trace sand; dry to moist (topsoil, 10-inch-thick root zone).

Stiff, light gray with orange mottled CLAY (CL/CH), some silt, trace sand; moist.

stiff to very stiff at 8.0 feet

Exploration completed at a depth of 12.5 feet.

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.

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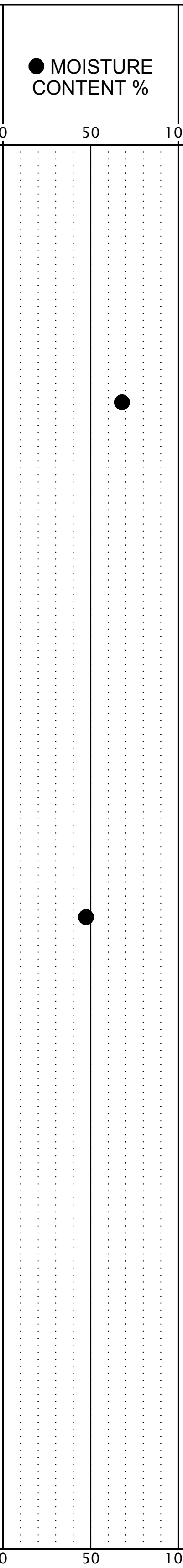
SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-20

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	COMMENTS
0.0	▥	Soft to medium stiff, brown SILT (ML), trace sand and organics; dry to moist (topsoil, 4-inch-thick root zone).				
2.5	▨	Medium stiff, brown CLAY (CL/CH), some silt, trace sand; moist.	2.5		☒	
		medium stiff to stiff, brown with orange mottles at 8.0 feet			☒	
11.0		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.



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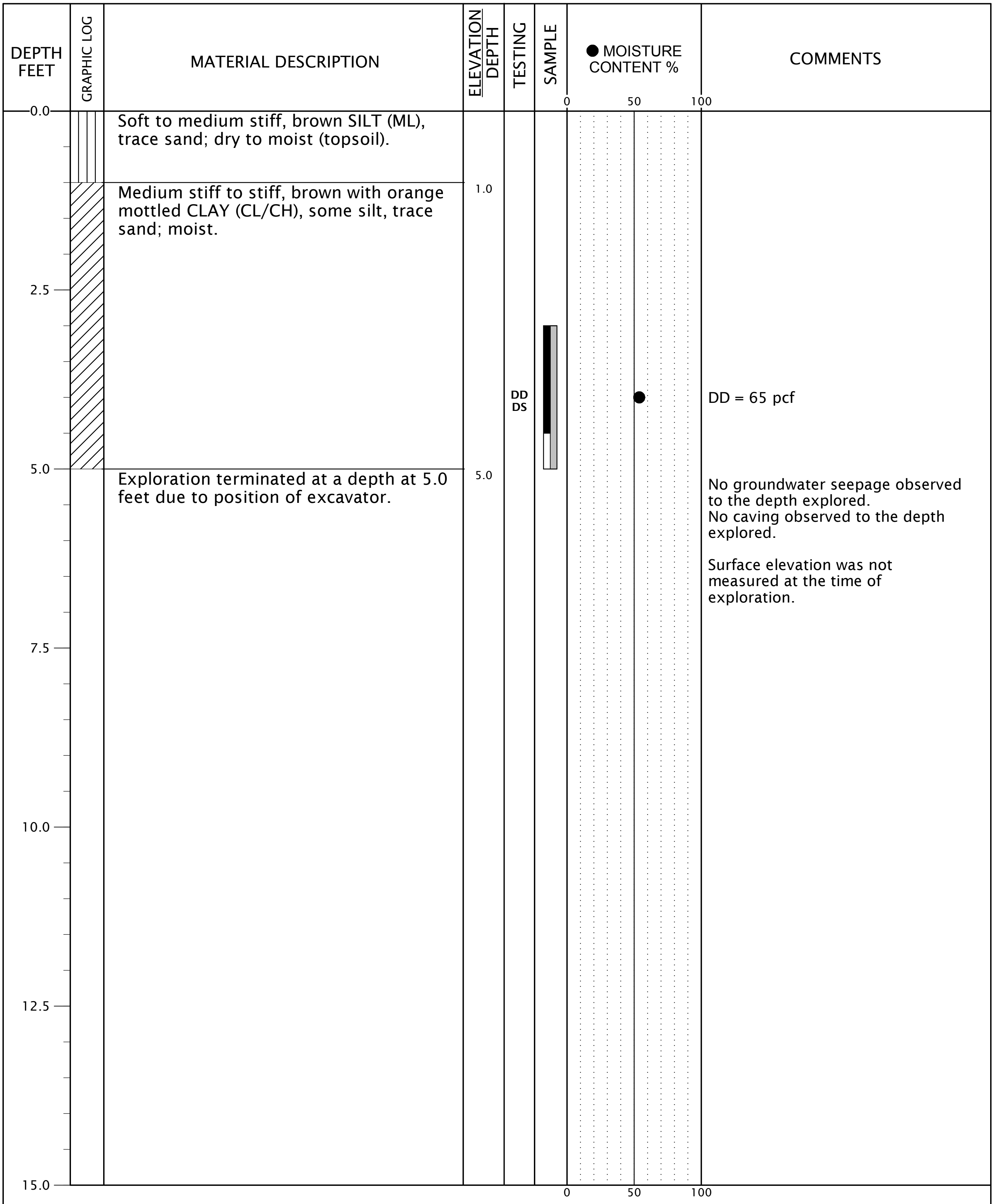
SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-21



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	COMMENTS
0.0	Soft to medium stiff, brown SILT (ML), trace sand; dry to moist (topsoil).			
1.0	Medium stiff to stiff, brown with orange mottled CLAY (CL/CH), some silt, trace sand; moist.	1.0	DD DS	DD = 65 pcf
5.0	Exploration terminated at a depth at 5.0 feet due to position of excavator.	5.0		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.

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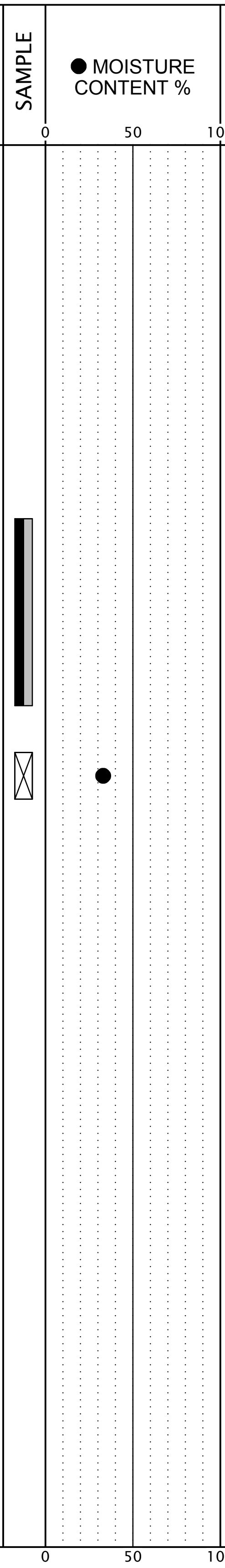
EXCAVATION METHOD: trackhoe (see document text)

TEST PIT TP-17

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-22

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	COMMENTS
0.0	Soft to medium stiff, brown SILT (ML), trace to minor organics, trace sand; moist (topsoil, 8-inch-thick root zone).		
	Medium stiff, light brown with orange mottled SILT (MH), some clay, trace sand; moist.	1.5	
2.5			
5.0	trace gravel at 5.0 feet		
	stiff at 6.0 feet		
7.5			
10.0			
	Exploration completed at a depth of 10.5 feet.	10.5	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			
15.0			



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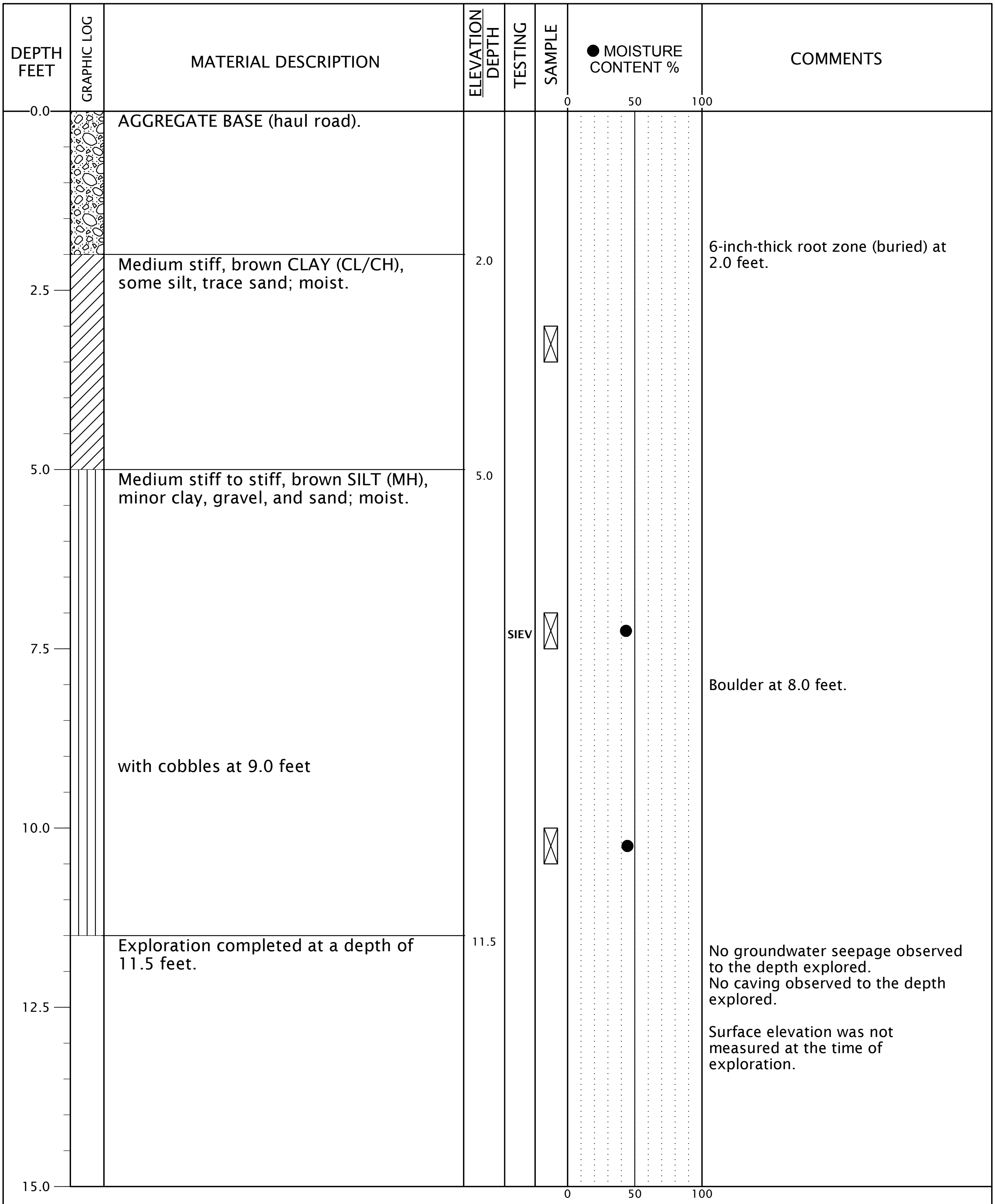
TEST PIT TP-18

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-23



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT %	COMMENTS
0.0	AGGREGATE BASE (haul road).					
	Medium stiff, brown CLAY (CL/CH), some silt, trace sand; moist.	2.0		☒		6-inch-thick root zone (buried) at 2.0 feet.
5.0	Medium stiff to stiff, brown SILT (MH), minor clay, gravel, and sand; moist.	5.0	SIEV	☒	●	Boulder at 8.0 feet.
	with cobbles at 9.0 feet			☒	●	
	Exploration completed at a depth of 11.5 feet.	11.5				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.

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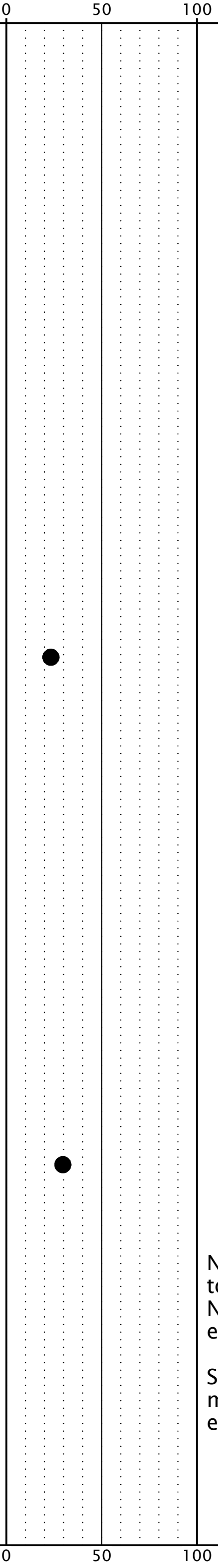
SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-24

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT %	COMMENTS
0.0		Medium stiff to stiff, light brown with orange mottled SILT (MH), some clay, trace sand; dry to moist (4-inch-thick root zone).					
2.5					☒		
		moist at 4.0 feet					
5.0							
					☒	●	
		stiff to very stiff at 7.0 feet					
7.5							
10.0							
					☒	●	
		Exploration completed at a depth of 12.0 feet.	12.0				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							
15.0							



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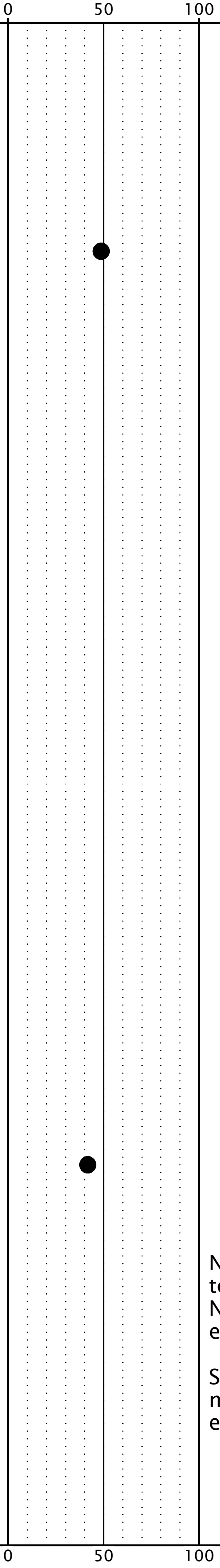
SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-25

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % 0 50 100	COMMENTS
0.0	▥	Soft to medium stiff, dark brown SILT (ML), trace to minor organics, trace sand; dry to moist (topsoil, 4-inch-thick root zone).					
2.5 5.0	▨	Medium stiff, light brown with orange mottled CLAY (CL/CH), some silt, trace sand; dry to moist. moist at 4.0 feet	1.5		☒	●	
7.5	▥	Very stiff, dark brown SILT with gravel and cobbles (MH), minor clay and sand; moist.	5.5				
10.0	▨	Stiff, light brown with orange mottled CLAY (CL/CH), some silt, trace sand; moist. soft to medium stiff, dark gray at 9.0 feet medium stiff, dark gray with light gray spots at 11.0 feet	8.0		☒ ☒	●	
12.5 15.0		Exploration completed at a depth of 12.0 feet.	12.0				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.



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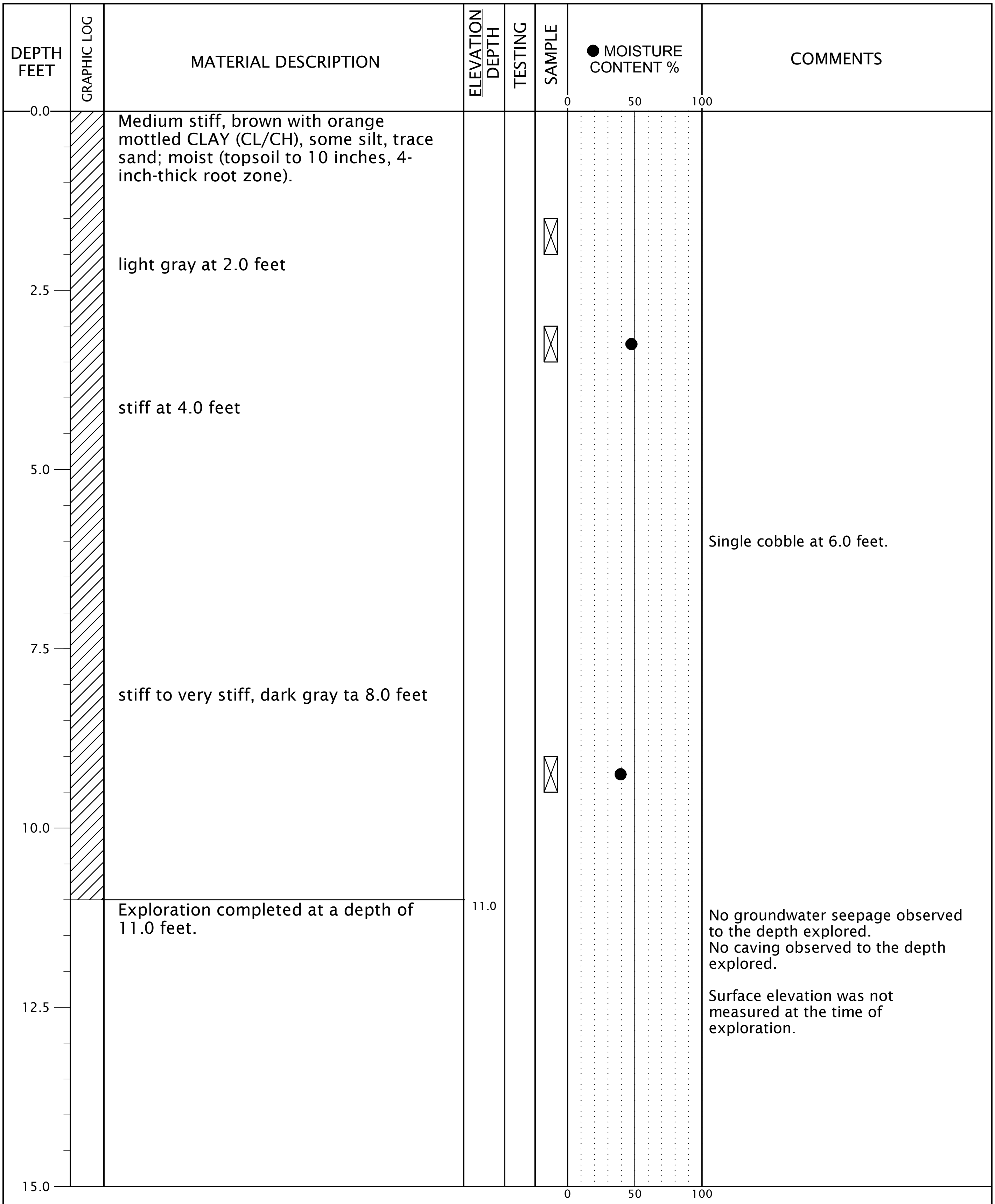
SEASIDESD-1-03-01

NOVEMBER 2017

TEST PIT TP-21

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-26



Medium stiff, brown with orange mottled CLAY (CL/CH), some silt, trace sand; moist (topsoil to 10 inches, 4-inch-thick root zone).

light gray at 2.0 feet

stiff at 4.0 feet

stiff to very stiff, dark gray ta 8.0 feet

Exploration completed at a depth of 11.0 feet.

Single cobble at 6.0 feet.

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.

EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: RSK

COMPLETED: 09/08/17

EXCAVATION METHOD: trackhoe (see document text)

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SEASIDESD-1-03-01

NOVEMBER 2017

TEST PIT TP-22

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-27

TEST PIT LOG - 1 PER PAGE SEASIDESD-1-03-01-B1_5-TP1_22,26.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:RC:KT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Medium stiff, dark brown SILT (ML), minor sand; dry to moist (topsoil).					
		Medium stiff, light brown with orange mottled CLAY (CH), some silt, minor clay, trace sand; moist (colluvium).	2.0				
2.5							
5.0							
7.5							
10.0							
		Exploration completed at a depth of 12.0 feet.	12.0				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							
15.0						0 50 100	

EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: CMC

COMPLETED: 09/13/17

EXCAVATION METHOD: trackhoe (see document text)

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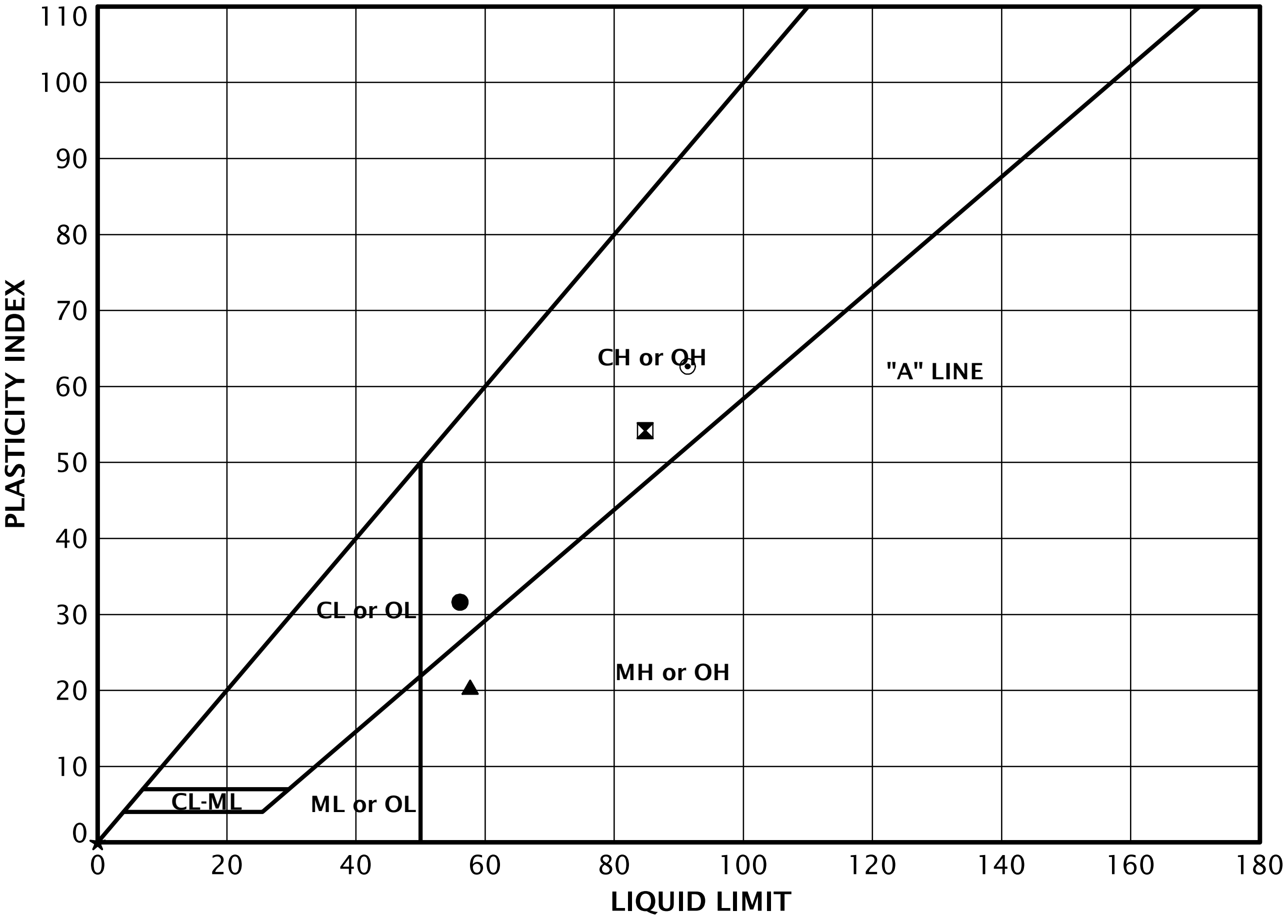
NOVEMBER 2017

TEST PIT TP-26

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-28



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-1	15.0	37	56	24	32
⧗	TP-5	7.5	42	85	31	54
▲	TP-6	7.0	74	58	37	21
★	TP-8	9.0	31	NP	NP	NP
⊙	TP-26	10.0		91	29	62

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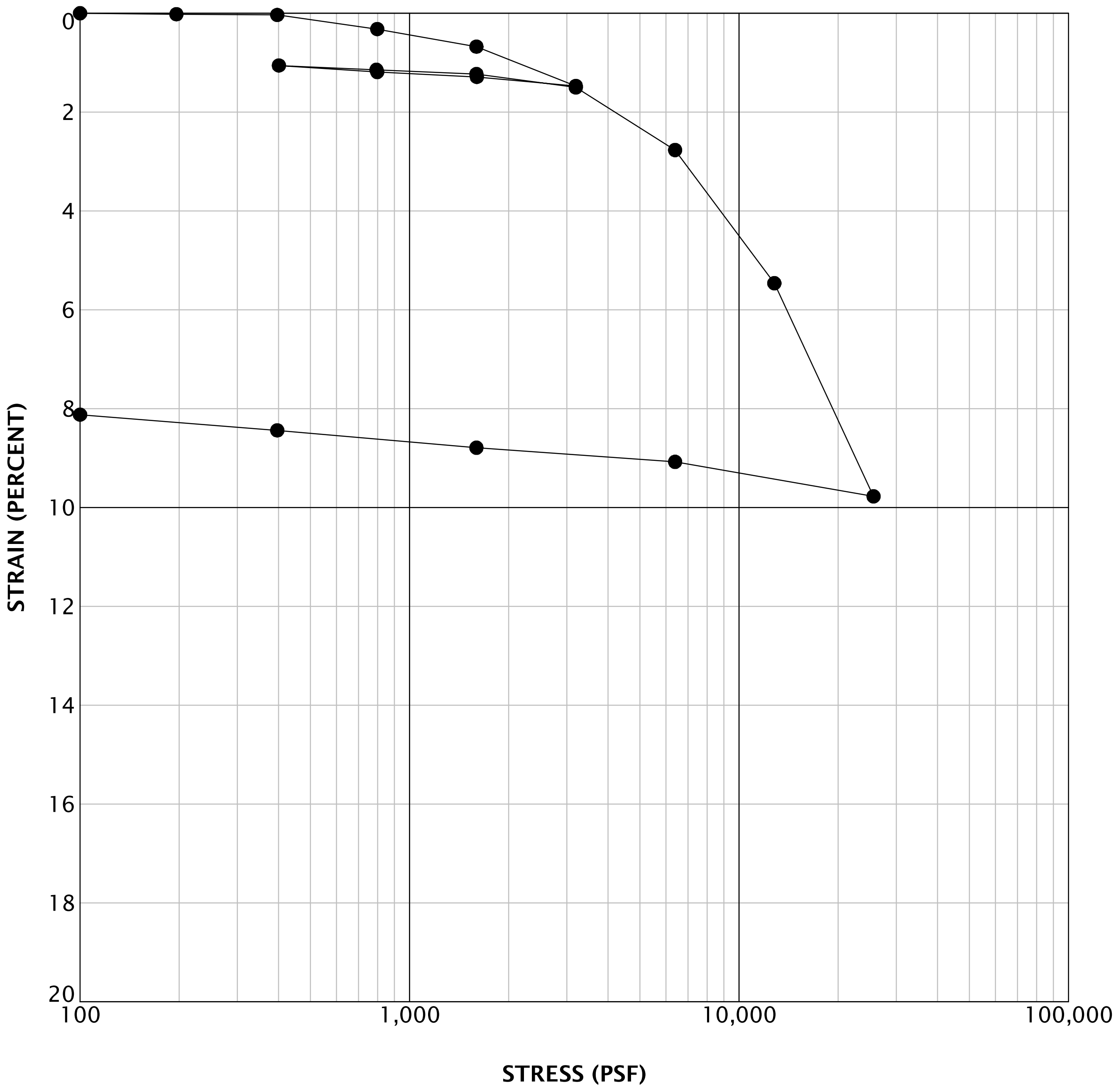
ATTERBERG LIMITS TEST RESULTS

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-29



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)
●	TP-7	5.0	46	73

CONSOL_STRAIN_100K SEASIDESD-1-03-01-B1_5-TP1_22,26.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:KT

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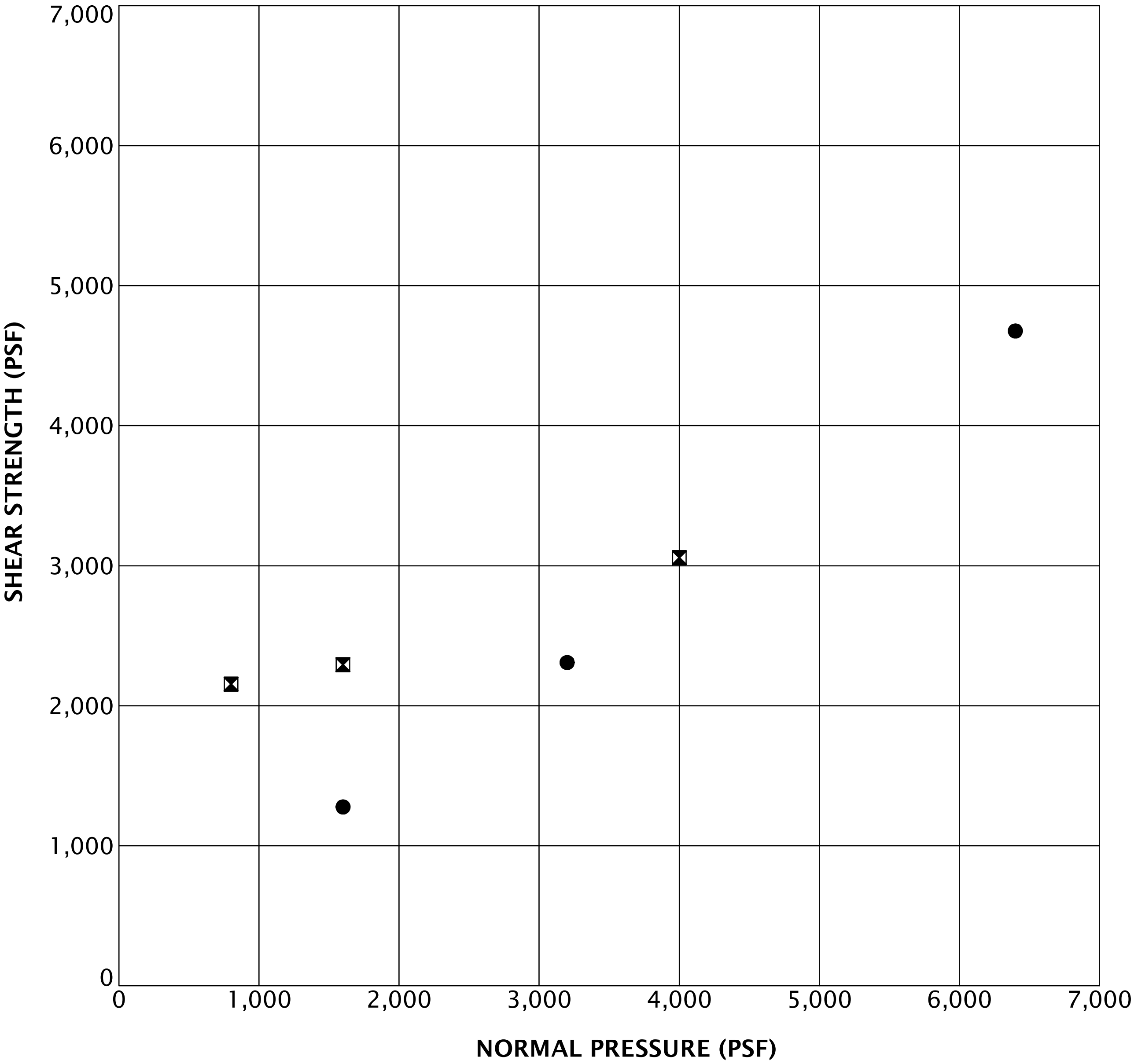
CONSOLIDATION TEST RESULTS

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-30



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SOAKED
●	B-3	37.0	36	84	YES
⊠	TP-17	3.0	52	68	YES

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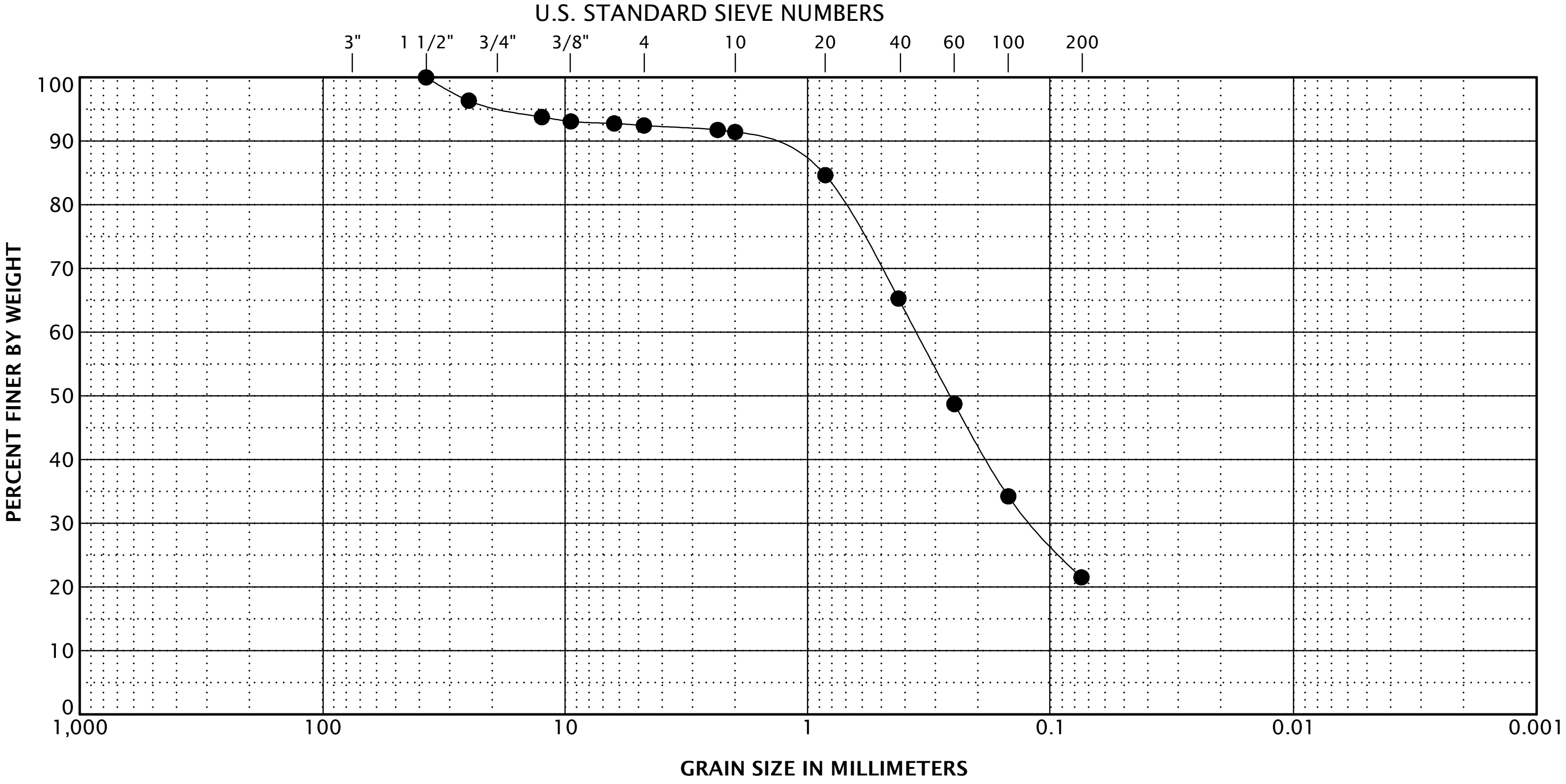
DIRECT SHEAR TEST RESULTS

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-31



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY

KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	D60	D50	D30	D10	D5	GRAVEL (PERCENT)	SAND (PERCENT)	SILT (PERCENT)	CLAY (PERCENT)
●	TP-19	7.0	43	0.36	0.26	0.12			8	71	22	

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NOVEMBER 2017

GRAIN-SIZE TEST RESULTS

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-32

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	5.0		29							
B-1	10.0		37							
B-1	15.0		37					56	24	32
B-1	20.0		30							
B-1	25.0		36							
B-1	30.0		37							
B-1	35.0		33							
B-1	40.0		34							
B-1	50.0		27							
B-1	60.0		27							
B-1	70.0		34							
B-1	80.0		28							
B-2	5.0		53							
B-2	10.0		39							
B-2	15.0		47							
B-2	20.0		46							
B-2	25.0		41							
B-2	30.0		16							
B-2	35.0		34							
B-2	40.0		29							
B-2	50.0		27							
B-2	60.0		40							
B-2	65.0		46							
B-2	70.0		45							
B-2	80.0		26							
B-3	5.0		37							
B-3	15.0		36							

LAB SUMMARY SEASIDESD-1-03-01-B1_5-TP1_22,26.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:KT

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SUMMARY OF LABORATORY DATA

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-33

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	20.0		33							
B-3	30.0		38							
B-3	35.0		70							
B-3	37.0		36	83						
B-3	39.0		51							
B-3	45.0		57							
B-3	50.0		36							
B-4	12.0		58							
B-4	15.0		61							
B-4	20.0		61							
B-4	25.0		43							
B-4	35.0		38							
B-5	5.0		55							
B-5	10.0		51							
B-5	15.0		43							
B-5	20.0		27							
B-5	30.0		49							
B-5	40.0		38							
TP-1	1.5		49							
TP-1	8.0		39							
TP-2	2.5		43							
TP-2	10.0		35							
TP-3	2.5		51							
TP-3	6.0		47							
TP-4	2.0		65							
TP-4	9.0		52							
TP-5	3.5		47							

LAB SUMMARY SEASIDESD-1-03-01-B1_5-TP1_22,26.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:KT

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SEASIDESD-1-03-01

NOVEMBER 2017

SUMMARY OF LABORATORY DATA
(continued)

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-33

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
TP-5	7.5		42					85	31	54
TP-6	2.5		60							
TP-6	7.0		74					58	37	21
TP-7	3.0		62							
TP-7	5.0		46	73						
TP-8	5.0		55							
TP-8	9.0		31					NP	NP	NP
TP-9	2.5		46							
TP-9	8.0		29							
TP-10	2.5		39							
TP-10	10.0		50							
TP-11	3.0		66							
TP-11	8.0		29							
TP-12	10.0		49							
TP-14	3.0		32							
TP-15	2.5		45							
TP-15	9.0		38							
TP-16	2.5		68							
TP-16	8.0		47							
TP-17	3.0		54	65						
TP-18	6.5		33							
TP-19	7.0		43		8	71	22			
TP-19	10.0		45							
TP-20	6.0		23							
TP-20	11.0		30							
TP-21	2.0		49							
TP-21	11.0		42							

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SEASIDESD-1-03-01

NOVEMBER 2017

SUMMARY OF LABORATORY DATA
(continued)

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-33

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
TP-22	3.0		47							
TP-22	9.0		39							
TP-26	10.0							91	29	62

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NOVEMBER 2017

SUMMARY OF LABORATORY DATA
(continued)

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-33

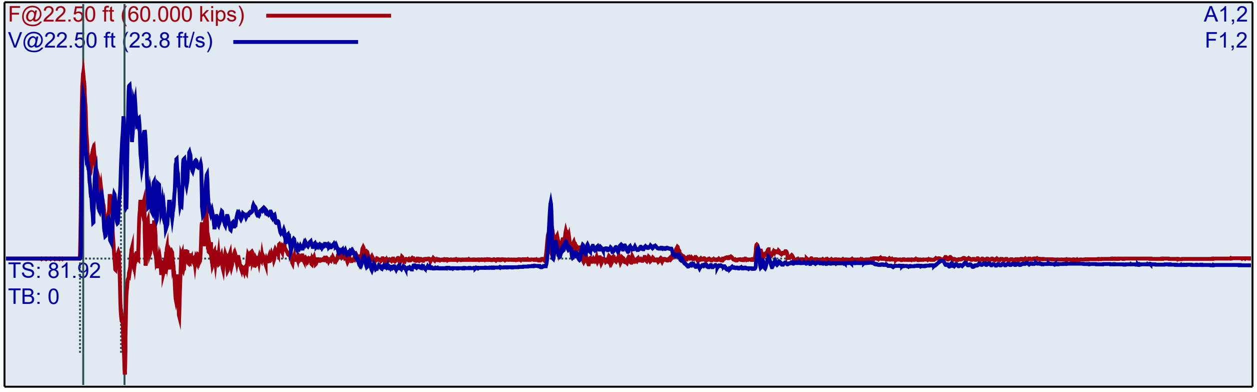
WSSC-8-01
JDT

RIG 7 - SERIAL NO. 383942
Test date: 12/28/2016

AR: 1.41 in^2
LE: 22.50 ft
WS: 16807.9 ft/s

SP: 0.492 k/ft3
EM: 30000 ksi

Depth: (15.50 - 16.50 ft], displaying BN: 9



LP: Length of Penetration
FMX: Maximum Force
VMX: Maximum Velocity

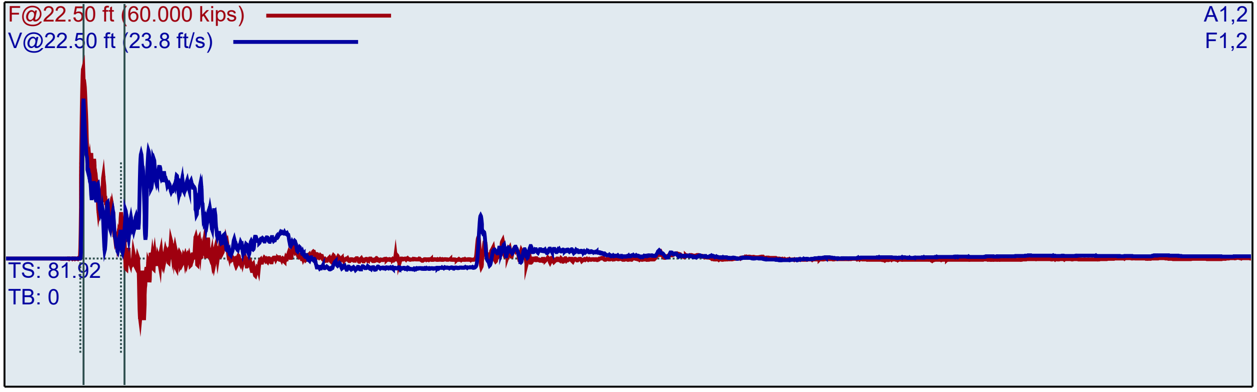
BPM: Blows/Minute
EMX: Maximum Energy
ETR: Energy Transfer Ratio - Rated

BL#	BC /6"	LP ft	FMX kips	VMX ft/s	BPM bpm	EMX ft-lb	ETR (%)
1	3	15.67	44.305	15.6	1.9	313.0	89.4
2	3	15.83	44.589	17.0	55.3	316.3	90.4
3	3	16.00	44.472	16.5	55.2	307.7	87.9
4	8	16.06	44.492	16.8	54.7	311.3	88.9
5	8	16.13	42.703	16.4	54.9	309.6	88.5
6	8	16.19	44.398	16.9	54.6	310.9	88.8
7	8	16.25	43.695	16.3	54.8	312.9	89.4
8	8	16.31	43.700	16.1	54.9	311.4	89.0
9	8	16.38	43.409	16.1	55.0	310.4	88.7
10	8	16.44	43.247	15.3	54.8	307.0	87.7
11	8	16.50	19.792	6.7	80.3	59.1	16.9
	Average	16.16	41.709	15.4	52.4	288.2	82.3
	Std Dev	0.24	6.955	2.8	17.5	72.5	20.7
	Maximum	16.50	44.589	17.0	80.3	316.3	90.4
	Minimum	15.67	19.792	6.7	1.9	59.1	16.9

N-value: 11

Sample Interval Time: 10.56 seconds.

Depth: (18.00 - 19.00 ft], displaying BN: 21

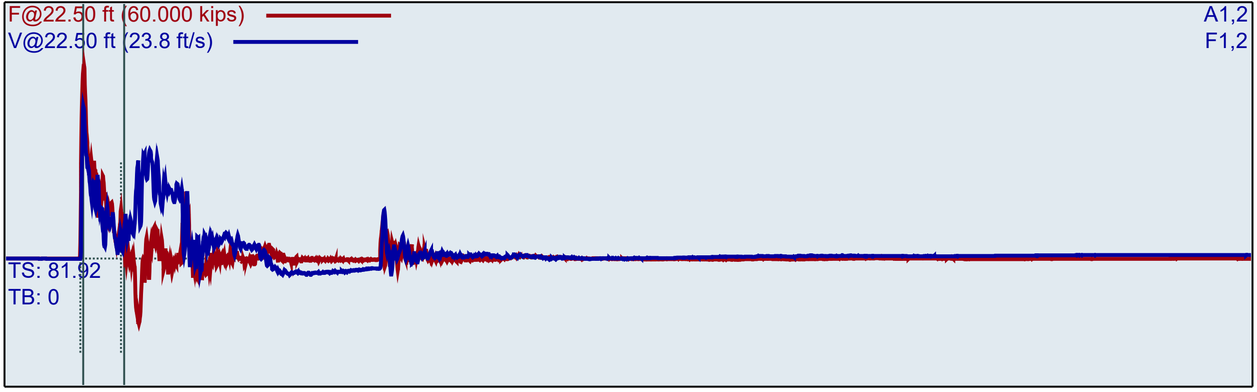


BL#	BC /6"	LP ft	FMX kips	VMX ft/s	BPM bpm	EMX ft-lb	ETR (%)
12	4	18.13	43.583	14.3	54.5	310.6	88.8
13	4	18.25	43.570	14.1	54.5	312.4	89.3
14	4	18.38	43.975	14.3	54.2	310.8	88.8
15	4	18.50	42.792	15.5	54.6	312.5	89.3
16	8	18.56	43.422	15.0	54.5	317.1	90.6
17	8	18.63	43.768	14.6	54.1	316.0	90.3
18	8	18.69	43.318	14.5	54.4	315.8	90.2
19	8	18.75	42.657	14.0	54.4	312.9	89.4
20	8	18.81	44.231	14.8	54.3	314.4	89.8
21	8	18.88	44.488	14.9	54.3	316.3	90.4
22	8	18.94	44.711	15.0	54.4	313.9	89.7
23	8	19.00	43.832	14.4	54.4	311.8	89.1
	Average	18.63	43.696	14.6	54.4	313.7	89.6
	Std Dev	0.26	0.591	0.4	0.1	2.1	0.6
	Maximum	19.00	44.711	15.5	54.6	317.1	90.6
	Minimum	18.13	42.657	14.0	54.1	310.6	88.8

N-value: 12

Sample Interval Time: 12.12 seconds.

Depth: (20.50 - 21.50 ft], displaying BN: 39

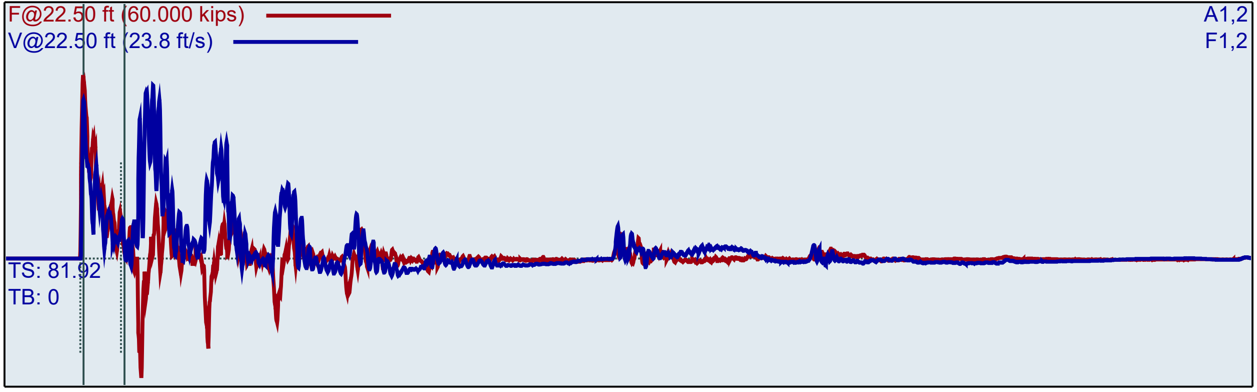


BL#	BC /6"	LP ft	FMX kips	VMX ft/s	BPM bpm	EMX ft-lb	ETR (%)
24	8	20.56	44.057	14.8	54.5	317.7	90.8
25	8	20.63	43.219	16.8	54.2	306.1	87.5
26	8	20.69	44.090	14.1	54.2	313.8	89.7
27	8	20.75	44.898	14.9	54.4	316.2	90.3
28	8	20.81	44.836	16.6	54.2	311.3	88.9
29	8	20.88	44.302	16.5	54.4	314.1	89.8
30	8	20.94	44.345	15.1	54.3	314.5	89.8
31	8	21.00	45.057	15.8	54.5	311.2	88.9
32	10	21.05	44.132	15.3	54.2	311.7	89.1
33	10	21.10	43.899	17.1	54.3	309.6	88.5
34	10	21.15	45.254	14.7	54.1	313.8	89.7
35	10	21.20	45.251	14.4	54.5	317.2	90.6
36	10	21.25	43.714	15.5	54.0	311.9	89.1
37	10	21.30	42.554	17.2	54.3	312.7	89.4
38	10	21.35	42.369	16.6	54.2	310.0	88.6
39	10	21.40	45.048	14.2	54.1	314.9	90.0
40	10	21.45	42.990	16.5	54.3	309.4	88.4
41	10	21.50	44.257	15.9	54.2	317.1	90.6
	Average	21.06	44.126	15.7	54.3	313.0	89.4
	Std Dev	0.28	0.861	1.0	0.1	3.0	0.9
	Maximum	21.50	45.254	17.2	54.5	317.7	90.8
	Minimum	20.56	42.369	14.1	54.0	306.1	87.5

N-value: 18

Sample Interval Time: 18.77 seconds.

Depth: (23.00 - 24.00 ft], displaying BN: 56



BL#	BC /6"	LP ft	FMX kips	VMX ft/s	BPM bpm	EMX ft-lb	ETR (%)
42	8	23.06	42.016	15.5	54.7	315.2	90.0
43	8	23.13	42.207	15.6	54.8	313.2	89.5
44	8	23.19	42.444	15.6	54.7	319.6	91.3
45	8	23.25	42.717	15.5	55.1	316.9	90.5
46	8	23.31	42.337	15.5	54.3	317.4	90.7
47	8	23.38	42.016	15.4	54.9	315.0	90.0
48	8	23.44	42.802	15.9	54.8	317.9	90.8
49	8	23.50	41.630	15.5	54.7	314.4	89.8
50	9	23.56	42.341	15.8	54.6	316.4	90.4
51	9	23.61	41.710	15.9	54.7	316.5	90.4
52	9	23.67	42.185	16.0	54.7	319.2	91.2
53	9	23.72	41.073	15.5	54.6	309.2	88.3
54	9	23.78	41.662	15.5	54.6	313.4	89.6
55	9	23.83	40.943	15.4	54.7	311.3	88.9
56	9	23.89	42.559	16.1	54.4	317.5	90.7
57	9	23.94	41.035	15.5	54.4	311.8	89.1
58	9	24.00	41.007	14.8	54.7	309.5	88.4
	Average	23.54	41.923	15.6	54.7	315.0	90.0
	Std Dev	0.29	0.599	0.3	0.2	3.1	0.9
	Maximum	24.00	42.802	16.1	55.1	319.6	91.3
	Minimum	23.06	40.943	14.8	54.3	309.2	88.3

N-value: 17

Sample Interval Time: 17.54 seconds.

Summary of SPT Test Results

Project: WSSC-8-01, Test Date: 12/28/2016

LP: Length of Penetration
FMX: Maximum Force
VMX: Maximum Velocity

BPM: Blows/Minute
EMX: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Blows Applied /6"	**N Value**	**N60 Value**	Average LP ft	Average FMX kips	Average VMX ft/s	Average BPM bpm	Average EMX ft-lb	Average ETR (%)
22.50	**3-8**	**8**	**11**	16.16	41.709	15.4	52.4	288.2	82.3
22.50	**4-8**	**8**	**11**	18.63	43.696	14.6	54.4	313.7	89.6
22.50	**8-10**	**10**	**14**	21.06	44.126	15.7	54.3	313.0	89.4
22.50	**8-9**	**9**	**13**	23.54	41.923	15.6	54.7	315.0	90.0
			Overall Average Values:	20.35	42.933	15.4	54.1	309.0	88.3 ←
			Standard Deviation:	2.68	3.275	1.4	7.7	33.2	9.5
			Overall Maximum Value:	24.00	45.254	17.2	80.3	319.6	91.3
			Overall Minimum Value:	15.67	19.792	6.7	1.9	59.1	16.9

Average Energy Transfer Ratio = 88.3%
Energy Correction Factor = 1.47

APPENDIX B

APPENDIX B

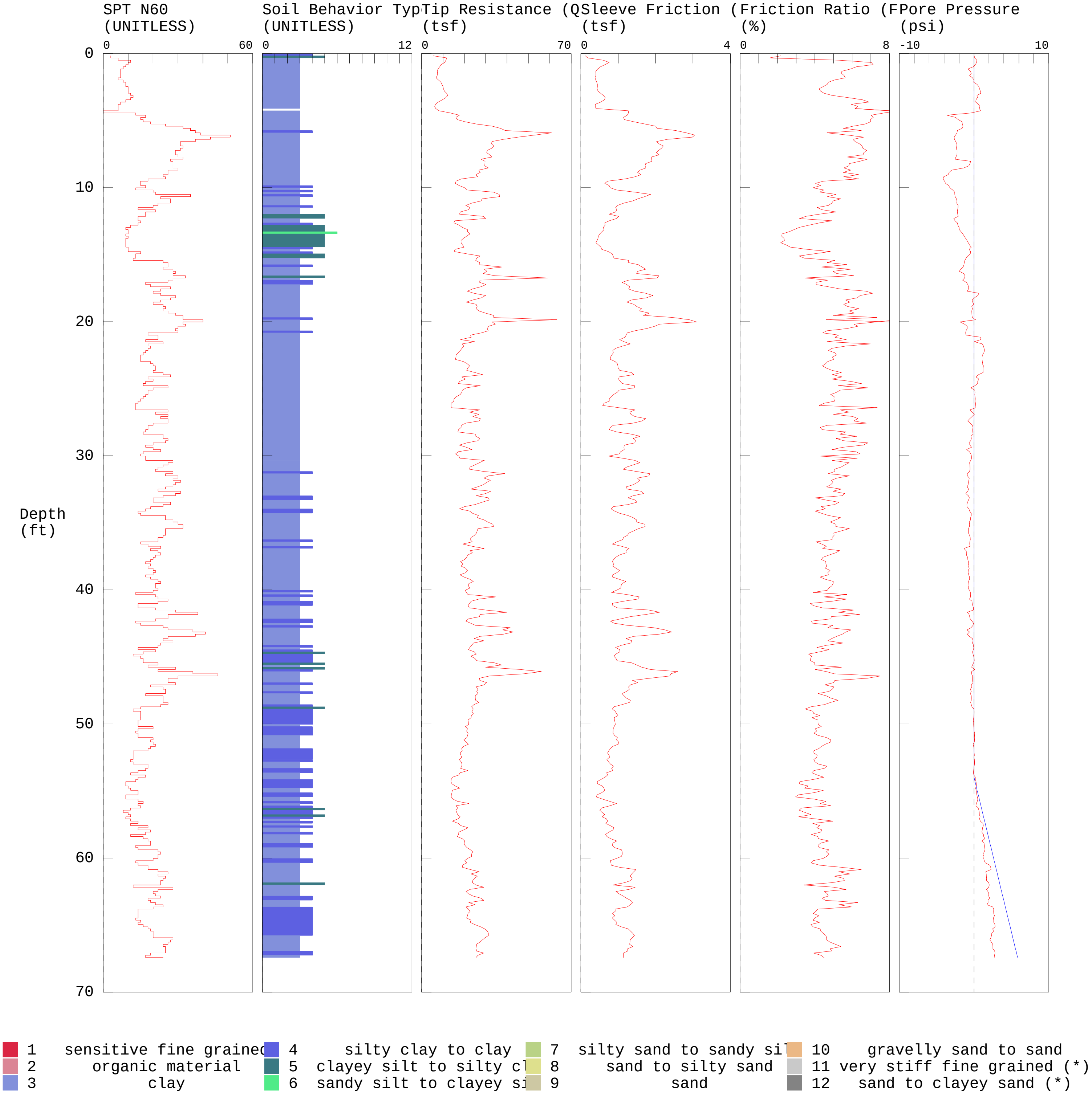
CONE PENETROMETER TESTING

Six CPT probes (CPT-2 through CPT-7a) were advanced to depths ranging between 44.3 and 77.3 feet BGS. Figure 2 shows the locations of the CPT probes relative to existing site features. The CPTs were performed in general accordance with ASTM D 5778 by Oregon Geotechnical Explorations, Inc. of Keizer, Oregon, on September 13 through 15, 2017.

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 2-meter intervals in CPT-3. This appendix presents the results of the CPT completed for this project.

GeoDesign / CPT-2 / Seaside Heights School

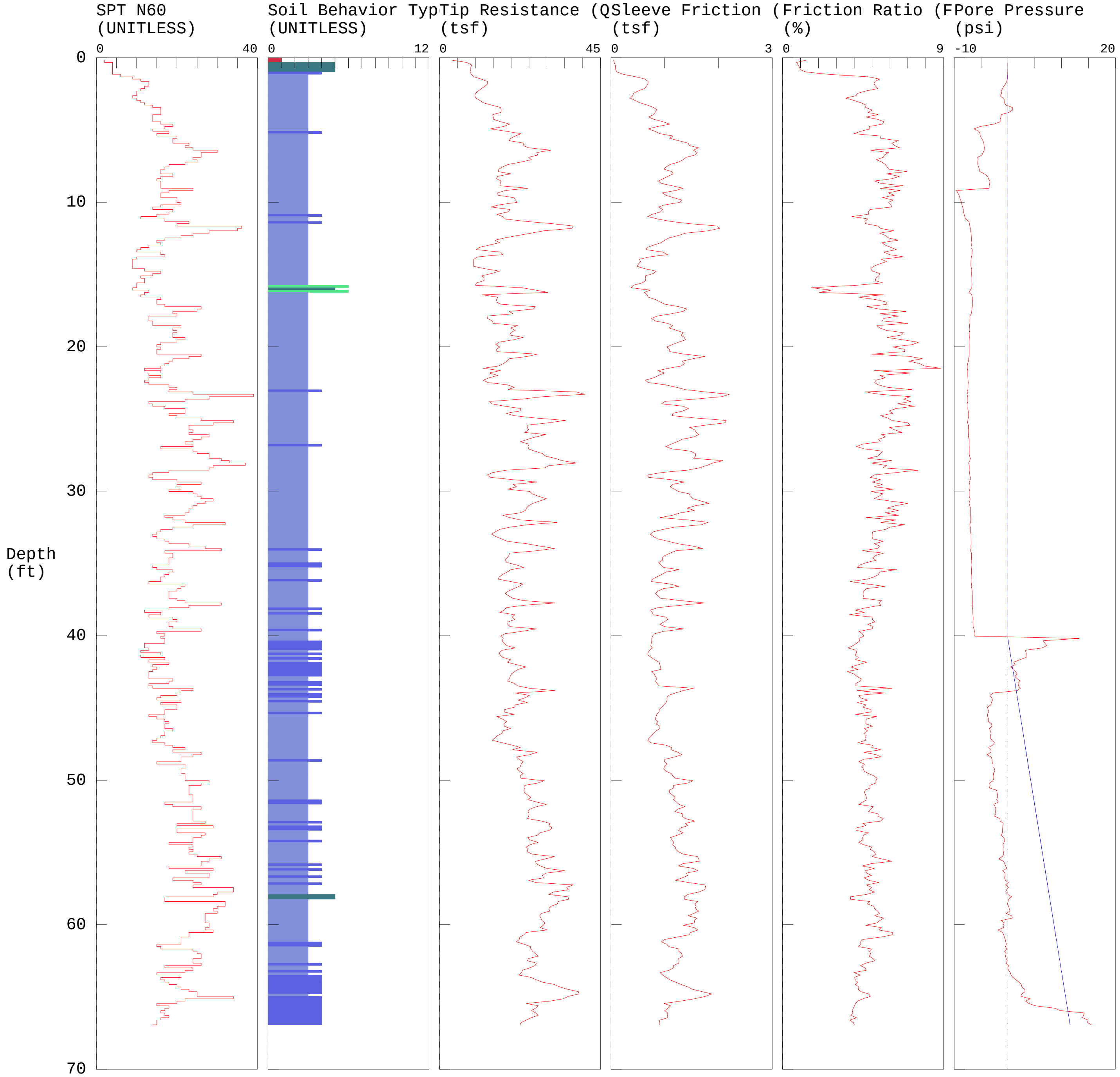
OPERATOR: OGE BB
CONE ID: DDG1415
HOLE NUMBER: CPT-2
TEST DATE: 9/14/2017 9:25:05 AM
TOTAL DEPTH: 67.421 ft



*SBT/SPT CORRELATION: UBC-1983

GeoDesign / CPT-3 / Seaside Heights School

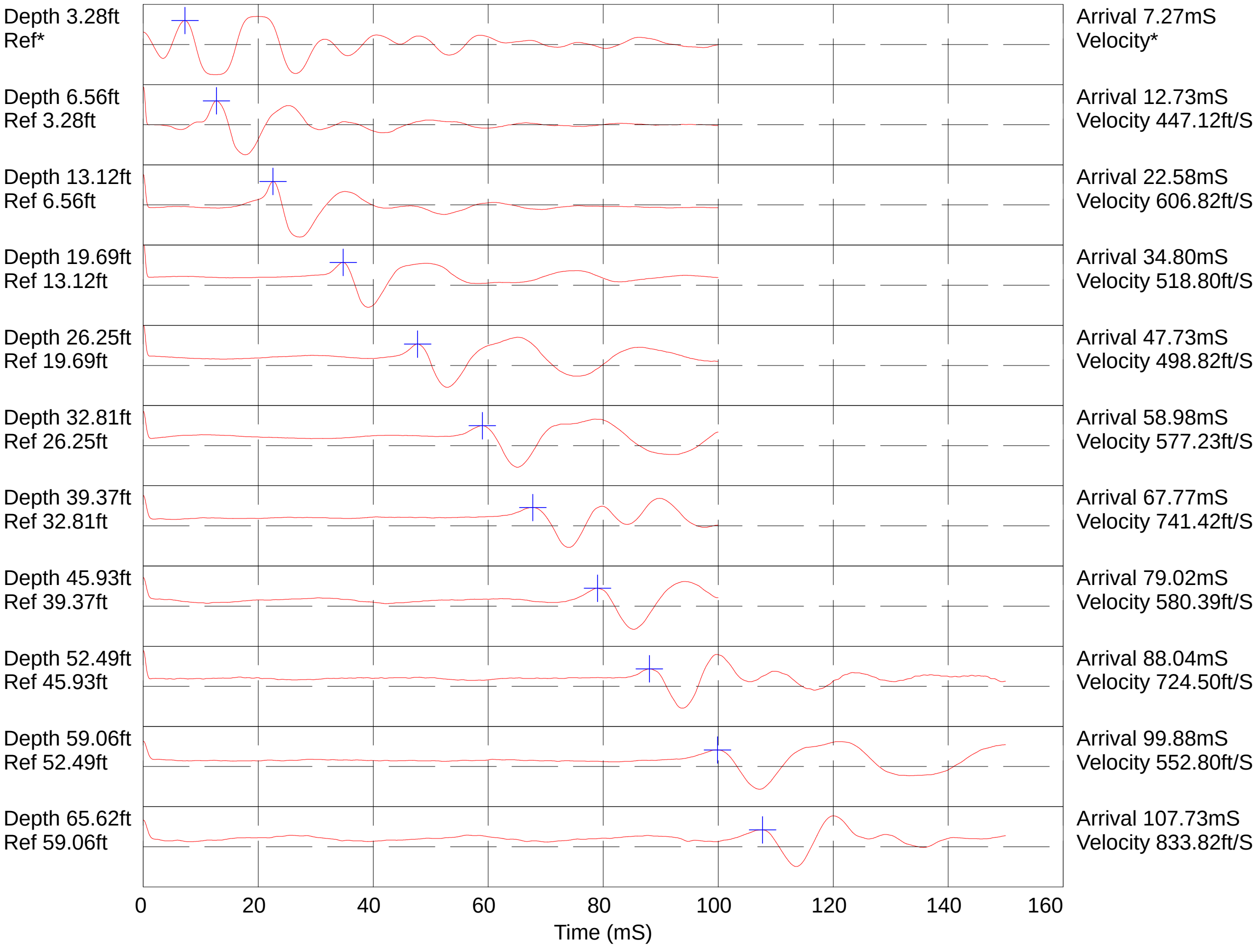
OPERATOR: OGE BB
CONE ID: DDG1415
HOLE NUMBER: CPT-3
TEST DATE: 9/13/2017 11:06:46 AM
TOTAL DEPTH: 66.929 ft



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

*SBT/SPT CORRELATION: UBC-1983

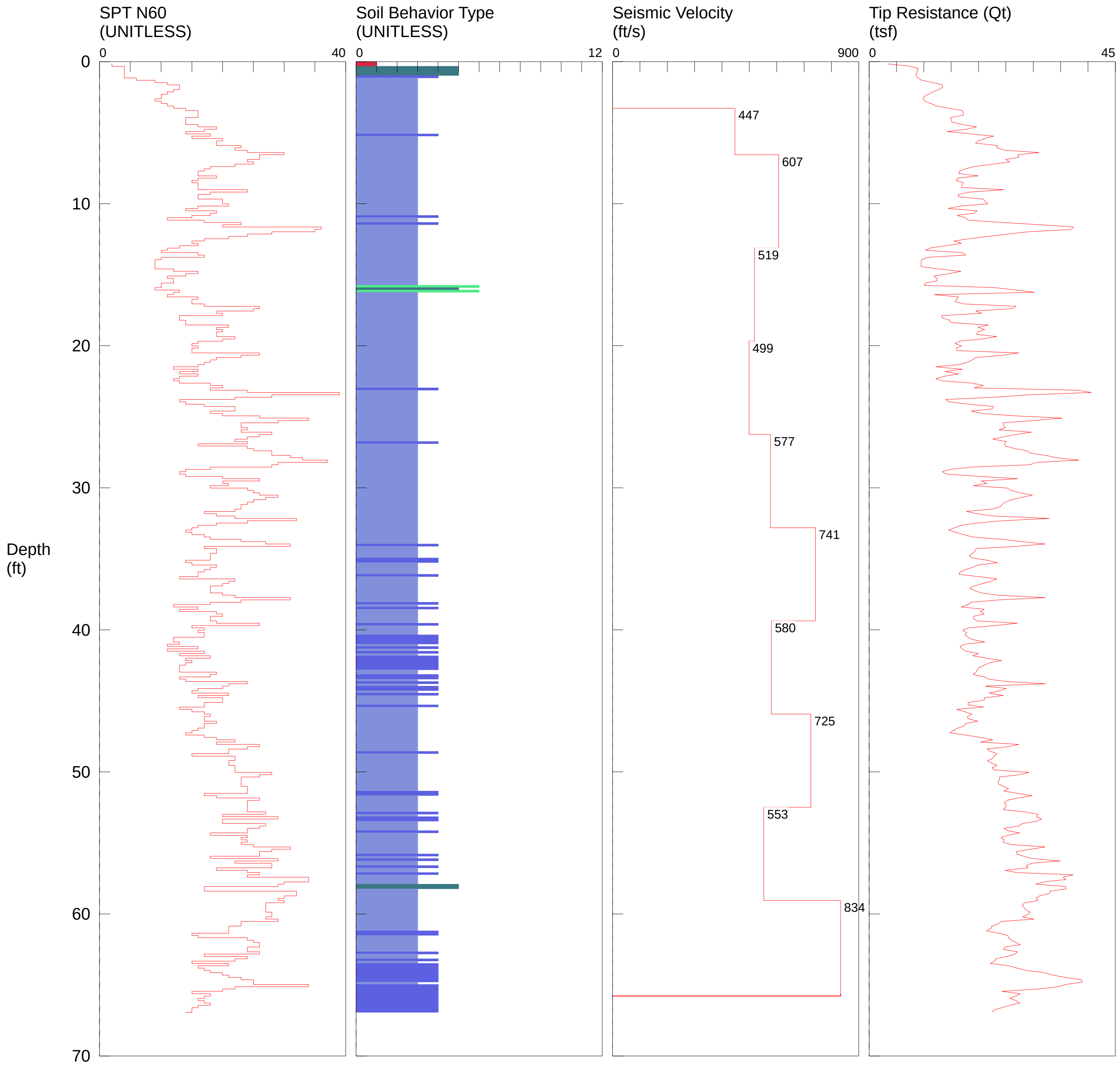
COMMENT: 17142 / GeoDesign / CPT-3 / Seaside SD



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

GeoDesign / CPT-3 / Seaside Heights School

OPERATOR: OGE BB
CONE ID: DDG1415
HOLE NUMBER: CPT-3
TEST DATE: 9/13/2017 11:06:46 AM
TOTAL DEPTH: 66.929 ft

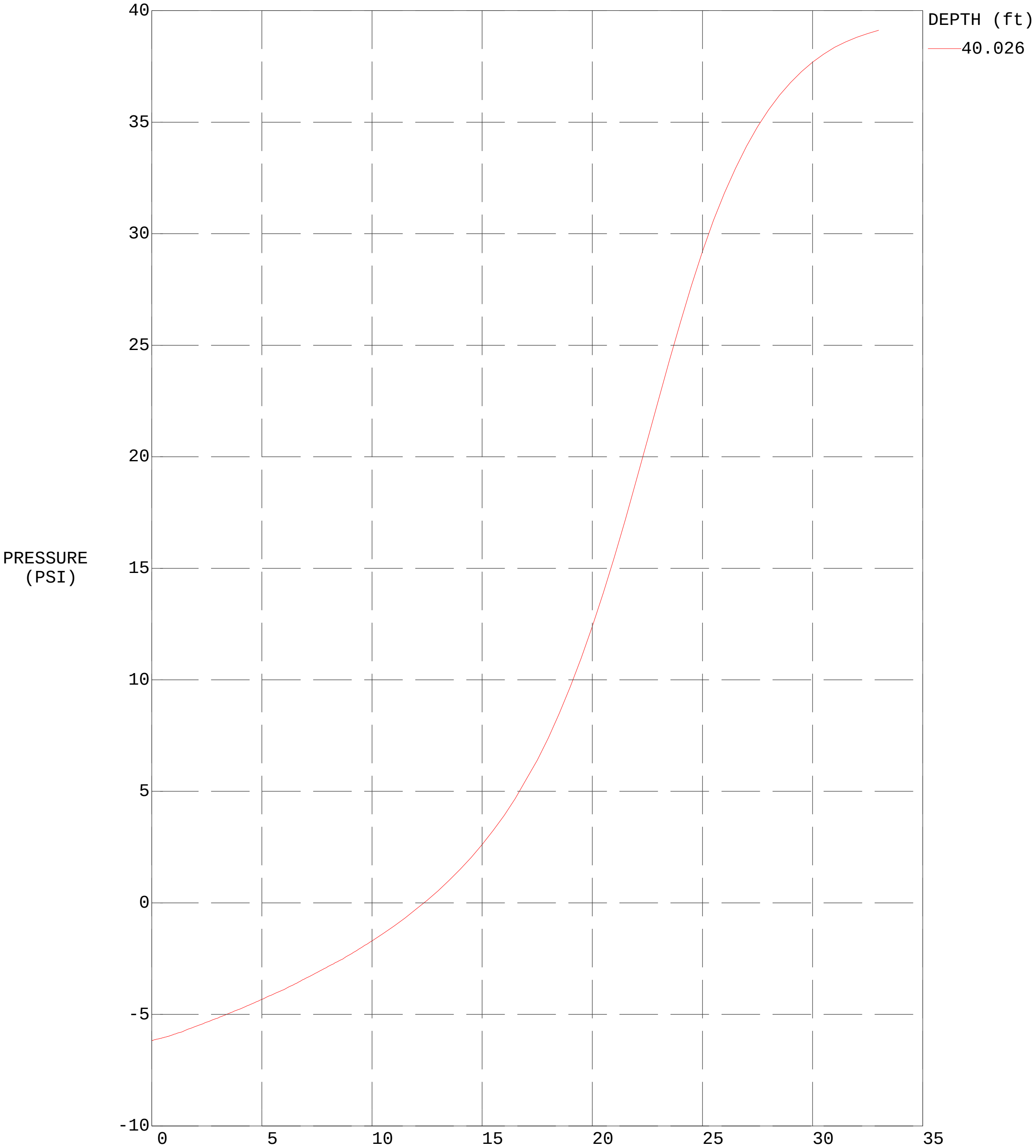


■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay
■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand
■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: 17142 / GeoDesign / CPT-3 / Seaside SD

TEST DATE: 9/13/2017 11:06:46 AM

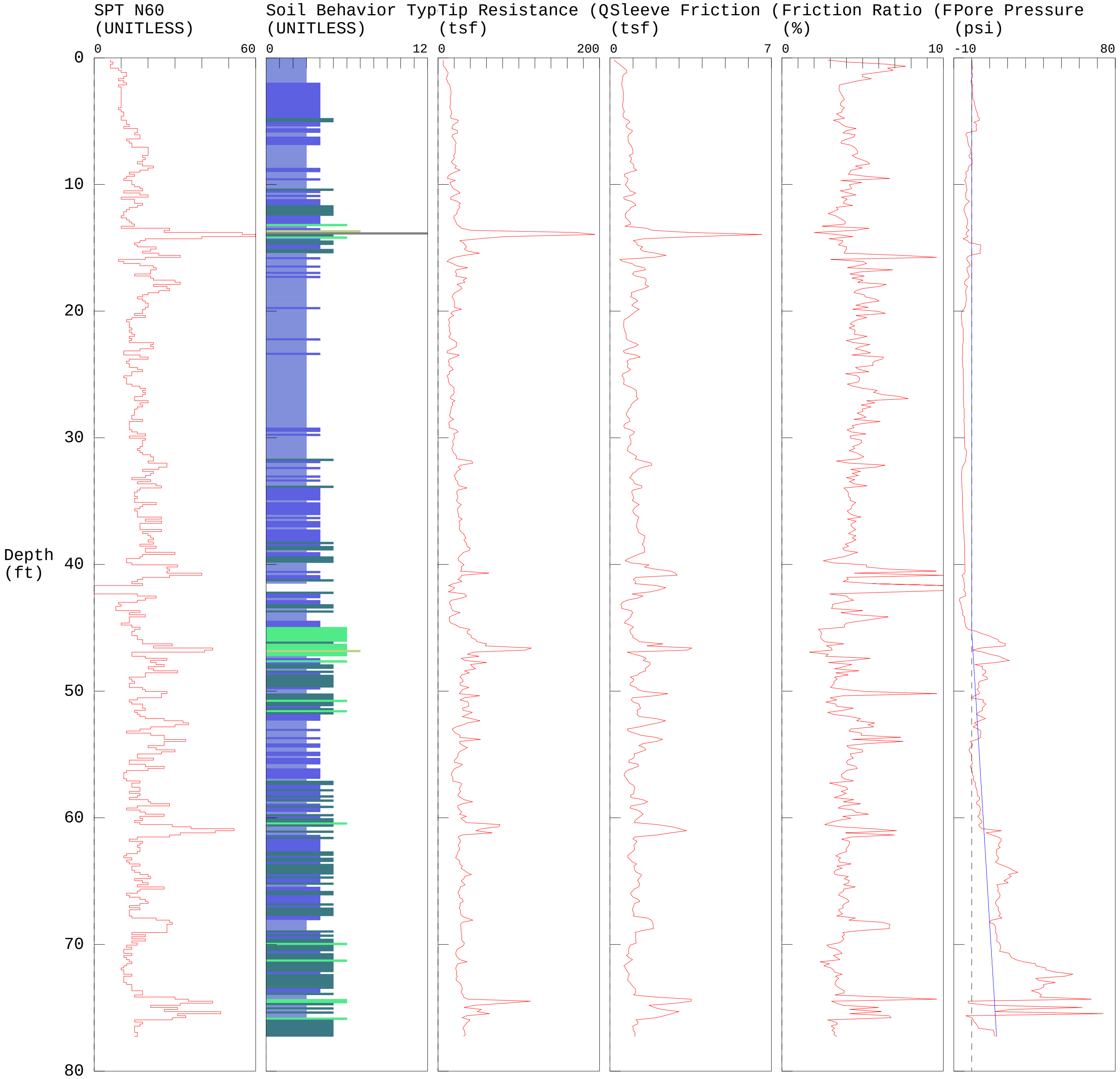


MAXIMUM PRESSURE = 39.121 (PSI)ME: (MINUTES)

HYDROSTATIC PRESSURE = 0.0 (PSI), WATER TABLE: 40.20 ft

GeoDesign / CPT-4 / Seaside Heights School

OPERATOR: OGE BB
CONE ID: DDG1415
HOLE NUMBER: CPT-4
TEST DATE: 9/14/2017 7:17:09 AM
TOTAL DEPTH: 77.264 ft

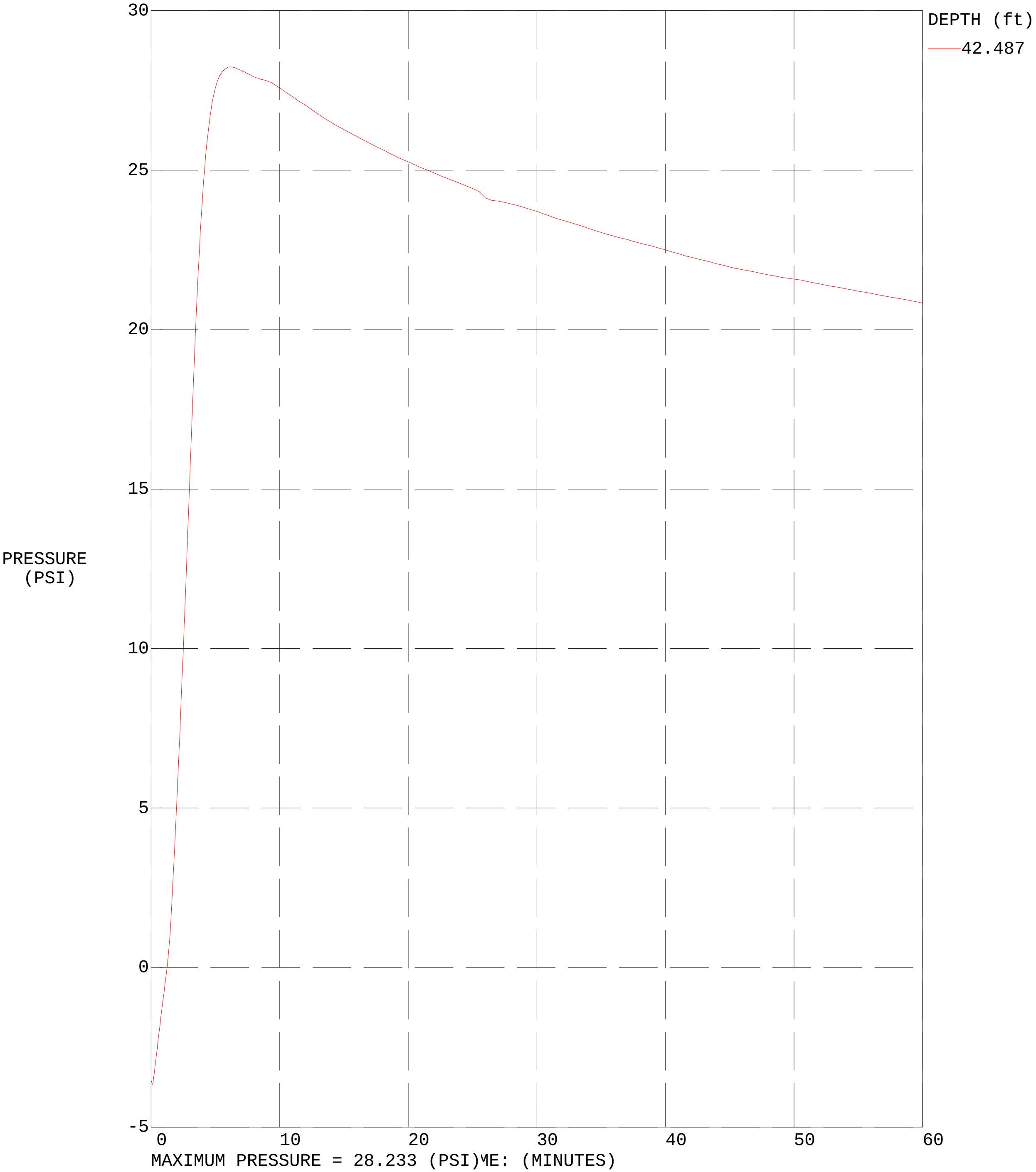


■ 1 sensitive fine grainec
■ 2 organic material
■ 3 clay
■ 4 silty clay to clay
■ 5 clayey silt to silty cl
■ 6 sandy silt to clayey si
■ 7 silty sand to sandy sil
■ 8 sand to silty sand
■ 9 sand
■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: 17142 / GeoDesign / CPT-4 / Seaside SD

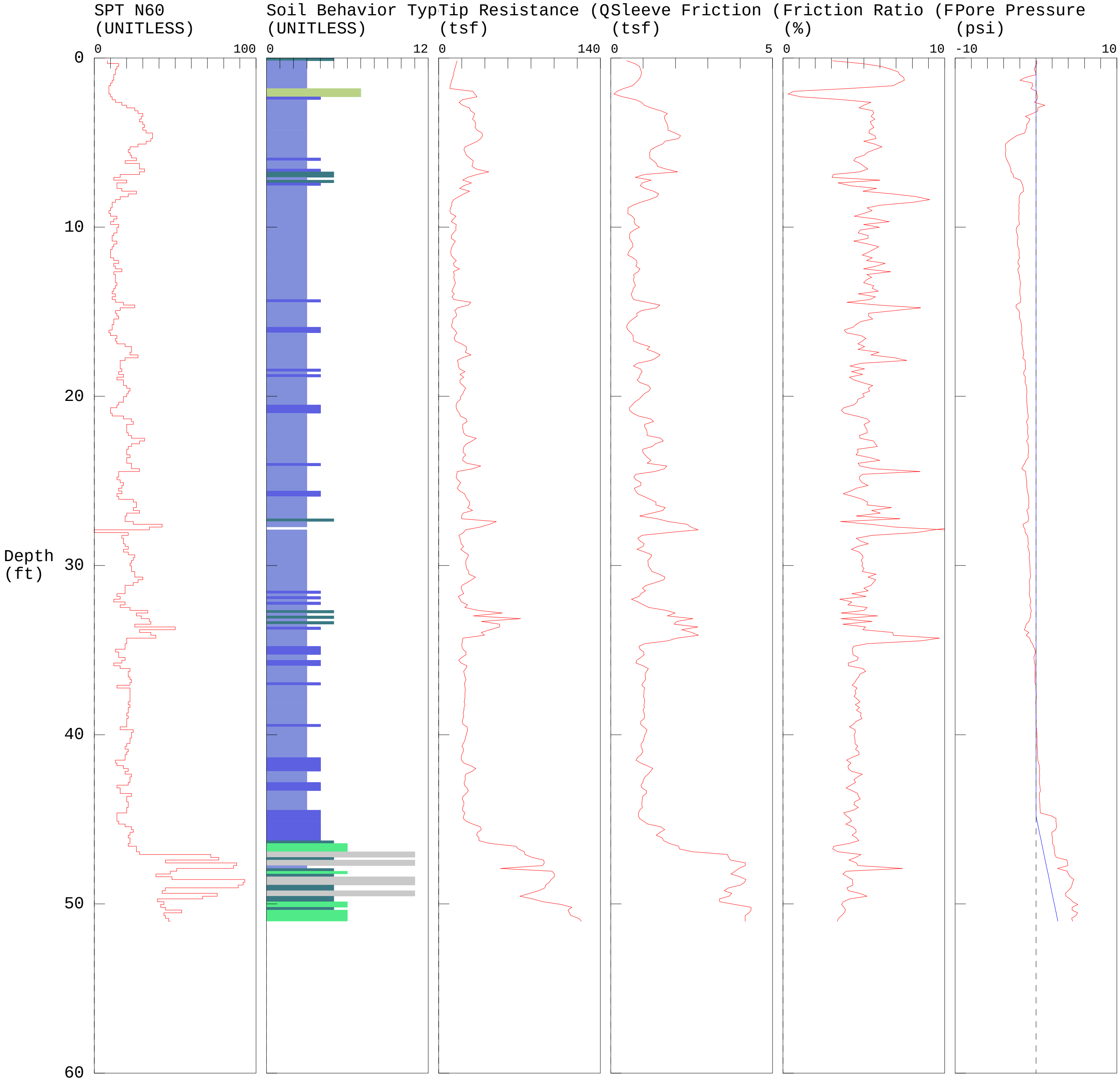
TEST DATE: 9/14/2017 7:17:09 AM



MAXIMUM PRESSURE = 28.233 (PSI)ᴹE: (MINUTES)
HYDROSTATIC PRESSURE = 0.0 (PSI), WATER TABLE: 45.26 ft

GeoDesign / CPT-5 / Seaside Heights School

OPERATOR: OGE BB
CONE ID: DDG1415
HOLE NUMBER: CPT-5
TEST DATE: 9/14/2017 10:55:58 AM
TOTAL DEPTH: 51.017 ft

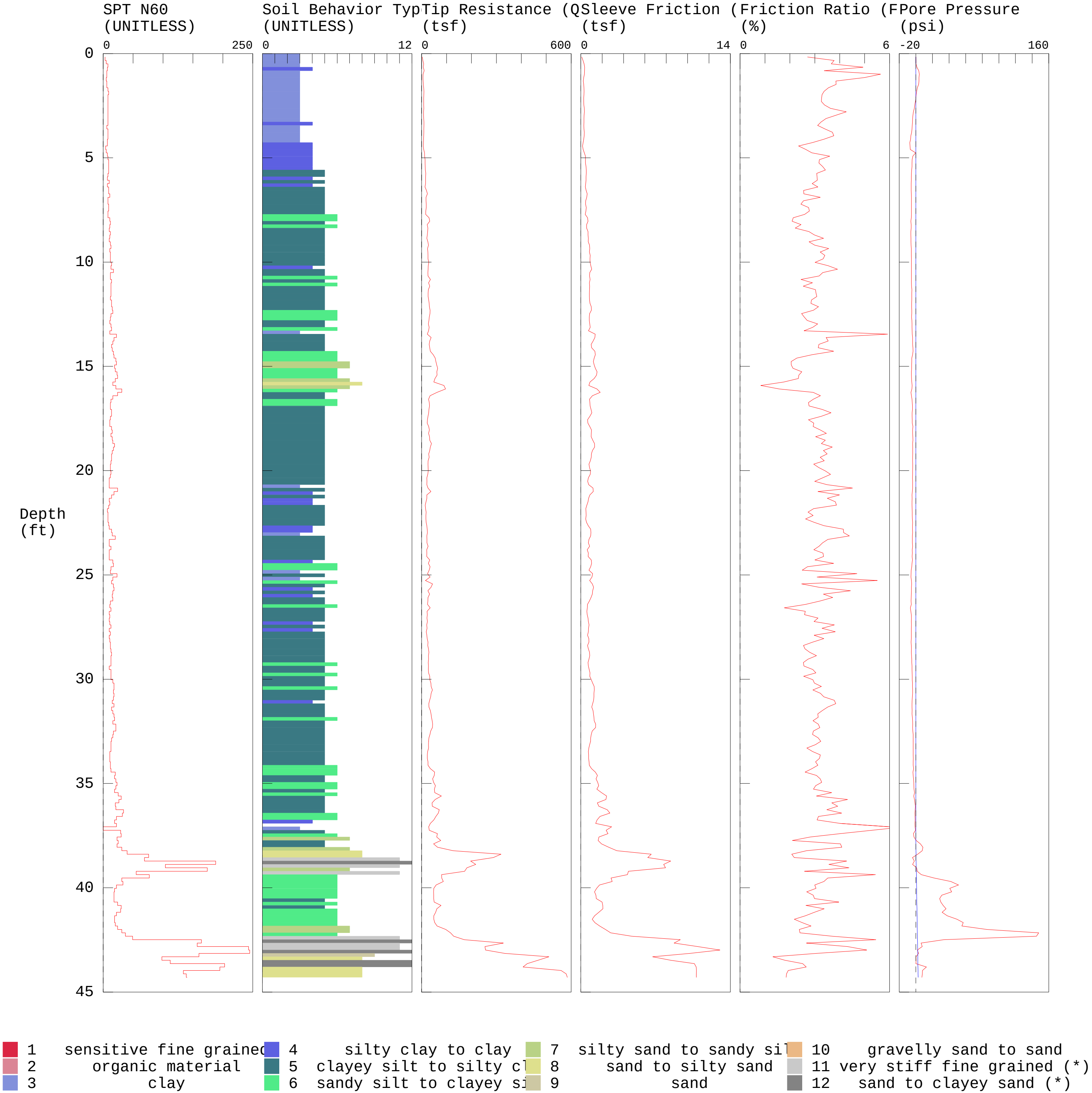


🟥 1 sensitive fine grainec
🟥 2 organic material
🟦 3 clay
🟦 4 silty clay to clay
🟦 5 clayey silt to silty cl
🟩 6 sandy silt to clayey si
🟩 7 silty sand to sandy sil
🟨 8 sand to silty sand
🟨 9 sand
🟧 10 gravelly sand to sand
⬜ 11 very stiff fine grained (*)
⬛ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

GeoDesign / CPT-6 / Seaside Heights School

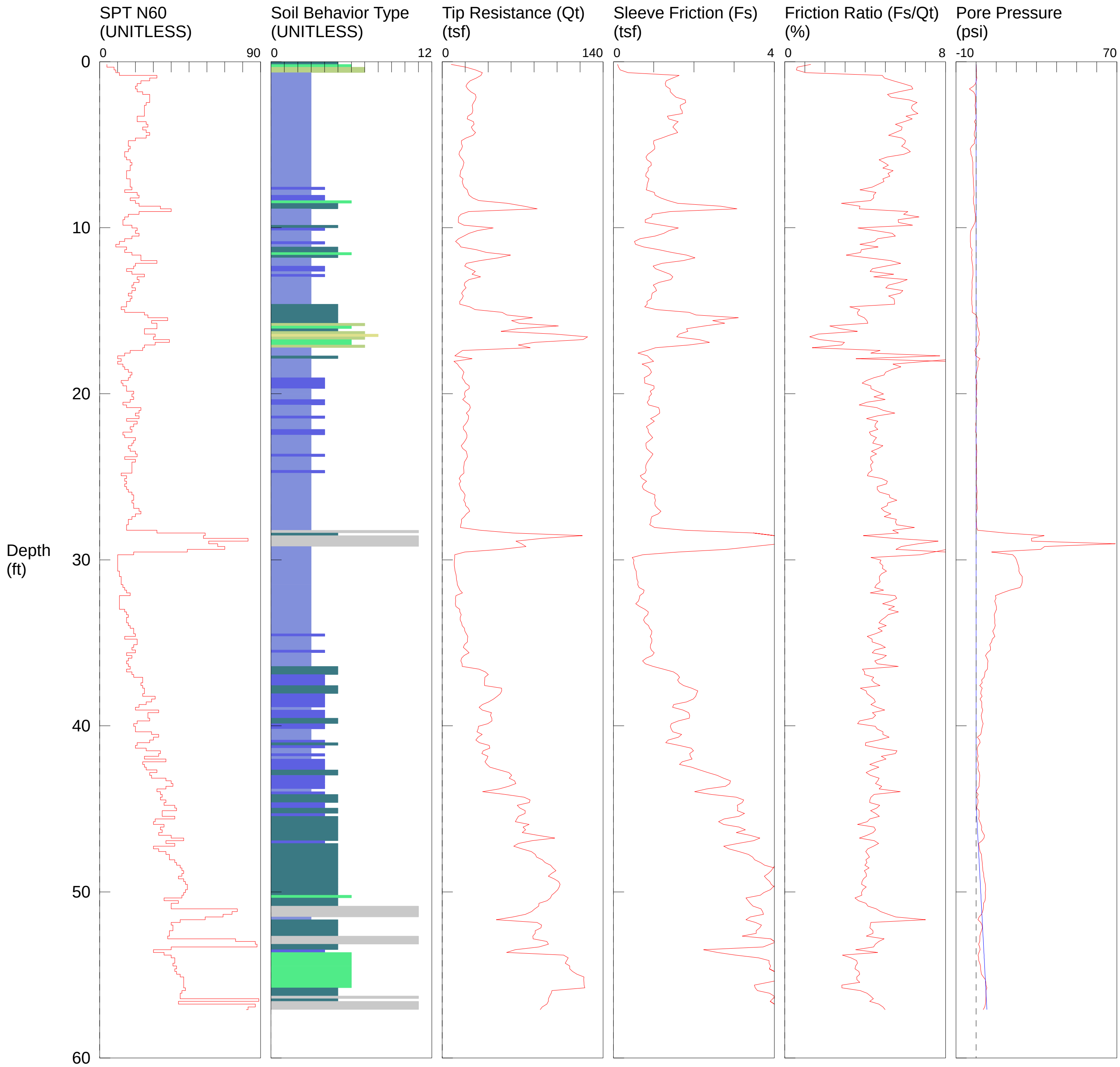
OPERATOR: OGE BB
CONE ID: DDG1415
HOLE NUMBER: CPT-6
TEST DATE: 9/15/2017 7:20:16 AM
TOTAL DEPTH: 44.291 ft



*SBT/SPT CORRELATION: UBC-1983

GeoDesign / CPT-7a / Seaside Heights School

OPERATOR: OGE BB
CONE ID: DDG1415
HOLE NUMBER: CPT-7a
TEST DATE: 9/15/2017 9:16:05 AM
TOTAL DEPTH: 57.087 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

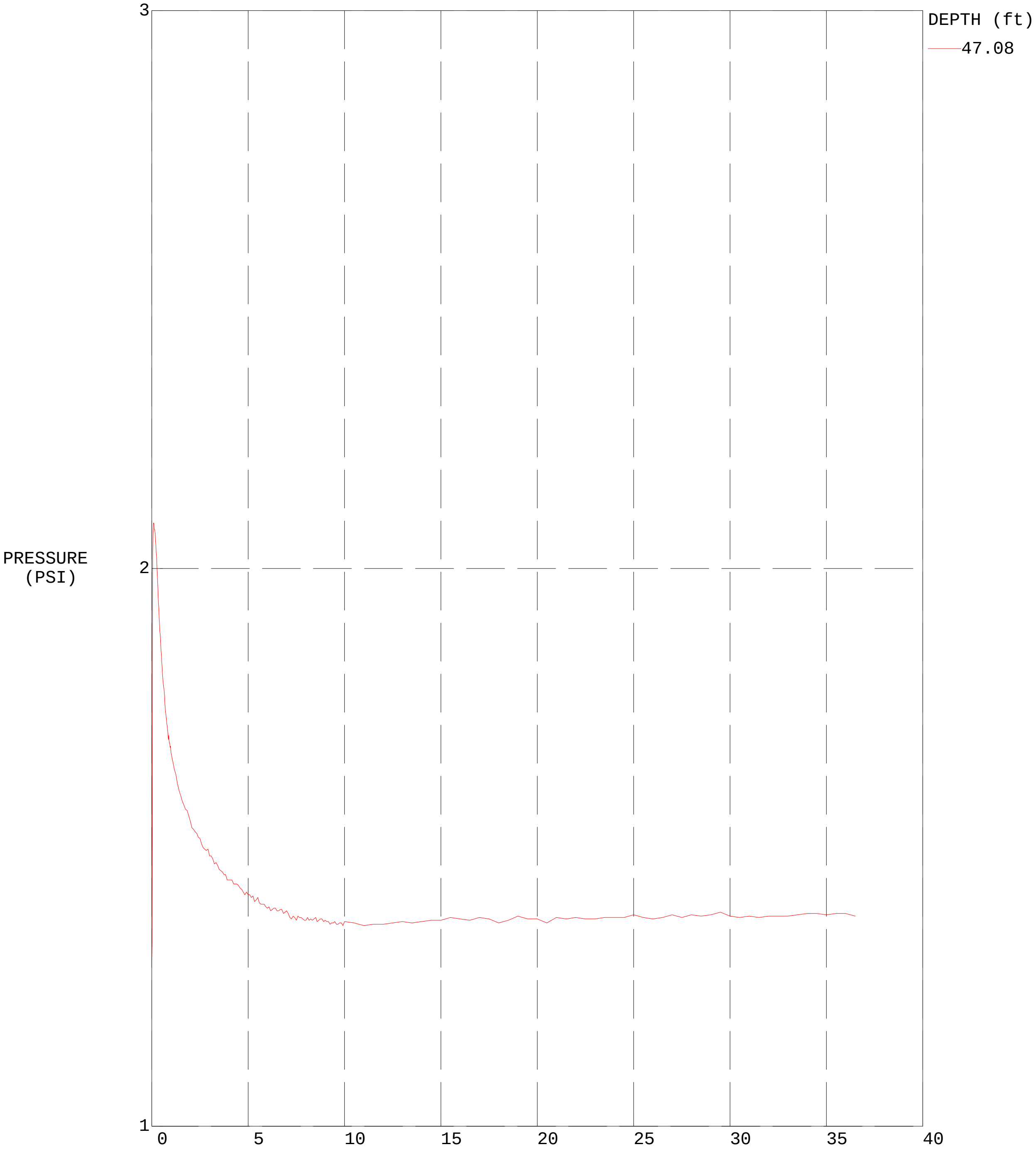
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand

■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: 17142 / GeoDesign / CPT-7a / Seaside SD

TEST DATE: 9/15/2017 9:16:05 AM



MAXIMUM PRESSURE = 2.082 (PSI)

HYDROSTATIC PRESSURE = 1.4 (PSI), WATER TABLE: 43.85 ft

APPENDIX C

APPENDIX C

OUTSIDE LABORATORY TESTING

EXPANSION INDEX TESTING

Expansion index testing was conducted on one soil sample (TP-7 at 10.0 feet BGS) from the Seaside middle/high school building site and two soil samples from the elementary school site in general accordance with ASTM D 4829. The testing consists of compacting soil at an approximately 50 percent degree of saturation, applying a confining pressure, inundating the sample with water, and measuring the resulting swell. The expansion index test results are presented in this appendix.

DRAINED RESIDUAL TORSIONAL SHEAR TESTING

Drained residual torsional shear testing was conducted on soil samples combined from boring B-1 at 65 to 70 feet BGS. The test method is performed by deforming a pre-sheared, reconstituted specimen at a controlled displacement rate until the constant drained shear resistance is reached for the shear plane. Tests were conducted under three confining stresses and the results are presented in this appendix.

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

EXPANSION INDEX TEST RESULTS

ASTM D 4829

Client Name: GeoDesign, Inc.
Project Name: Seaside School Campus
Project No.: SeasideSD-1-03

AP Job No.: 17-1045
Date: 10/25/17

Boring No.	Sample No.	Depth (ft)	Soil Description	Molded Dry Density (pcf)	Molded Moisture Content (%)	Init. Degree Saturation (%)	Measured Expansion Index	Corrected Expansion Index
TP-07	1	10	Sandy Clay	77.7	23.1	53.4	42	44
TP-31	2	3	Sandy Clay	72.9	25.6	52.8	81	83
B-07	3	15-25	Sandy Clay	85.8	17.4	48.7	115	113

ASTM EXPANSION CLASSIFICATION

Expansion Index	Classification
0-20	V. Low
21-50	Low
51-90	Medium
91-130	High
>130	V. High

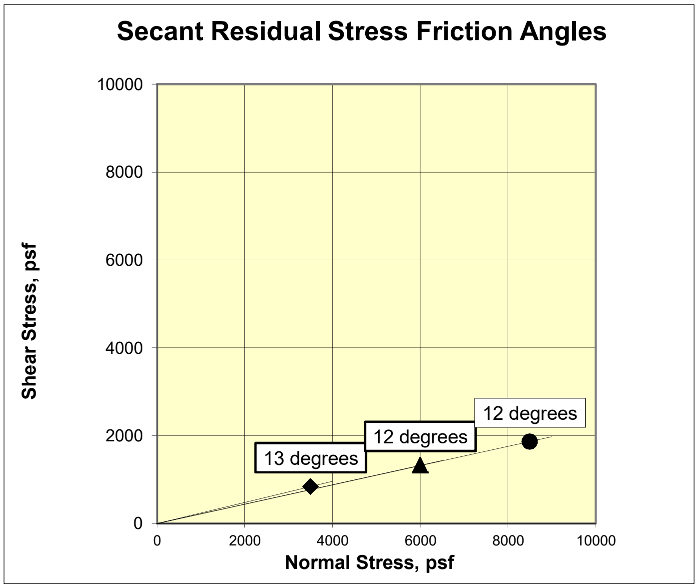
COOPER
TESTING LABORATORY

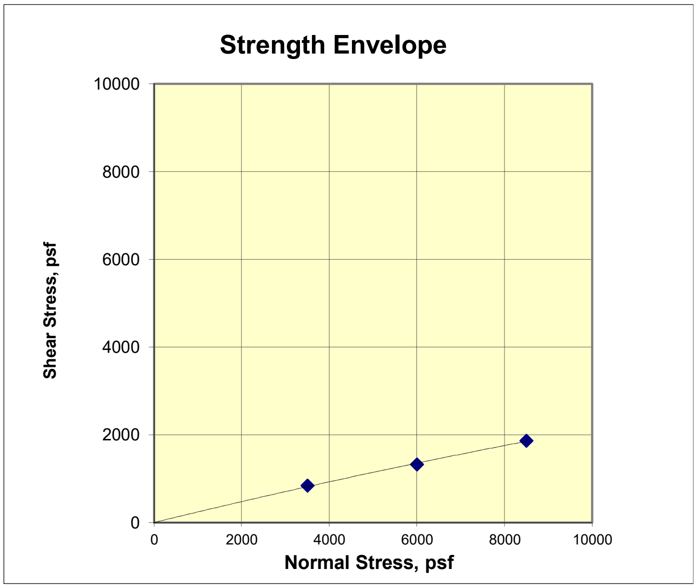
Drained Residual Torsional Shear Strength
(ASTM D6467)

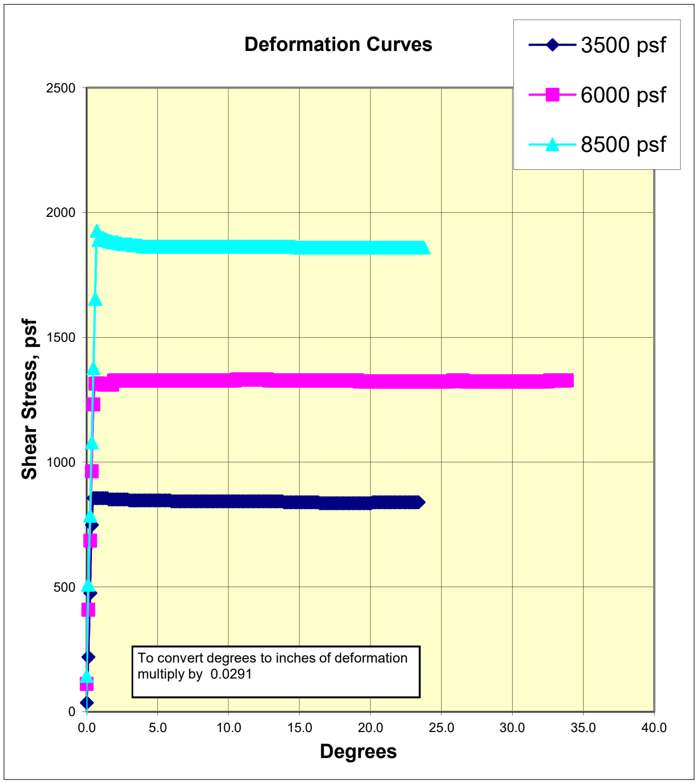
CTL Job No.: 932-004
Boring: B-1
Date: 10/23/2017
Clay, %:
Client: GeoDesign, Inc.
Sample:
By: PJ
LL:
Project Name: Seaside School Campus
Depth (ft): 65-70
Checked: DC
PL:
Project Number: SeasideSD-1-03-01
Test Type: Fully Softened Residual
Soil Type: Dark Grayish Brown Sandy CLAY

Normal Stress, psf:	3500	6000	8500
Secant Phi, deg.:	13	12	12

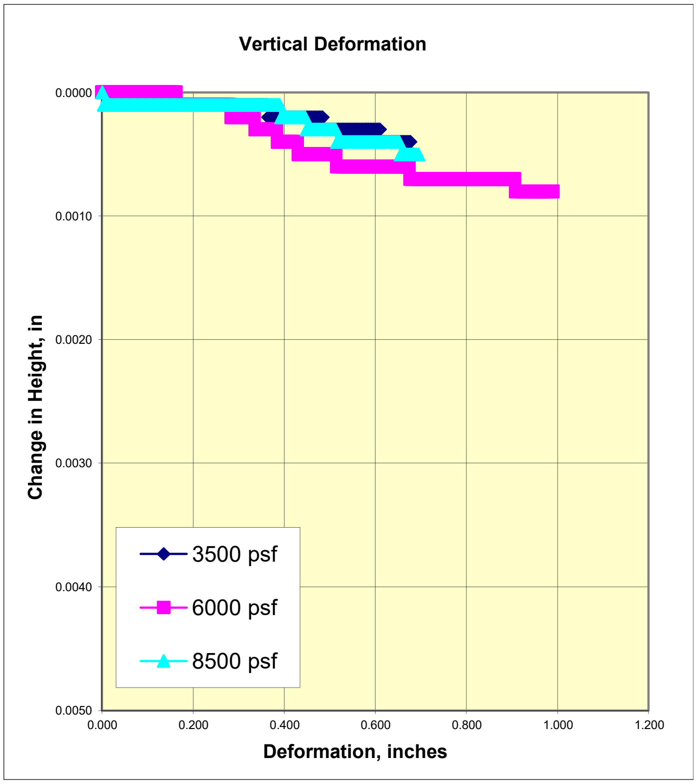
Remarks: A small friction correction was applied to each point.







To convert degrees to inches of deformation multiply by 0.0291



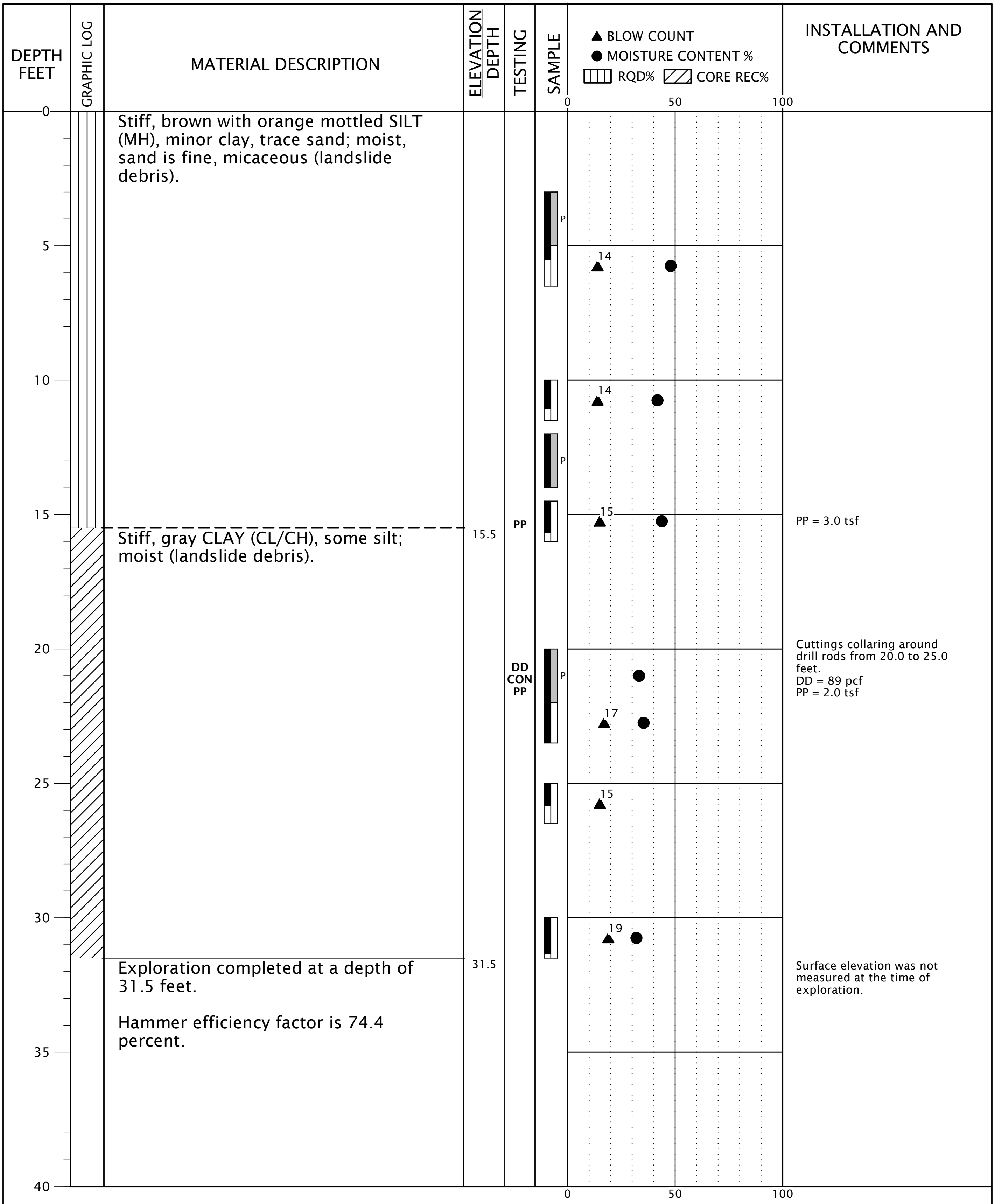
APPENDIX D

APPENDIX D

GEODESIGN NEARBY EXPLORATIONS

GeoDesign completed explorations for the proposed water reservoir near the upper east end of the site, which consisted of drilling three borings (B-9 through B-11) to depths ranging between 31.5 and 71.5 feet BGS, excavating three test pits (TP-23 through TP-25) to depths ranging between 10.0 and 11.0 feet BGS, and advancing one CPT probe (CPT-1) to a depth of 52.5 feet BGS. The borings were drilled using a track-mounted drill rig and mud rotary drilling methods by Western States Soil Conservation, Inc. of Hubbard, Oregon, on September 14 through 15, 2017. The test pits were excavated using a Komatsu PC60 tracked excavator by Dan J. Fischer Excavating, Inc. of Forest Grove, Oregon, on September 8, 2017. The CPT was performed in general accordance with ASTM D 5778 by Oregon Geotechnical Explorations, Inc. of Keizer, Oregon, on September 13, 2017. The associated exploration logs are presented in this appendix. Although not presented in this appendix, GeoDesign also completed explorations for the proposed elementary school expansion below the west end of the site.

Approximate locations of the explorations are shown on Figure 2. 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.



Stiff, brown with orange mottled SILT (MH), minor clay, trace sand; moist, sand is fine, micaceous (landslide debris).

Stiff, gray CLAY (CL/CH), some silt; moist (landslide debris).

Exploration completed at a depth of 31.5 feet.

Hammer efficiency factor is 74.4 percent.

PP = 3.0 tsf

Cuttings collaring around drill rods from 20.0 to 25.0 feet.
DD = 89 pcf
PP = 2.0 tsf

Surface elevation was not measured at the time of exploration.

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: CR COMPLETED: 09/14/17

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 3 7/8 inches

GEODESIGN INC
9450 SW Commerce Circle - Suite 300
Wilsonville OR 97070
503.968.8787 www.geodesigninc.com

SEASIDEC-3-01

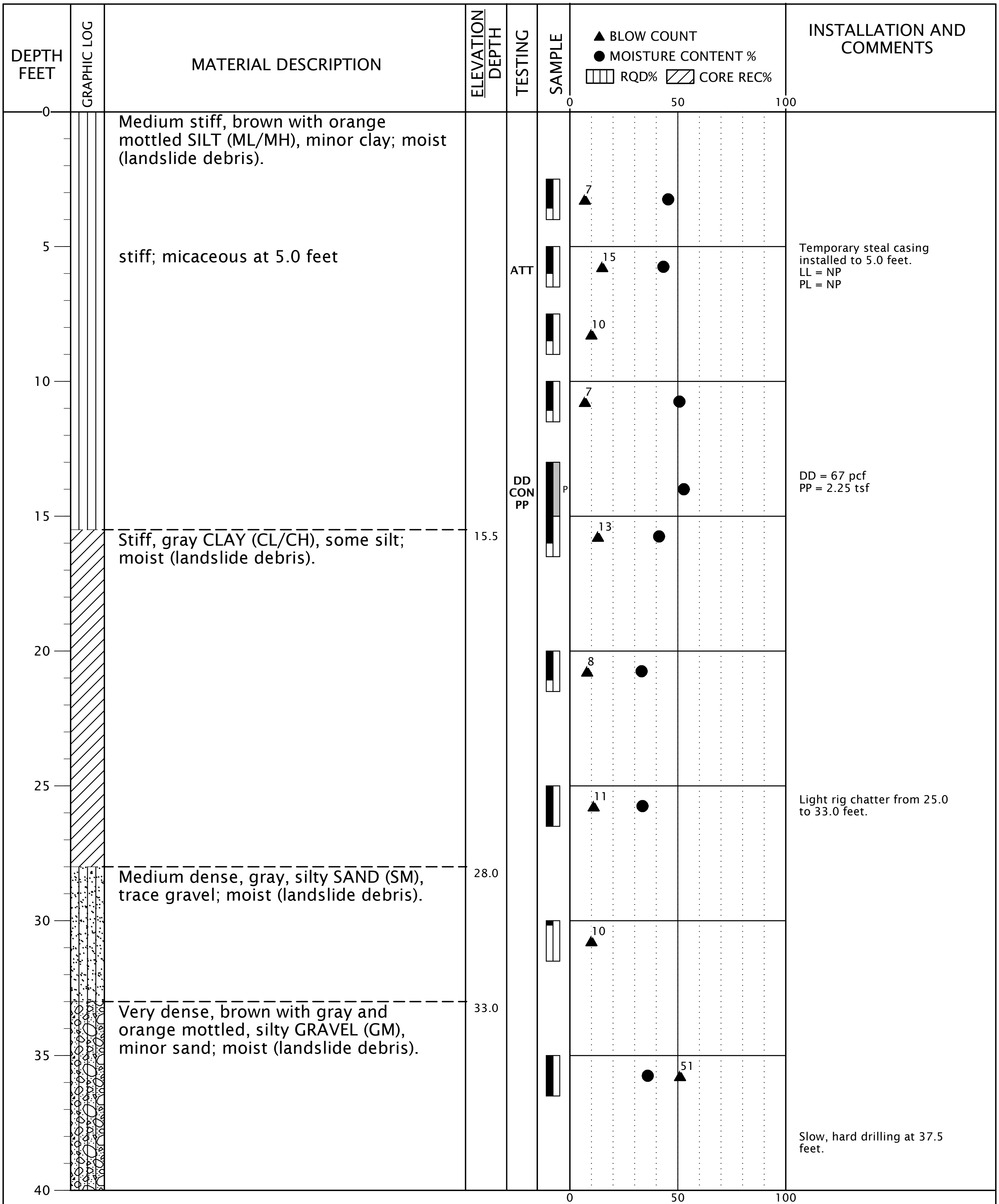
NOVEMBER 2017

BORING B-9

SEASIDE CAMPUS WATER TANK
SEASIDE, OR

FIGURE A-1

BORING LOG SEASIDEC-3-01-B9_11-TP23_25.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:RC:KT



Medium stiff, brown with orange mottled SILT (ML/MH), minor clay; moist (landslide debris).

stiff; micaceous at 5.0 feet

Stiff, gray CLAY (CL/CH), some silt; moist (landslide debris).

Medium dense, gray, silty SAND (SM), trace gravel; moist (landslide debris).

Very dense, brown with gray and orange mottled, silty GRAVEL (GM), minor sand; moist (landslide debris).

Temporary steal casing installed to 5.0 feet.
LL = NP
PL = NP

DD = 67 pcf
PP = 2.25 tsf

Light rig chatter from 25.0 to 33.0 feet.

Slow, hard drilling at 37.5 feet.

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: CR

COMPLETED: 09/14/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 3 7/8 inches

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SEASIDEC-3-01

NOVEMBER 2017

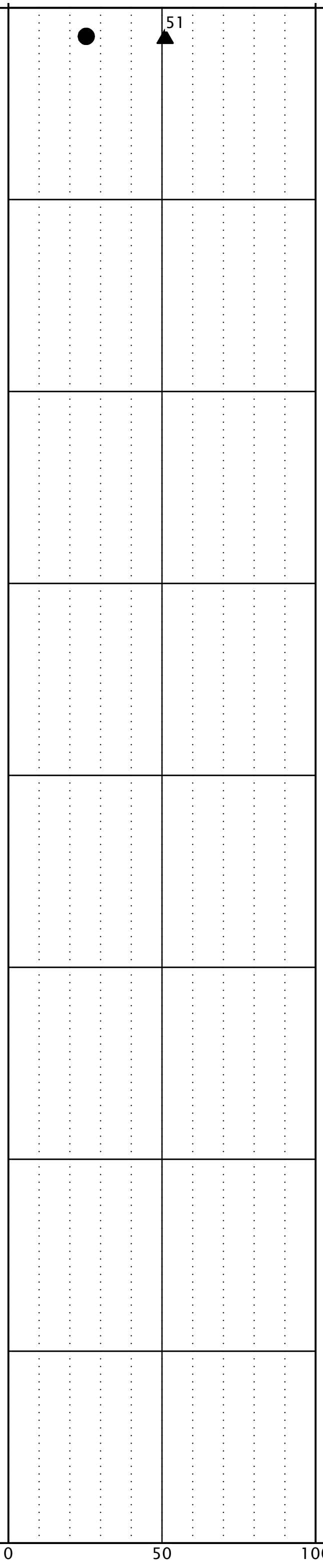
BORING B-10

SEASIDE CAMPUS WATER TANK
SEASIDE, OR

FIGURE A-2

BORING LOG SEASIDEC-3-01-B9_11-TP23_25.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:RC:KT

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)					
		Exploration completed at a depth of 41.5 feet. Hammer efficiency factor is 74.4 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: CR
COMPLETED: 09/14/17

BORING METHOD: mud rotary (see document text)
BORING BIT DIAMETER: 3 7/8 inches

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SEASIDEC-3-01

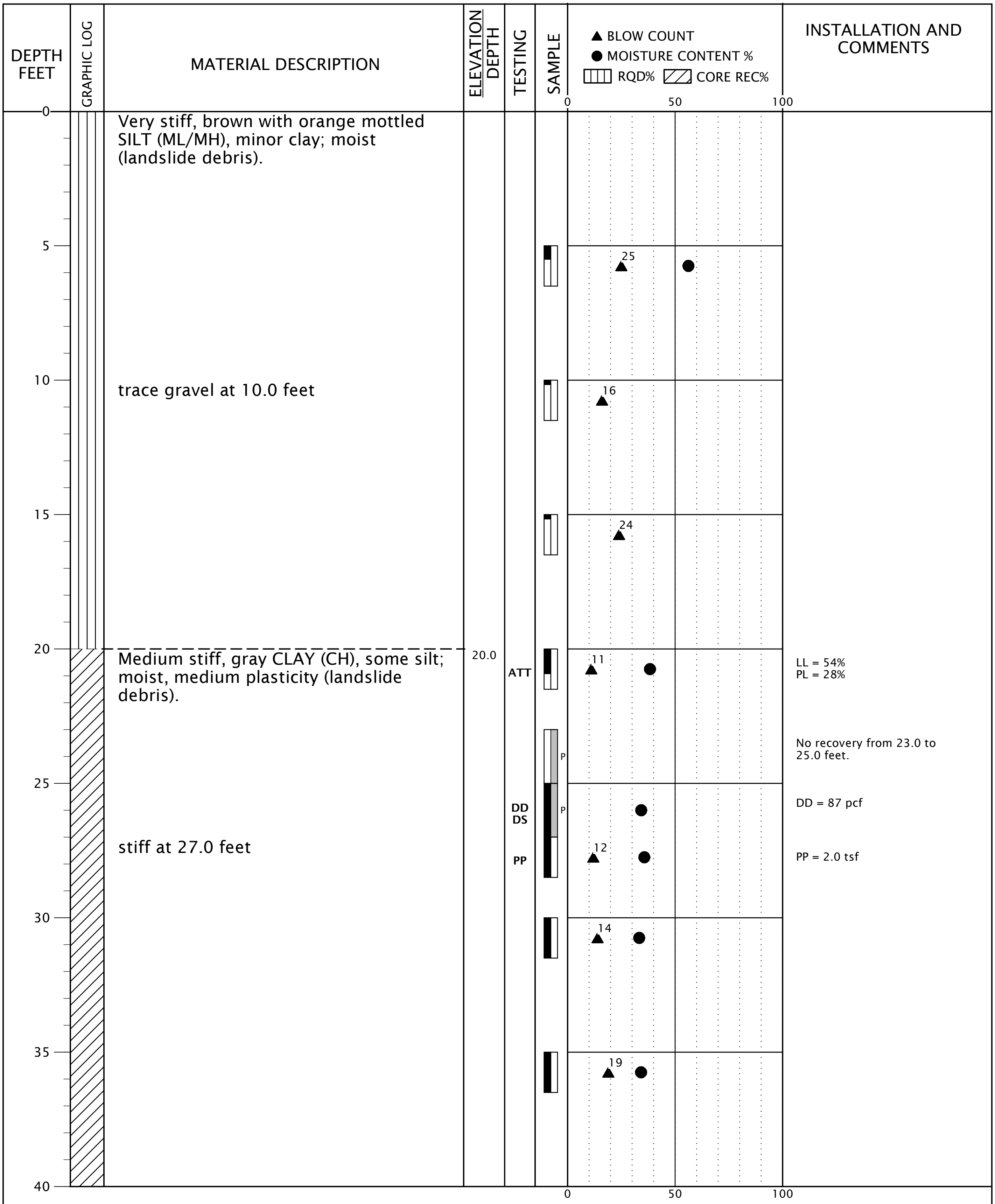
NOVEMBER 2017

BORING B-10
(continued)

SEASIDE CAMPUS WATER TANK
SEASIDE, OR

FIGURE A-2

BORING LOG SEASIDEC-3-01-B9_11-TP23_25.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:RC:KT



Very stiff, brown with orange mottled SILT (ML/MH), minor clay; moist (landslide debris).

trace gravel at 10.0 feet

20.0 Medium stiff, gray CLAY (CH), some silt; moist, medium plasticity (landslide debris).

stiff at 27.0 feet

ATT — LL = 54%
PL = 28%

No recovery from 23.0 to 25.0 feet.

DD
DS — DD = 87 pcf

PP — PP = 2.0 tsf

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: CR COMPLETED: 09/15/17

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 3 7/8 inches

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SEASIDEC-3-01

NOVEMBER 2017

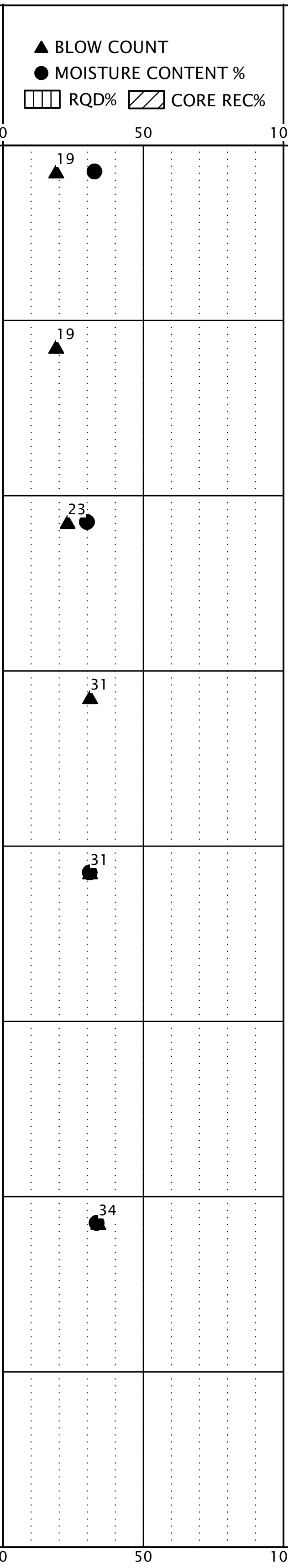
BORING B-11

SEASIDE CAMPUS WATER TANK
SEASIDE, OR

FIGURE A-3

BORING LOG SEASIDEC-3-01-B9_11-TP23_25.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:RC:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	BLOW COUNT	INSTALLATION AND COMMENTS
40	(continued from previous page)				19	
45					19	
50	very stiff at 50.0 feet				23	
55	hard at 55.0 feet				31	
60					31	
65						
70					34	
	Exploration completed at a depth of 71.5 feet. Hammer efficiency factor is 74.4 percent.	71.5				Surface elevation was not measured at the time of exploration.
75						
80						



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: CR

COMPLETED: 09/15/17

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 3 7/8 inches

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SEASIDEC-3-01

NOVEMBER 2017

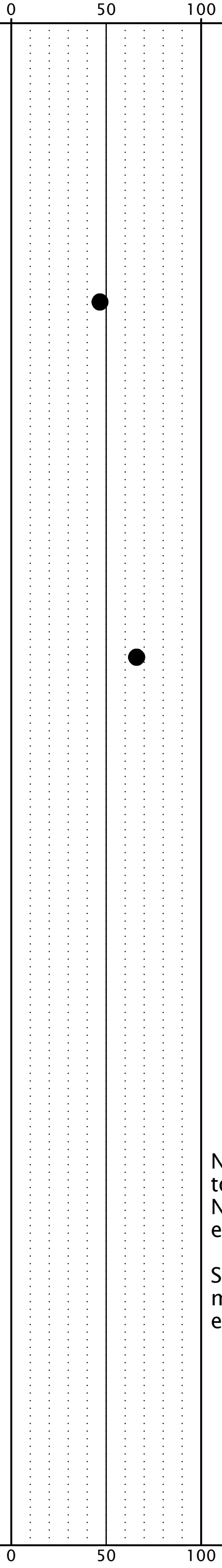
BORING B-11
(continued)

SEASIDE CAMPUS WATER TANK
SEASIDE, OR

FIGURE A-3

BORING LOG SEASIDEC-3-01-B9_11-TP23_25.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:RC:KT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT %	COMMENTS
0.0	▨	Medium stiff, brown CLAY (CL/CH), some silt, trace sand; moist (topsoil to 12 inches, 8-inch-thick root zone).					
2.5	▨				☒	●	
5.0	▨				☒	●	
7.5	▨						
10.0	▨	stiff, gray with orange mottles at 10.0 feet					
		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							
15.0							



EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: RSK

COMPLETED: 09/08/17

EXCAVATION METHOD: trackhoe (see document text)

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SEASIDEC-3-01

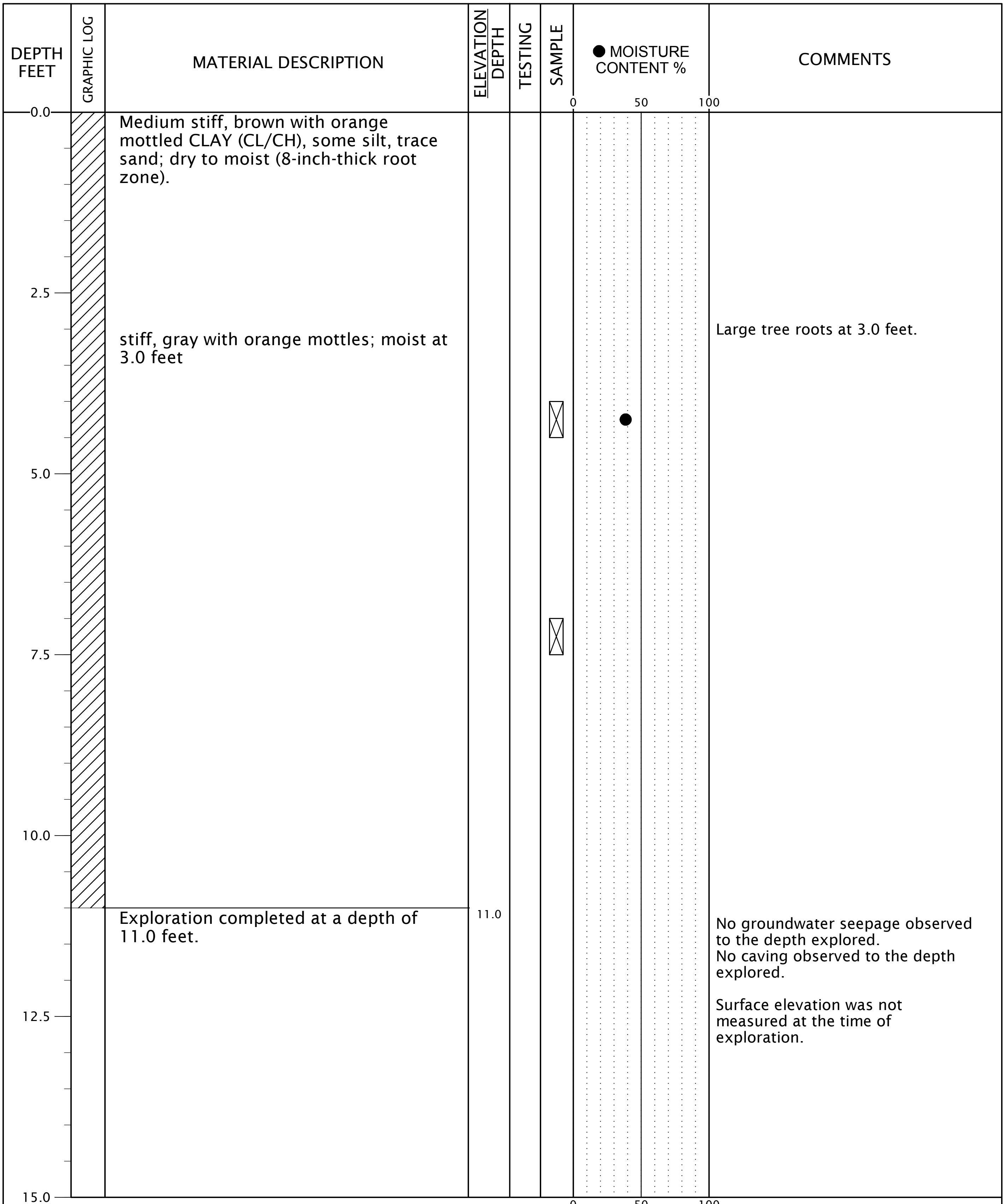
NOVEMBER 2017

TEST PIT TP-23

SEASIDE CAMPUS WATER TANK
SEASIDE, OR

FIGURE A-4

TEST PIT LOG - 1 PER PAGE SEASIDEC-3-01-B9_11-TP23_25.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:RC:KT



Medium stiff, brown with orange mottled CLAY (CL/CH), some silt, trace sand; dry to moist (8-inch-thick root zone).

stiff, gray with orange mottles; moist at 3.0 feet

Large tree roots at 3.0 feet.

Exploration completed at a depth of 11.0 feet.

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.

EXCAVATED BY: Dan J. Fischer Excavating, Inc.

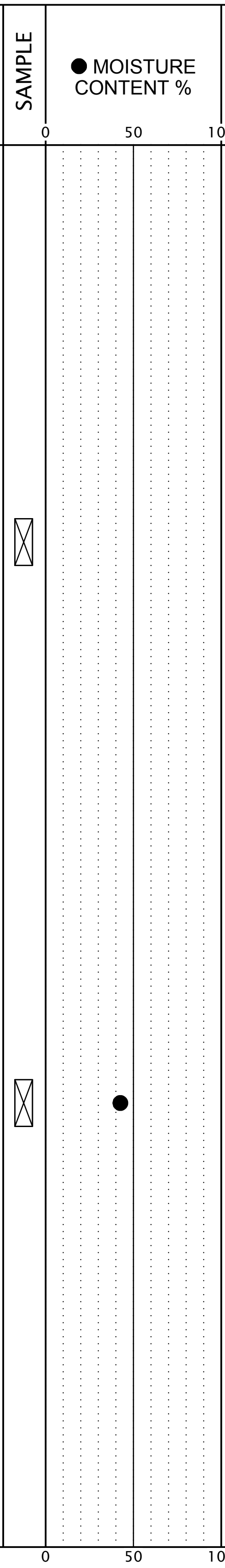
LOGGED BY: RSK

COMPLETED: 09/08/17

EXCAVATION METHOD: trackhoe (see document text)

GEODESIGN INC 9450 SW Commerce Circle - Suite 300 Wilsonville OR 97070 503.968.8787 www.geodesigninc.com	SEASIDEC-3-01	**TEST PIT TP-24**	
	NOVEMBER 2017	SEASIDE CAMPUS WATER TANK SEASIDE, OR	**FIGURE A-5**

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium stiff, brown CLAY (CL/CH), some silt, trace sand; moist (topsoil to 12 inches, 10-inch-thick root zone).				
2.5					
	stiff, gray-brown at 3.0 feet				
				☒	
5.0					
					Boulder at 7.0 feet.
7.5					
	stiff to very stiff, gray-brown with orange mottles at 8.0 feet				
	with cobbles at 9.0 feet				
10.0				☒	Boulder at 10.0 feet.
	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					
15.0					



EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: RSK

COMPLETED: 09/08/17

EXCAVATION METHOD: trackhoe (see document text)

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SEASIDEC-3-01

NOVEMBER 2017

TEST PIT TP-25

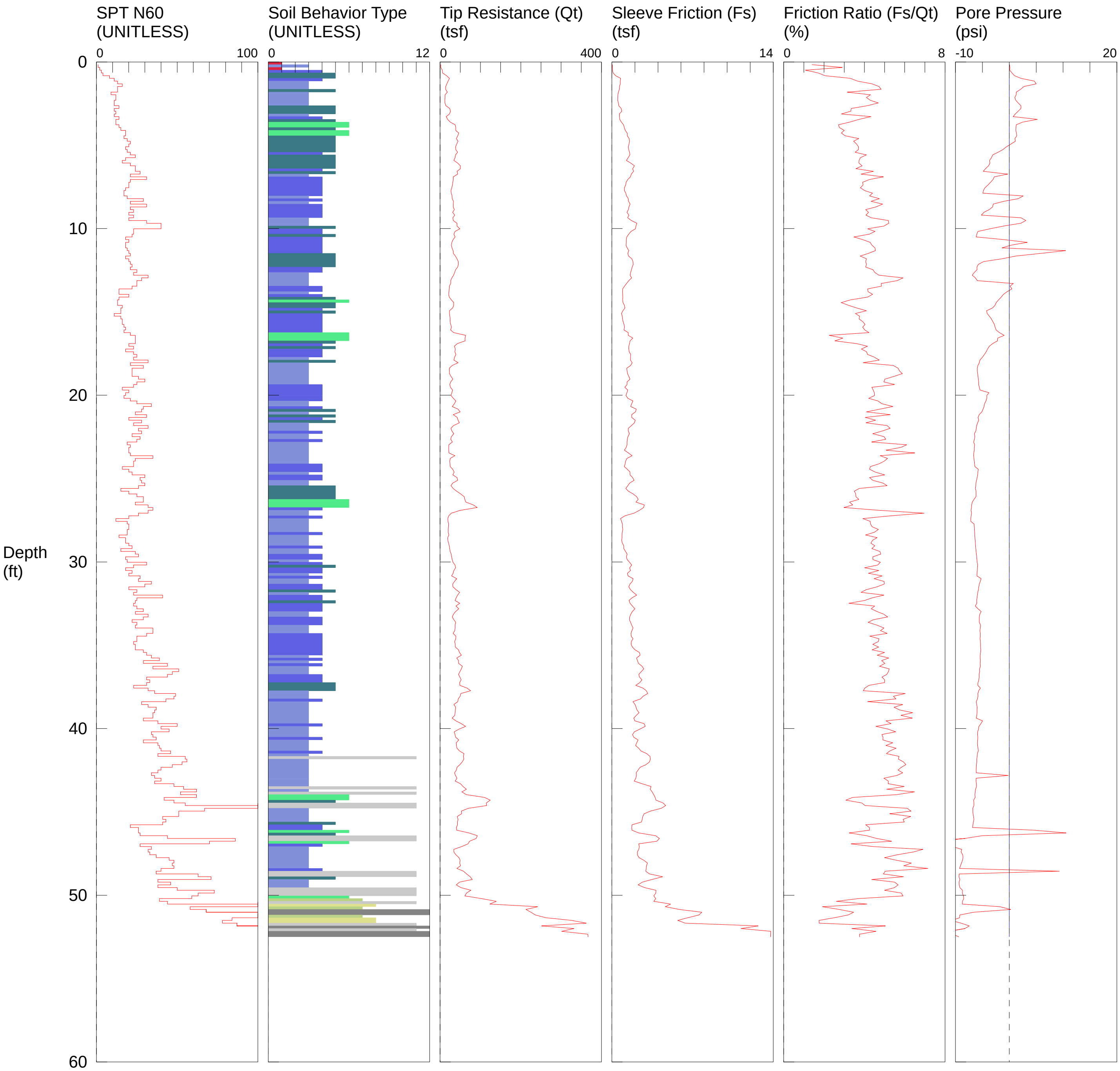
SEASIDE CAMPUS WATER TANK
SEASIDE, OR

FIGURE A-6

TEST PIT LOG - 1 PER PAGE SEASIDEC-3-01-B9_11-TP23_25.GPJ GEODESIGN.GDT PRINT DATE: 11/18/17:RC:KT

GeoDesign / CPT-1 / Seaside Heights School Tank

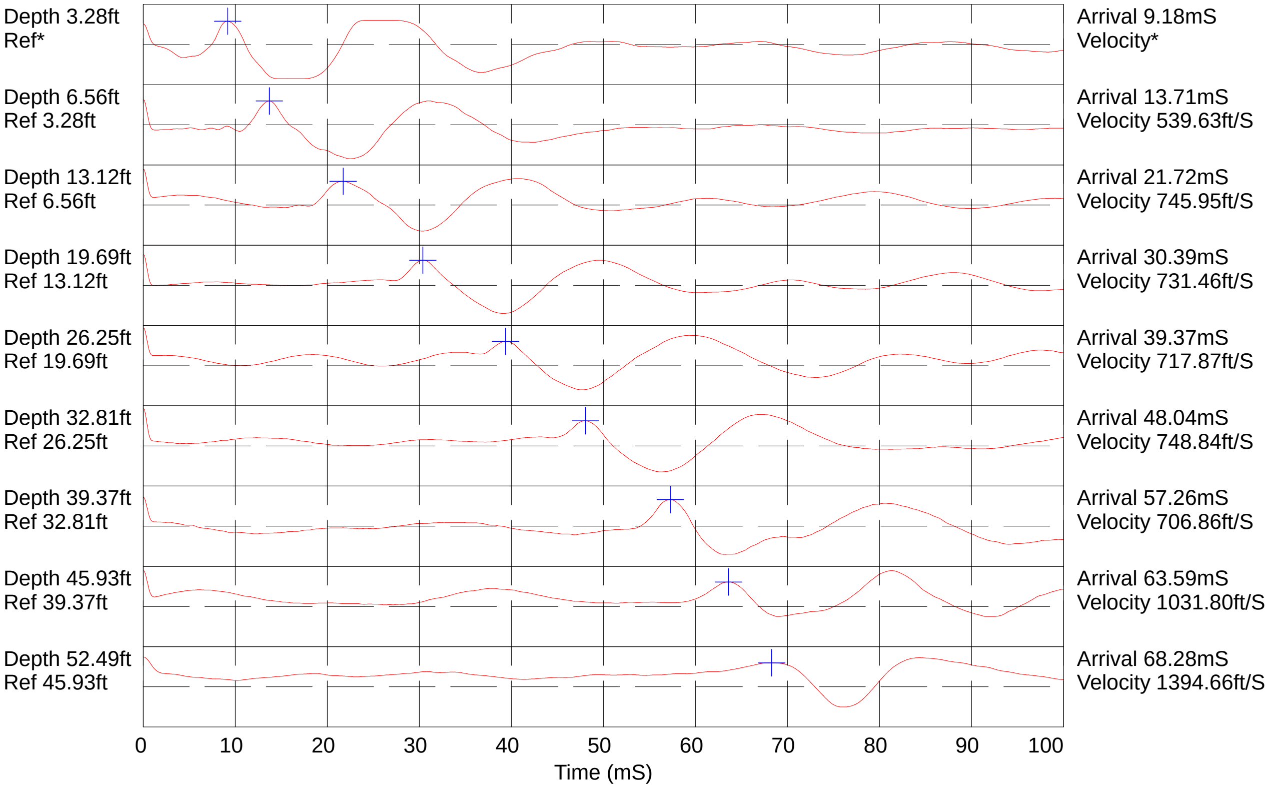
OPERATOR: OGE BB
CONE ID: DDG1415
HOLE NUMBER: CPT-1
TEST DATE: 9/13/2017 9:08:33 AM
TOTAL DEPTH: 52.493 ft



🟥 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

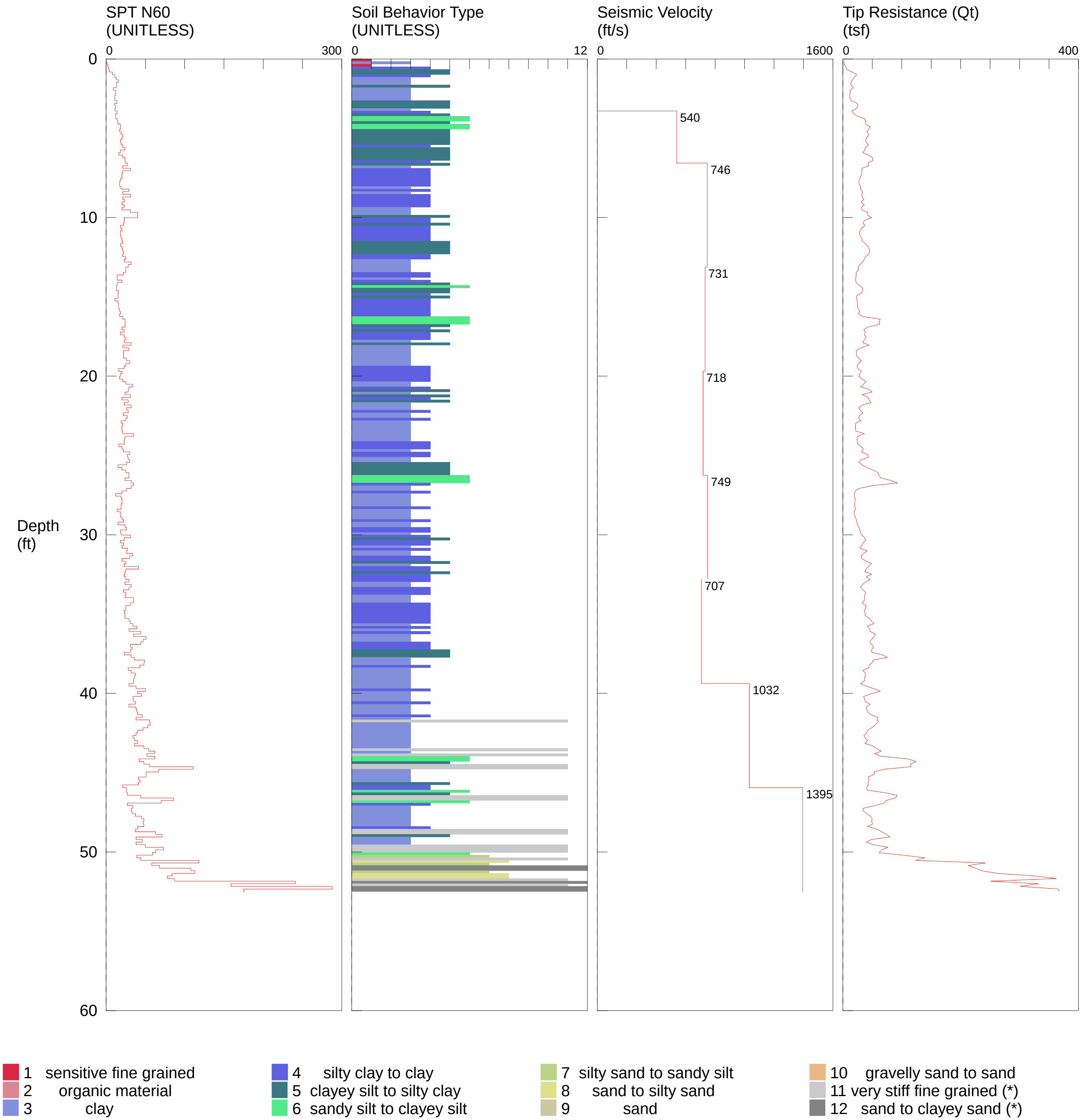
COMMENT: 17142 / GeoDesign / CPT-1 / Seaside SD



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

GeoDesign / CPT-1 / Seaside Heights School Tank

OPERATOR: OGE BB
CONE ID: DDG1415
HOLE NUMBER: CPT-1
TEST DATE: 9/13/2017 9:08:33 AM
TOTAL DEPTH: 52.493 ft



*SBT/SPT CORRELATION: UBC-1983

GeoDesign / CPT-1 / Seaside Heights School Tank

OPERATOR: OGE BB
CONE ID: DDG1415
HOLE NUMBER: CPT-1
TEST DATE: 9/13/2017 9:08:33 AM
TOTAL DEPTH: 52.493 ft

Depth ft	Tip Resistance (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	Friction Ratio (Fs/Qt) (%)	Pore Pressure (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
0.164	1.84	0.0258	1.406	0.043	1	1	sensitive fine grained
0.328	1.91	0.0554	2.899	0.098	2	3	clay
0.492	5.46	0.0589	1.078	0.094	3	1	sensitive fine grained
0.656	6.24	0.1065	1.706	0.518	4	4	silty clay to clay
0.820	16.02	0.3284	2.051	0.983	8	5	clayey silt to silty clay
0.984	23.13	0.7647	3.305	2.154	11	5	clayey silt to silty clay
1.148	19.80	0.7233	3.653	4.641	13	4	silty clay to clay
1.312	16.45	0.7213	4.384	4.996	16	3	clay
1.476	13.93	0.6575	4.719	2.722	13	3	clay
1.640	13.49	0.6509	4.827	2.108	13	3	clay
1.804	17.84	0.5627	3.155	1.307	9	5	clayey silt to silty clay
1.969	12.12	0.5212	4.302	1.262	12	3	clay
2.133	12.32	0.5053	4.102	1.024	12	3	clay
2.297	11.54	0.4956	4.293	1.211	11	3	clay
2.461	11.94	0.5606	4.695	1.648	11	3	clay
2.625	14.29	0.5932	4.151	2.161	14	3	clay
2.789	23.79	0.7966	3.348	2.168	11	5	clayey silt to silty clay
2.953	25.71	0.8556	3.327	1.664	12	5	clayey silt to silty clay
3.117	22.93	0.6543	2.853	1.118	11	5	clayey silt to silty clay
3.281	15.06	0.6516	4.327	0.710	14	3	clay
3.445	18.28	0.6830	3.737	5.212	12	4	silty clay to clay
3.609	25.35	0.8294	3.271	2.454	12	5	clayey silt to silty clay
3.773	37.02	1.0043	2.713	1.305	14	6	sandy silt to clayey silt
3.937	38.85	1.0707	2.756	1.240	15	6	sandy silt to clayey silt
4.101	38.60	1.1573	2.999	1.209	18	5	clayey silt to silty clay
4.265	46.23	1.3204	2.856	1.334	18	6	sandy silt to clayey silt
4.429	43.73	1.3349	3.053	1.391	17	6	sandy silt to clayey silt
4.593	40.33	1.5003	3.720	1.043	19	5	clayey silt to silty clay
4.757	44.31	1.5351	3.464	1.125	21	5	clayey silt to silty clay
4.921	41.22	1.4889	3.612	0.324	20	5	clayey silt to silty clay
5.085	38.41	1.4222	3.703	-0.508	18	5	clayey silt to silty clay
5.249	38.99	1.4467	3.711	-1.178	19	5	clayey silt to silty clay
5.413	43.36	1.5363	3.543	-2.094	21	5	clayey silt to silty clay
5.577	38.20	1.5631	4.092	-3.089	24	4	silty clay to clay
5.741	37.11	1.4115	3.804	-3.324	18	5	clayey silt to silty clay
5.906	33.84	1.2634	3.734	-3.672	16	5	clayey silt to silty clay
6.070	43.50	1.6197	3.723	-3.578	21	5	clayey silt to silty clay
6.234	50.35	1.9603	3.894	-3.852	24	5	clayey silt to silty clay
6.398	50.38	1.8026	3.578	-4.449	24	5	clayey silt to silty clay
6.562	42.19	1.8768	4.448	-4.823	27	4	silty clay to clay
6.726	43.90	1.6858	3.840	-0.297	21	5	clayey silt to silty clay
6.890	32.43	1.6013	4.937	-2.804	31	3	clay
7.054	32.32	1.3758	4.256	-3.012	21	4	silty clay to clay
7.218	31.58	1.2362	3.914	-3.480	20	4	silty clay to clay

Depth ft	Tip Resistance (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	Friction Ratio (Fs/Qt) (%)	Pore Pressure (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
7.382	30.55	1.2008	3.931	-3.972	20	4	silty clay to clay
7.546	28.65	1.0889	3.801	-4.531	18	4	silty clay to clay
7.710	26.88	1.0772	4.007	-4.811	17	4	silty clay to clay
7.874	27.39	1.2043	4.397	-4.926	17	4	silty clay to clay
8.038	29.79	1.2675	4.255	2.578	19	4	silty clay to clay
8.202	30.10	1.4267	4.740	1.664	29	3	clay
8.366	33.25	1.4404	4.332	-1.024	21	4	silty clay to clay
8.530	32.39	1.5885	4.904	-2.993	31	3	clay
8.694	32.27	1.4738	4.567	-3.096	21	4	silty clay to clay
8.858	35.29	1.4331	4.061	-3.751	23	4	silty clay to clay
9.022	30.90	1.2994	4.206	-4.720	20	4	silty clay to clay
9.186	36.20	1.4765	4.078	-5.197	23	4	silty clay to clay
9.350	31.58	1.3687	4.335	2.161	20	4	silty clay to clay
9.514	32.36	1.6789	5.188	3.099	31	3	clay
9.678	41.86	2.1797	5.207	2.120	40	3	clay
9.843	41.96	2.0846	4.967	-1.065	40	3	clay
10.007	48.96	2.0405	4.168	-3.610	23	5	clayey silt to silty clay
10.171	36.71	1.6614	4.525	-5.838	23	4	silty clay to clay
10.335	34.23	1.4504	4.238	-6.054	22	4	silty clay to clay
10.499	37.14	1.2912	3.477	-6.152	18	5	clayey silt to silty clay
10.663	32.00	1.2518	3.913	-1.000	20	4	silty clay to clay
10.827	28.90	1.2393	4.287	3.336	18	4	silty clay to clay
10.991	27.93	1.2209	4.371	0.257	18	4	silty clay to clay
11.155	29.25	1.3229	4.522	-1.326	19	4	silty clay to clay
11.319	32.07	1.4598	4.551	10.452	20	4	silty clay to clay
11.483	32.92	1.3815	4.197	5.854	21	4	silty clay to clay
11.647	37.91	1.4394	3.797	1.218	18	5	clayey silt to silty clay
11.811	42.06	1.7244	4.100	-1.355	20	5	clayey silt to silty clay
11.975	44.82	1.8204	4.062	-4.768	21	5	clayey silt to silty clay
12.139	45.36	1.8579	4.096	-5.737	22	5	clayey silt to silty clay
12.303	43.40	1.7632	4.063	-6.008	21	5	clayey silt to silty clay
12.467	38.38	1.6981	4.425	-5.989	25	4	silty clay to clay
12.631	36.07	1.6304	4.520	-6.516	23	4	silty clay to clay
12.795	33.48	1.5815	4.724	-6.893	32	3	clay
12.959	28.92	1.7112	5.918	-6.301	28	3	clay
13.123	26.05	1.4641	5.620	-5.953	25	3	clay
13.287	26.21	1.2659	4.830	0.736	25	3	clay
13.451	22.71	1.1003	4.844	0.125	22	3	clay
13.615	22.56	0.9374	4.155	0.482	14	4	silty clay to clay
13.780	22.01	0.9198	4.179	-0.401	14	4	silty clay to clay
13.944	21.13	0.9302	4.403	-1.084	20	3	clay
14.108	22.63	0.9430	4.167	-1.506	14	4	silty clay to clay
14.272	27.98	0.9309	3.327	-2.000	13	5	clayey silt to silty clay
14.436	33.59	0.9566	2.848	-2.413	13	6	sandy silt to clayey silt
14.600	33.60	1.0820	3.220	-2.679	16	5	clayey silt to silty clay
14.764	30.99	1.1226	3.622	-3.137	15	5	clayey silt to silty clay
14.928	23.12	0.9485	4.103	-4.236	15	4	silty clay to clay
15.092	23.11	0.8210	3.553	-4.104	11	5	clayey silt to silty clay
15.256	24.02	0.9021	3.756	-3.696	15	4	silty clay to clay
15.420	24.84	0.9283	3.737	-3.339	16	4	silty clay to clay
15.584	24.67	0.9697	3.930	-3.039	16	4	silty clay to clay
15.748	26.06	1.0506	4.031	-2.792	17	4	silty clay to clay
15.912	28.12	1.0997	3.911	-2.629	18	4	silty clay to clay

Depth ft	Tip Resistance (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	Friction Ratio (Fs/Qt) (%)	Pore Pressure (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
16.076	26.00	1.0446	4.018	-2.466	17	4	silty clay to clay
16.240	33.30	1.4069	4.225	-1.861	21	4	silty clay to clay
16.404	63.16	1.4230	2.253	-0.964	24	6	sandy silt to clayey silt
16.568	61.65	1.8095	2.935	-2.156	24	6	sandy silt to clayey silt
16.732	61.90	1.5722	2.540	-2.207	24	6	sandy silt to clayey silt
16.896	42.37	1.5338	3.620	-3.101	20	5	clayey silt to silty clay
17.060	35.30	1.4650	4.150	-3.713	23	4	silty clay to clay
17.224	37.91	1.4596	3.851	-4.037	18	5	clayey silt to silty clay
17.388	36.52	1.5004	4.108	-4.315	23	4	silty clay to clay
17.552	39.18	1.6277	4.154	-4.619	25	4	silty clay to clay
17.717	35.94	1.6143	4.492	-5.001	23	4	silty clay to clay
17.881	33.83	1.6024	4.737	-5.464	32	3	clay
18.045	44.55	1.7594	3.949	-5.583	21	5	clayey silt to silty clay
18.209	29.98	1.6275	5.428	-5.835	29	3	clay
18.373	22.68	1.2867	5.673	-5.958	22	3	clay
18.537	22.84	1.3104	5.738	-5.965	22	3	clay
18.701	23.00	1.3531	5.882	-5.883	22	3	clay
18.865	26.92	1.4720	5.467	-5.799	26	3	clay
19.029	31.49	1.5789	5.015	-5.766	30	3	clay
19.193	26.51	1.3159	4.965	-5.694	25	3	clay
19.357	23.88	1.3144	5.503	-5.687	23	3	clay
19.521	25.58	1.1181	4.370	-5.595	16	4	silty clay to clay
19.685	31.14	1.3809	4.434	-5.437	20	4	silty clay to clay
19.849	28.94	1.3004	4.493	-3.777	18	4	silty clay to clay
20.013	27.07	1.2195	4.505	-4.183	17	4	silty clay to clay
20.177	32.95	1.3873	4.210	-4.200	21	4	silty clay to clay
20.341	39.04	1.8237	4.672	-4.346	25	4	silty clay to clay
20.505	35.18	1.7139	4.871	-4.593	34	3	clay
20.669	30.01	1.6245	5.413	-4.691	29	3	clay
20.833	44.31	2.1131	4.769	-4.893	28	4	silty clay to clay
20.997	49.70	2.0384	4.101	-5.121	24	5	clayey silt to silty clay
21.161	32.45	1.7161	5.288	-5.605	31	3	clay
21.325	42.54	1.7164	4.035	-5.811	20	5	clayey silt to silty clay
21.490	44.33	2.0196	4.556	-5.711	28	4	silty clay to clay
21.654	47.68	1.9469	4.083	-5.890	23	5	clayey silt to silty clay
21.818	33.13	1.6978	5.124	-6.082	32	3	clay
21.982	26.92	1.4204	5.277	-6.130	26	3	clay
22.146	29.09	1.4236	4.893	-6.238	28	3	clay
22.310	34.18	1.5100	4.418	-6.485	22	4	silty clay to clay
22.474	27.99	1.3906	4.967	-6.397	27	3	clay
22.638	26.41	1.3355	5.058	-6.492	25	3	clay
22.802	30.22	1.3178	4.361	-6.627	19	4	silty clay to clay
22.966	21.57	1.3159	6.100	-6.444	21	3	clay
23.130	21.25	1.2333	5.804	-6.524	20	3	clay
23.294	21.25	1.0791	5.079	-6.524	20	3	clay
23.458	22.06	1.4334	6.499	-6.608	21	3	clay
23.622	36.46	1.7455	4.788	-6.634	35	3	clay
23.786	25.21	1.2973	5.146	-6.581	24	3	clay
23.950	23.82	1.1880	4.988	-6.495	23	3	clay
24.114	24.37	1.1504	4.721	-6.471	23	3	clay
24.278	25.02	1.0740	4.293	-6.409	16	4	silty clay to clay
24.442	31.22	1.3304	4.261	-5.756	20	4	silty clay to clay
24.606	34.80	1.5763	4.529	-5.847	22	4	silty clay to clay

Depth ft	Tip Resistance (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	Friction Ratio (Fs/Qt) (%)	Pore Pressure (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
24.770	31.62	1.5817	5.003	-5.881	30	3	clay
24.934	42.04	1.7911	4.260	-5.974	27	4	silty clay to clay
25.098	43.65	1.9266	4.413	-6.092	28	4	silty clay to clay
25.262	31.28	1.5403	4.924	-6.209	30	3	clay
25.427	26.92	1.3803	5.128	-6.226	26	3	clay
25.591	32.21	1.2070	3.748	-6.315	15	5	clayey silt to silty clay
25.755	40.92	1.4320	3.500	-6.200	20	5	clayey silt to silty clay
25.919	51.36	1.8374	3.578	-6.197	25	5	clayey silt to silty clay
26.083	59.95	2.1457	3.579	-6.202	29	5	clayey silt to silty clay
26.247	61.25	2.2817	3.725	-6.552	29	5	clayey silt to silty clay
26.411	63.88	2.0858	3.265	-6.895	24	6	sandy silt to clayey silt
26.575	82.24	2.8150	3.423	-7.015	32	6	sandy silt to clayey silt
26.739	92.06	2.7504	2.988	-7.032	35	6	sandy silt to clayey silt
26.903	50.66	2.3845	4.707	-7.126	32	4	silty clay to clay
27.067	27.63	1.9244	6.964	-7.037	26	3	clay
27.231	21.12	1.1217	5.311	-7.109	20	3	clay
27.395	19.07	0.7478	3.922	-7.162	12	4	silty clay to clay
27.559	19.45	0.8342	4.290	-7.162	19	3	clay
27.723	20.98	0.9031	4.305	-6.552	20	3	clay
27.887	20.68	0.9068	4.385	-6.512	20	3	clay
28.051	20.03	0.9393	4.690	-6.476	19	3	clay
28.215	20.05	0.9017	4.497	-6.444	19	3	clay
28.379	21.64	0.8782	4.059	-6.425	14	4	silty clay to clay
28.543	18.93	0.8754	4.625	-6.382	18	3	clay
28.707	19.21	0.8454	4.401	-6.346	18	3	clay
28.871	20.86	0.9024	4.326	-6.317	20	3	clay
29.035	22.66	1.0238	4.518	-6.267	22	3	clay
29.199	23.75	1.0371	4.367	-6.233	15	4	silty clay to clay
29.364	24.70	1.1776	4.767	-6.183	24	3	clay
29.528	27.52	1.3259	4.817	-6.116	26	3	clay
29.692	28.54	1.2615	4.420	-6.068	18	4	silty clay to clay
29.856	29.98	1.3268	4.426	-6.022	19	4	silty clay to clay
30.020	31.89	1.5288	4.793	-5.974	31	3	clay
30.184	36.80	1.7154	4.662	-5.876	23	4	silty clay to clay
30.348	38.38	1.5404	4.014	-5.934	18	5	clayey silt to silty clay
30.512	34.09	1.6051	4.709	-5.953	22	4	silty clay to clay
30.676	32.00	1.3445	4.202	-5.996	20	4	silty clay to clay
30.840	28.60	1.3962	4.882	-6.018	27	3	clay
31.004	41.26	1.8500	4.484	-5.231	26	4	silty clay to clay
31.168	35.48	1.7670	4.980	-5.394	34	3	clay
31.332	31.26	1.5585	4.985	-5.562	30	3	clay
31.496	31.77	1.4677	4.620	-5.665	20	4	silty clay to clay
31.660	39.57	1.6185	4.090	-5.749	25	4	silty clay to clay
31.824	48.49	1.8594	3.834	-5.864	23	5	clayey silt to silty clay
31.988	43.21	2.1482	4.971	-5.912	41	3	clay
32.152	39.75	1.7429	4.385	-6.001	25	4	silty clay to clay
32.316	37.08	1.4873	4.011	-6.066	24	4	silty clay to clay
32.480	48.82	1.5755	3.227	-6.056	23	5	clayey silt to silty clay
32.644	39.47	1.7802	4.510	-6.329	25	4	silty clay to clay
32.808	46.07	1.9971	4.335	-5.811	29	4	silty clay to clay
32.972	38.25	1.7696	4.626	-5.219	24	4	silty clay to clay
33.136	33.11	1.6427	4.961	-5.490	32	3	clay
33.301	29.95	1.5425	5.151	-5.588	29	3	clay

Depth ft	Tip Resistance (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	Friction Ratio (Fs/Qt) (%)	Pore Pressure (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
33.465	34.03	1.5136	4.448	-5.564	22	4	silty clay to clay
33.629	38.47	1.6104	4.186	-5.562	25	4	silty clay to clay
33.793	37.02	1.7094	4.618	-5.336	24	4	silty clay to clay
33.957	36.93	1.8351	4.969	-5.401	35	3	clay
34.121	36.26	1.7408	4.800	-5.449	35	3	clay
34.285	32.21	1.6517	5.128	-5.396	31	3	clay
34.449	39.05	1.6664	4.267	-5.387	25	4	silty clay to clay
34.613	38.86	1.8336	4.718	-5.430	25	4	silty clay to clay
34.777	35.93	1.6870	4.695	-5.384	23	4	silty clay to clay
34.941	37.61	1.6618	4.419	-5.358	24	4	silty clay to clay
35.105	37.83	1.7791	4.703	-5.341	24	4	silty clay to clay
35.269	45.69	2.0054	4.390	-5.296	29	4	silty clay to clay
35.433	47.97	2.3968	4.997	-5.353	31	4	silty clay to clay
35.597	52.67	2.4428	4.638	-5.418	34	4	silty clay to clay
35.761	41.20	2.1432	5.202	-5.413	39	3	clay
35.925	45.60	2.2128	4.853	-5.372	29	4	silty clay to clay
36.089	45.91	2.3019	5.014	-5.423	44	3	clay
36.253	55.16	2.6205	4.751	-5.377	35	4	silty clay to clay
36.417	52.85	2.7671	5.236	-5.538	51	3	clay
36.581	48.95	2.5406	5.191	-5.512	47	3	clay
36.745	45.66	2.3320	5.107	-5.629	44	3	clay
36.909	48.93	2.3828	4.870	-5.600	31	4	silty clay to clay
37.073	52.00	2.6022	5.005	-5.663	33	4	silty clay to clay
37.238	48.44	2.4228	5.002	-5.821	31	4	silty clay to clay
37.402	48.90	2.0650	4.223	-5.859	23	5	clayey silt to silty clay
37.566	66.92	2.6900	4.020	-5.372	32	5	clayey silt to silty clay
37.730	75.63	2.9843	3.946	-5.706	36	5	clayey silt to silty clay
37.894	51.59	3.1124	6.033	-5.847	49	3	clay
38.058	49.72	2.7042	5.438	-5.814	48	3	clay
38.222	44.70	2.4876	5.566	-5.926	43	3	clay
38.386	43.76	1.8252	4.171	-6.142	28	4	silty clay to clay
38.550	33.88	2.0005	5.904	-6.075	32	3	clay
38.714	38.21	2.0905	5.471	-5.970	37	3	clay
38.878	37.93	2.1771	5.740	-6.001	36	3	clay
39.042	36.93	2.3616	6.396	-6.001	35	3	clay
39.206	36.30	2.1113	5.817	-6.046	35	3	clay
39.370	29.83	1.9032	6.380	-6.126	29	3	clay
39.534	39.17	1.9864	5.071	-4.967	38	3	clay
39.698	52.41	2.7893	5.322	-5.272	50	3	clay
39.862	63.34	2.8962	4.573	-5.639	40	4	silty clay to clay
40.026	47.51	2.4505	5.157	-5.732	45	3	clay
40.190	35.10	1.9482	5.551	-5.883	34	3	clay
40.354	36.73	1.7875	4.866	-5.905	35	3	clay
40.518	38.27	1.8849	4.925	-5.866	37	3	clay
40.682	46.10	2.2673	4.918	-5.890	29	4	silty clay to clay
40.846	39.81	2.1526	5.408	-5.936	38	3	clay
41.011	40.51	2.0531	5.068	-5.900	39	3	clay
41.175	41.68	2.3212	5.570	-5.831	40	3	clay
41.339	47.89	2.5186	5.259	-5.756	46	3	clay
41.503	59.33	3.0174	5.086	-5.929	38	4	silty clay to clay
41.667	57.74	3.2992	5.714	-5.977	55	3	clay
41.831	58.78	3.3401	5.683	-6.027	56	11	very stiff fine grained (*)
41.995	54.84	3.2573	5.939	-6.039	53	3	clay

Depth ft	Tip Resistance (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	Friction Ratio (Fs/Qt) (%)	Pore Pressure (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
42.159	48.86	2.9584	6.055	-6.111	47	3	clay
42.323	41.94	2.4449	5.830	-6.140	40	3	clay
42.487	40.05	2.2706	5.669	-6.164	38	3	clay
42.651	35.64	2.1021	5.898	-6.142	34	3	clay
42.815	38.01	2.1335	5.613	-0.192	36	3	clay
42.979	41.78	2.0741	4.964	-6.181	40	3	clay
43.143	37.23	1.9312	5.187	-6.200	36	3	clay
43.307	50.60	2.6186	5.175	-6.114	48	3	clay
43.471	56.85	3.3952	5.972	-6.104	54	3	clay
43.635	65.17	3.3285	5.108	-6.272	62	11	very stiff fine grained (*)
43.799	53.80	3.4865	6.480	-6.346	52	3	clay
43.963	64.84	3.6279	5.595	-6.382	62	11	very stiff fine grained (*)
44.127	110.00	3.7541	3.413	-6.344	42	6	sandy silt to clayey silt
44.291	124.21	3.8303	3.084	-6.620	48	6	sandy silt to clayey silt
44.455	114.52	4.4296	3.868	-6.574	55	5	clayey silt to silty clay
44.619	115.60	4.6624	4.033	-6.572	111	11	very stiff fine grained (*)
44.783	69.78	4.2772	6.129	-6.797	67	11	very stiff fine grained (*)
44.948	52.95	3.3415	6.311	-6.787	51	3	clay
45.112	53.28	2.8016	5.258	-6.749	51	3	clay
45.276	42.54	2.6778	6.295	-6.665	41	3	clay
45.440	44.43	2.6281	5.915	-6.576	43	3	clay
45.604	42.90	2.5650	5.979	-6.675	41	3	clay
45.768	43.02	1.7467	4.060	-6.780	21	5	clayey silt to silty clay
45.932	40.39	1.7141	4.244	-6.840	26	4	silty clay to clay
46.096	40.93	1.7405	4.252	4.588	26	4	silty clay to clay
46.260	71.30	2.3154	3.247	10.563	27	6	sandy silt to clayey silt
46.424	92.27	3.7883	4.106	-5.020	44	5	clayey silt to silty clay
46.588	89.44	4.1175	4.604	-8.274	86	11	very stiff fine grained (*)
46.752	72.92	3.8977	5.345	-10.834	70	11	very stiff fine grained (*)
46.916	69.29	2.3169	3.344	-10.591	27	6	sandy silt to clayey silt
47.080	53.50	2.3739	4.437	-10.481	34	4	silty clay to clay
47.244	33.79	2.3309	6.898	-8.826	32	3	clay
47.408	34.25	2.2005	6.425	-8.963	33	3	clay
47.572	39.15	2.2055	5.633	-8.665	37	3	clay
47.736	47.02	2.3538	5.006	-8.617	45	3	clay
47.900	49.68	2.7791	5.594	-8.704	48	3	clay
48.064	48.76	3.0838	6.325	-8.898	47	3	clay
48.228	50.42	3.0007	5.952	-9.083	48	3	clay
48.392	41.53	2.9708	7.153	-9.270	40	3	clay
48.556	58.15	2.9274	5.034	9.301	37	4	silty clay to clay
48.720	65.88	3.2568	4.944	-9.301	63	11	very stiff fine grained (*)
48.885	74.23	4.4095	5.940	-9.351	71	11	very stiff fine grained (*)
49.049	79.81	3.4809	4.362	-9.207	38	5	clayey silt to silty clay
49.213	48.01	2.6395	5.497	-9.337	46	3	clay
49.377	39.98	2.2679	5.672	-9.291	38	3	clay
49.541	51.91	2.8470	5.485	-9.179	50	3	clay
49.705	76.48	3.8263	5.003	-8.699	73	11	very stiff fine grained (*)
49.869	65.46	3.8141	5.826	-8.735	63	11	very stiff fine grained (*)
50.033	61.34	3.6283	5.915	-8.548	59	11	very stiff fine grained (*)
50.197	102.04	3.7979	3.722	-8.457	39	6	sandy silt to clayey silt
50.361	139.06	3.6236	2.606	-8.697	44	7	silty sand to sandy silt
50.525	123.44	5.0911	4.124	-8.709	118	11	very stiff fine grained (*)
50.689	241.64	4.6434	1.922	-1.741	58	8	sand to silty sand

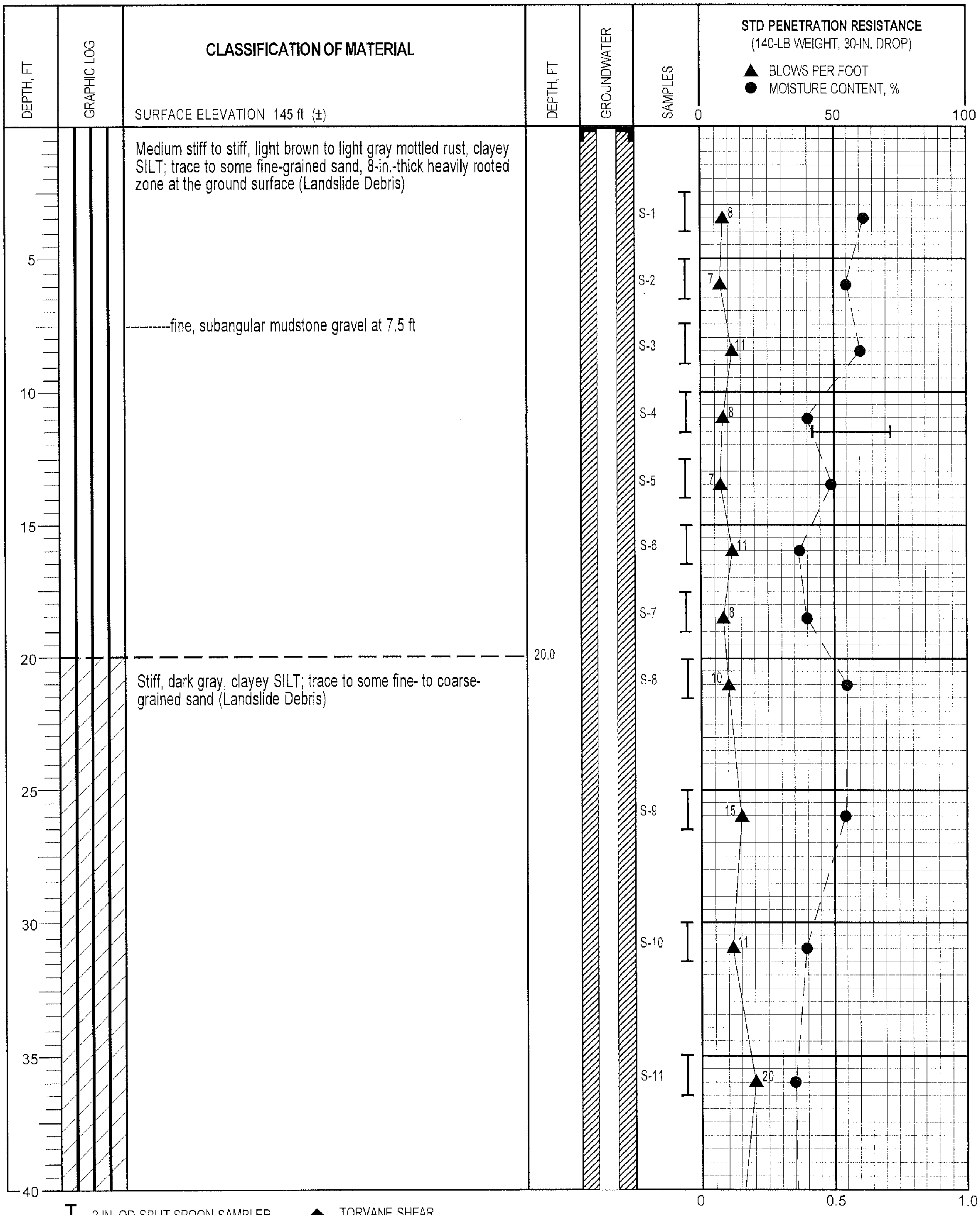
Depth ft	Tip Resistance (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	Friction Ratio (Fs/Qt) (%)	Pore Pressure (psi)	SPT N60 (UNITLESS)	Zone	Soil Behavior Type UBC-1983
50.853	212.62	5.8587	2.756	0.254	68	7	silty sand to sandy silt
51.017	225.14	7.8000	3.465	-6.610	108	12	sand to clayey sand (*)
51.181	236.29	7.5426	3.192	-9.215	113	12	sand to clayey sand (*)
51.345	262.62	6.5737	2.503	-9.248	84	7	silty sand to sandy silt
51.509	326.11	5.7184	1.753	-10.788	78	8	sand to silty sand
51.673	362.35	6.3668	1.757	-8.951	87	8	sand to silty sand
51.837	251.51	12.6827	5.043	-7.447	241	11	very stiff fine grained (*)
52.001	331.49	11.1962	3.378	-8.246	159	12	sand to clayey sand (*)
52.165	301.24	13.7817	4.575	-12.133	288	11	very stiff fine grained (*)
52.329	365.94	13.7736	3.764	-10.500	175	12	sand to clayey sand (*)
52.493	366.78	13.7736	3.755	-9.392	176	12	sand to clayey sand (*)

APPENDIX E

APPENDIX E

PAST EXPLORATIONS AND LABORATORY TESTING BY OTHERS

Past explorations completed at the site include nine borings (B-1, B-2, B-3/SI-3, B-4, B-5, B-6, B-7, SI-1, and SI-3) completed by Geocon in November 2012 and one boring (SI-4) and five test pits (TP-1 through TP-5) completed by GRI in May 2013. Laboratory testing associated with the prior explorations included Atterberg limits tests, grain-size tests (combined sieve and hydrometers), consolidation tests, direct shear tests, and residual torsional ring shear tests. The approximate locations of the explorations are shown on Figure 2 and the associated exploration logs and laboratory test results are presented in this appendix.



Medium stiff to stiff, light brown to light gray mottled rust, clayey SILT; trace to some fine-grained sand, 8-in.-thick heavily rooted zone at the ground surface (Landslide Debris)

----------fine, subangular mudstone gravel at 7.5 ft

Stiff, dark gray, clayey SILT; trace to some fine- to coarse-grained sand (Landslide Debris)

I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

Ⓟ VIBRATING-WIRE PIEZOMETER

SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

☐ PERCENT PASSING NO. 200 SIEVE (WASHED)

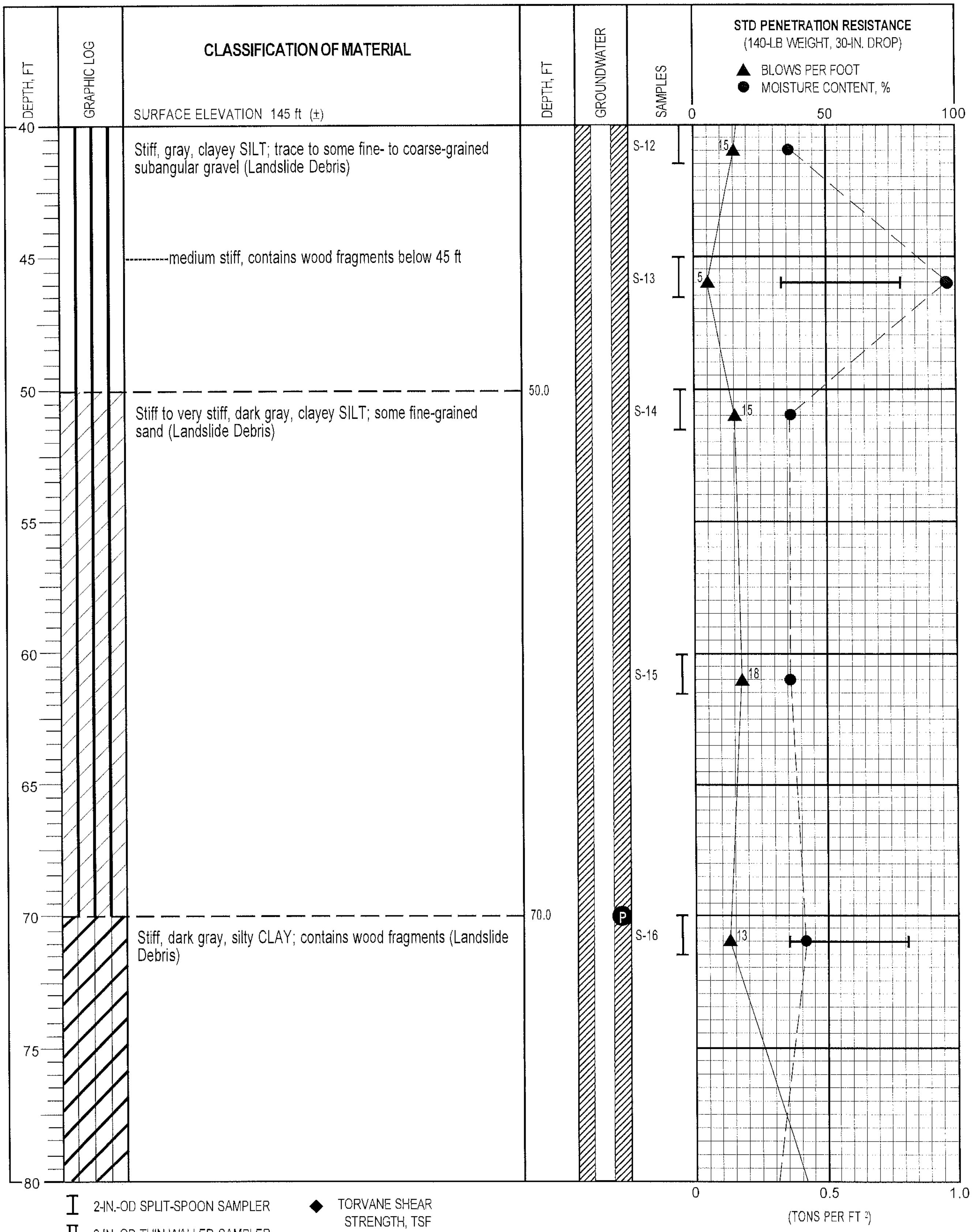
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING SI-4

MAY 2013 JOB. NO. 5422 FIG. 1A



Stiff, gray, clayey SILT; trace to some fine- to coarse-grained subangular gravel (Landslide Debris)

----------medium stiff, contains wood fragments below 45 ft

Stiff to very stiff, dark gray, clayey SILT; some fine-grained sand (Landslide Debris)

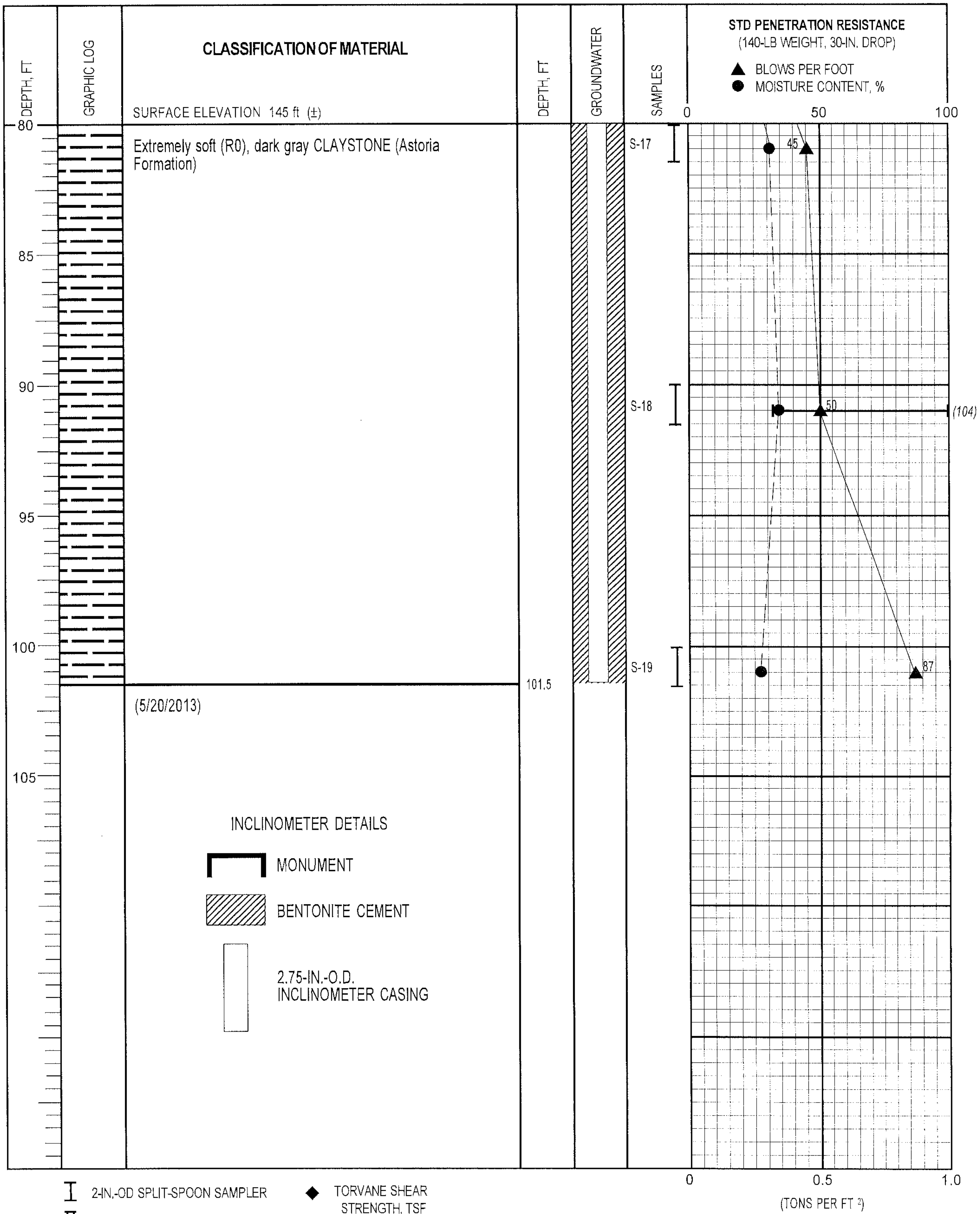
Stiff, dark gray, silty CLAY; contains wood fragments (Landslide Debris)

I 2-IN.-OD SPLIT-SPOON SAMPLER	◆ TORVANE SHEAR STRENGTH, TSF
II 3-IN.-OD THIN-WALLED SAMPLER	☐ PERCENT PASSING NO. 200 SIEVE (WASHED)
G GRAB SAMPLE OF DRILL CUTTINGS	* NO RECOVERY
Ⓟ VIBRATING-WIRE PIEZOMETER	
├── SLOTTED PVC PIPE	├─●─┤ Liquid Limit, Moisture Content, Plastic Limit
▼ Water Level (date)	

GRI

BORING SI-4 (cont.)

MAY 2013 JOB. NO. 5422 FIG. 1A



I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

Ⓟ VIBRATING-WIRE PIEZOMETER

|—SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

□ PERCENT PASSING NO. 200 SIEVE (WASHED)

* NO RECOVERY

Liquid Limit

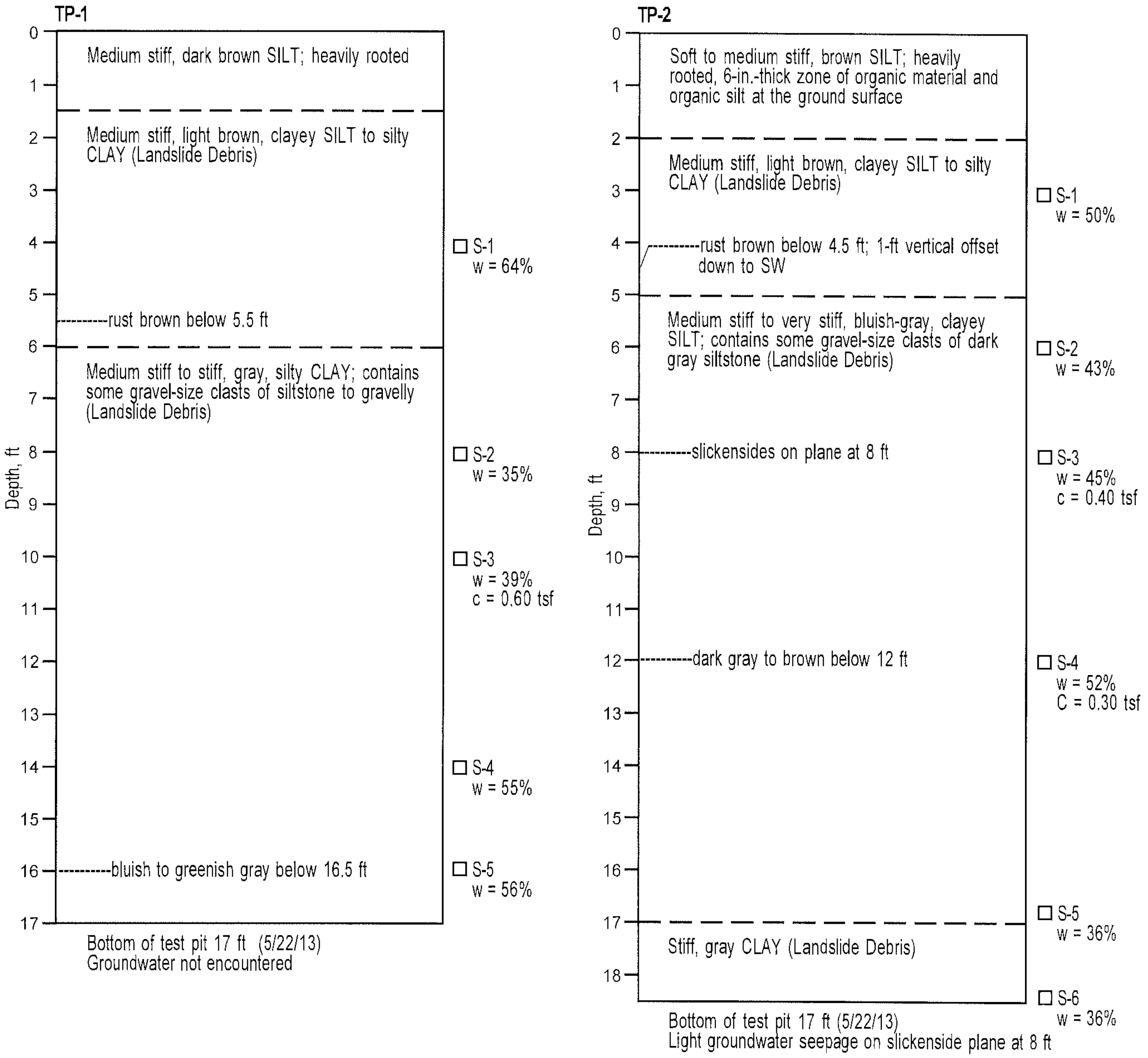
Moisture Content

Plastic Limit

GRI

BORING SI-4 (cont.)

MAY 2013 JOB. NO. 5422 FIG. 1A



LEGEND

☐ = GRAB SAMPLE

w = NATURAL MOISTURE CONTENT

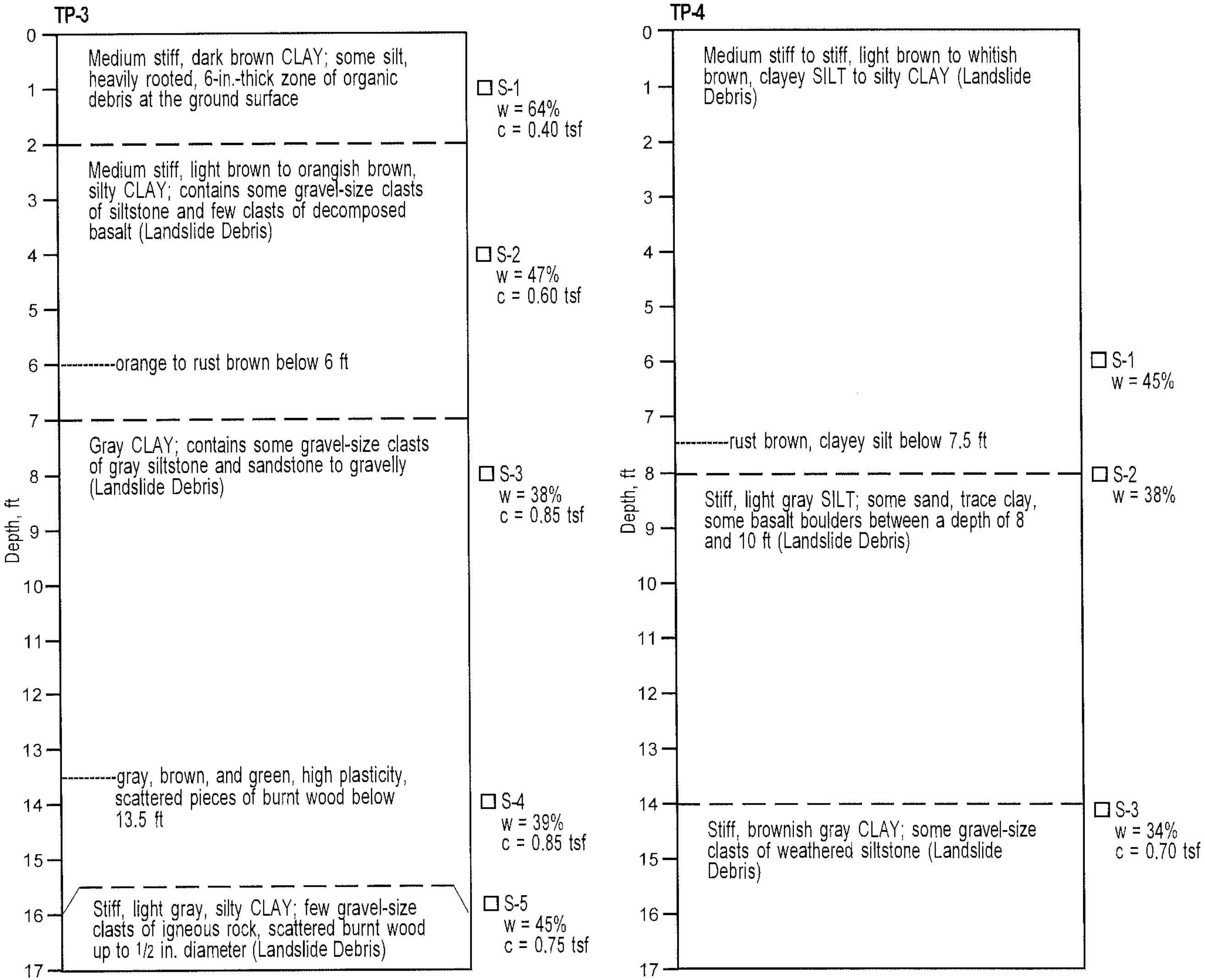
c = TORVANE SHEAR STRENGTH

GROUND SURFACE ELEVATIONS NOT AVAILABLE

GRI

TEST PIT LOGS

JUNE 2013 JOB NO. 5422 FIG. 2A



TP-3

Medium stiff, dark brown CLAY; some silt, heavily rooted, 6-in.-thick zone of organic debris at the ground surface

☐ S-1
w = 64%
c = 0.40 tsf

Medium stiff, light brown to orangish brown, silty CLAY; contains some gravel-size clasts of siltstone and few clasts of decomposed basalt (Landslide Debris)

☐ S-2
w = 47%
c = 0.60 tsf

----------orange to rust brown below 6 ft

Gray CLAY; contains some gravel-size clasts of gray siltstone and sandstone to gravelly (Landslide Debris)

☐ S-3
w = 38%
c = 0.85 tsf

----------gray, brown, and green, high plasticity, scattered pieces of burnt wood below 13.5 ft

☐ S-4
w = 39%
c = 0.85 tsf

Stiff, light gray, silty CLAY; few gravel-size clasts of igneous rock, scattered burnt wood up to 1/2 in. diameter (Landslide Debris)

☐ S-5
w = 45%
c = 0.75 tsf

Bottom of test pit 17 ft (5/22/13)
Groundwater not encountered

TP-4

Medium stiff to stiff, light brown to whitish brown, clayey SILT to silty CLAY (Landslide Debris)

☐ S-1
w = 45%

----------rust brown, clayey silt below 7.5 ft

Stiff, light gray SILT; some sand, trace clay, some basalt boulders between a depth of 8 and 10 ft (Landslide Debris)

☐ S-2
w = 38%

Stiff, brownish gray CLAY; some gravel-size clasts of weathered siltstone (Landslide Debris)

☐ S-3
w = 34%
c = 0.70 tsf

Practical refusal on boulder at 16.3 ft (5/22/13)
Groundwater seepage at 8 ft (< 1 gpm)

LEGEND

☐ = GRAB SAMPLE

w = NATURAL MOISTURE CONTENT

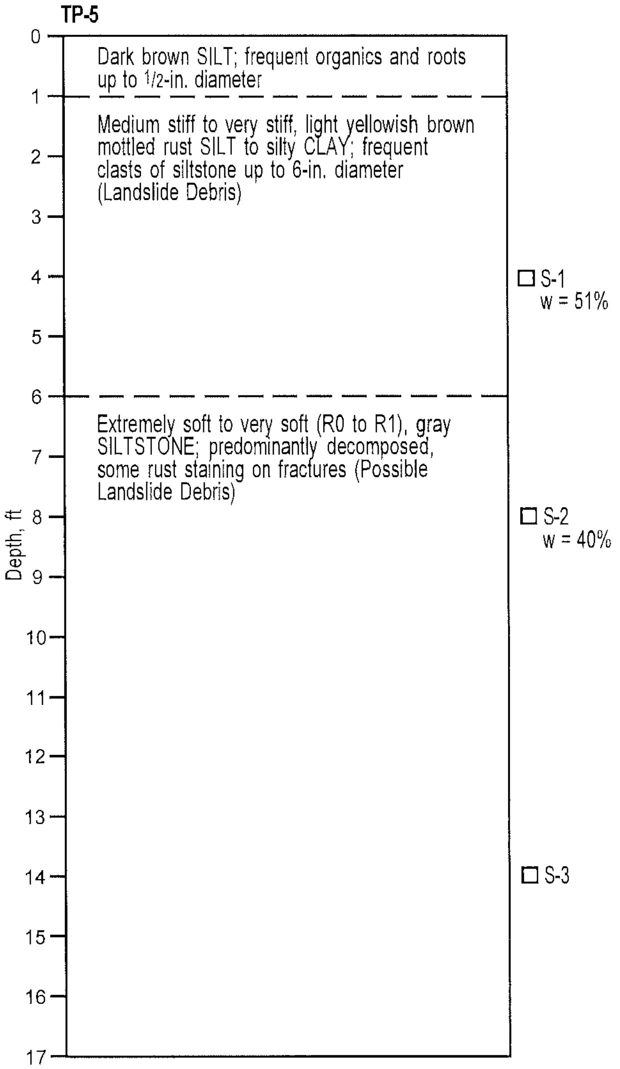
c = TORVANE SHEAR STRENGTH

GROUND SURFACE ELEVATIONS NOT AVAILABLE

GRI

TEST PIT LOGS

JUNE 2013 JOB NO. 5422 FIG. 3A



Dark brown SILT; frequent organics and roots up to 1/2-in. diameter

Medium stiff to very stiff, light yellowish brown mottled rust SILT to silty CLAY; frequent clasts of siltstone up to 6-in. diameter (Landslide Debris)

Extremely soft to very soft (R0 to R1), gray SILTSTONE; predominantly decomposed, some rust staining on fractures (Possible Landslide Debris)

Bottom of test pit 17 ft (5/22/13)
Light groundwater seepage at 16 ft

LEGEND

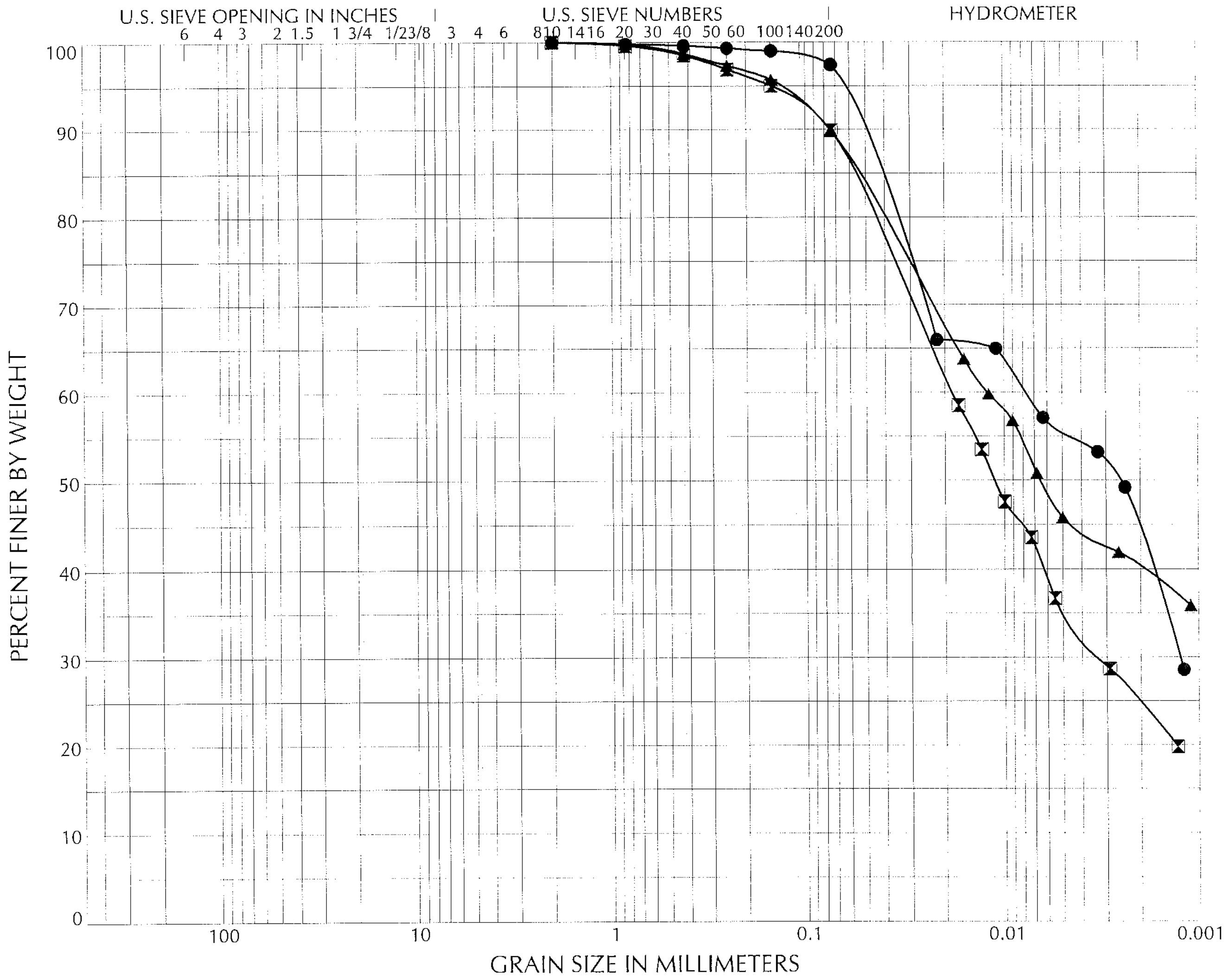
☐ = GRAB SAMPLE
w = NATURAL MOISTURE CONTENT
c = TORVANE SHEAR STRENGTH

GROUND SURFACE ELEVATIONS NOT AVAILABLE

GRI

TEST PIT LOGS

JUNE 2013 JOB NO. 5422 FIG. 4A



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

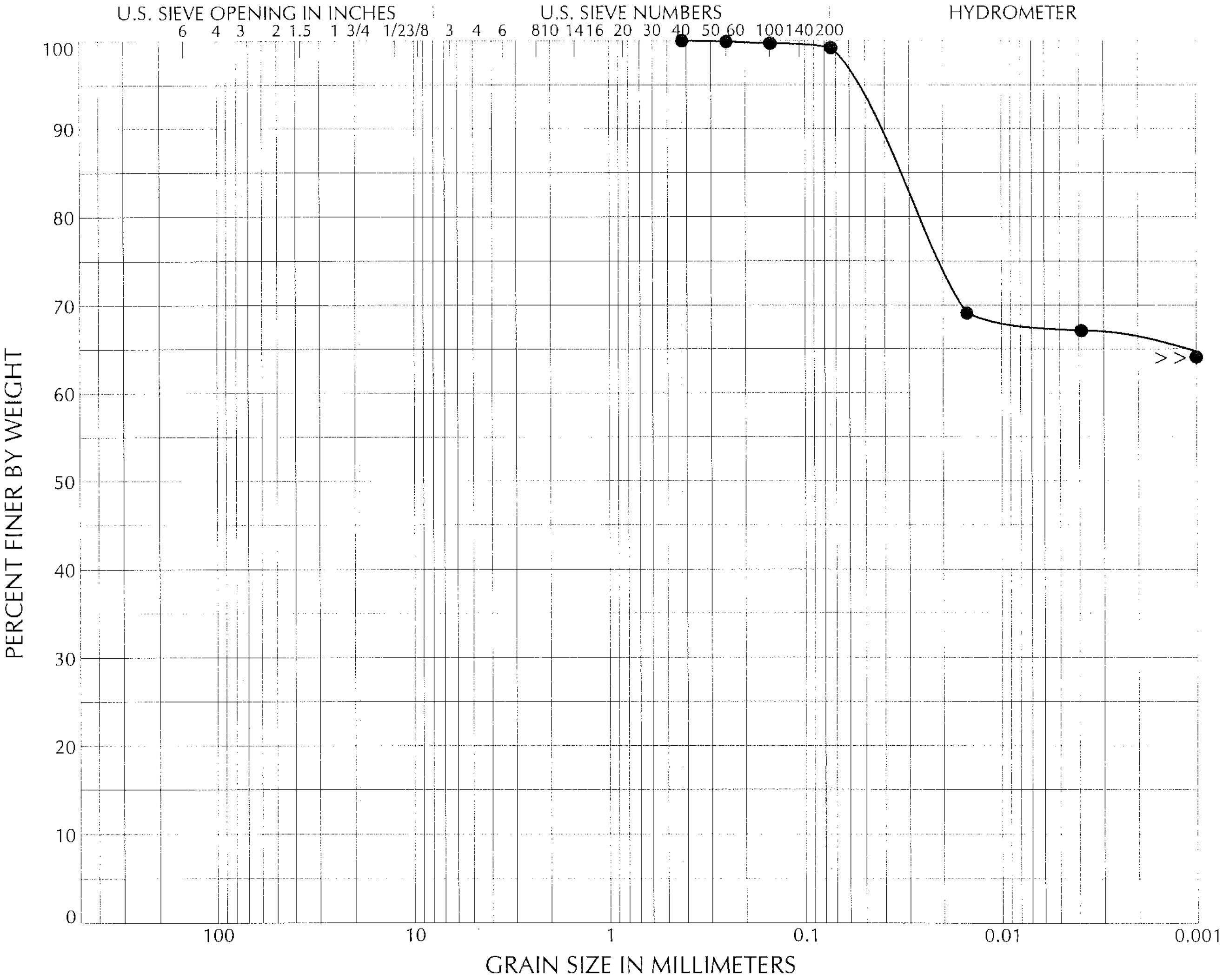
	Sample	Depth, ft	Classification	LL	PL	PI	Cc	Cu
●	SI-4	17.5	Clayey SILT; trace sand	72	42	30		
⧗	SI-4	45.0	Clayey SILT; trace sand	79	42	37		
▲	SI-4	70.0	Silty CLAY; trace sand	81	35	46		

	Sample	Depth, ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	SI-4	17.5	2	0.008	0.001		0.0	2.6	97.4	
⧗	SI-4	45.0	2	0.018	0.003		0.0	10.0	90.0	
▲	SI-4	70.0	2	0.012			0.0	10.0	90.0	

GRI

GRAIN SIZE DISTRIBUTION

JUNE 2013 PROJECT NO. 5422 FIG. 5A



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

	Sample	Depth, ft	Classification	LL	PL	PI	Cc	Cu
●	SI-4	90.0	CLAYSTONE	104	32	72		

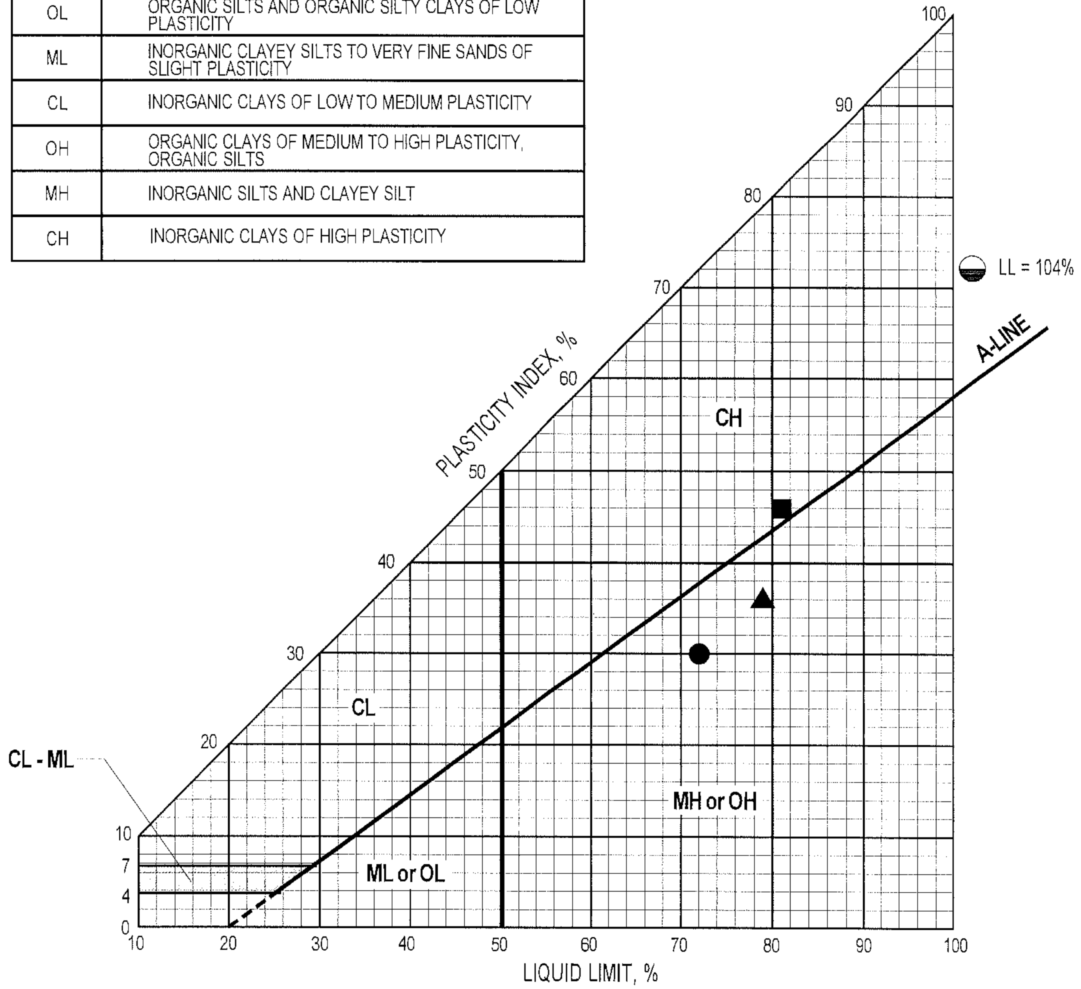
	Sample	Depth, ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	SI-4	90.0	0.42				0.0	0.8	99.2	

GRI

GRAIN SIZE DISTRIBUTION

JUNE 2013 PROJECT NO. 5422 FIG. 6A

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



SYMBOL	LOCATION	SAMPLE	MOISTURE CONTENT,%	SOIL DESCRIPTION
●	SI-4	S-7	40	DARK GRAY, CLAYEY SILT; TRACE FINE-GRAINED SAND
▲	SI-4	S-13	97	GRAY, CLAYEY SILT; CONTAINS WOOD FRAGMENTS/FIBERS
■	SI-4	S-16	42	DARK GRAY, SILTY CLAY; CONTAINS WOOD FRAGMENTS
◒	SI-4	S-18	34	DARK GRAY CLAYSTONE

GRI

PLASTICITY CHART

JUNE 2013　JOB NO. 5422　FIG. 7A

PROJECT NO. P1910-05-01

BORING B 1

ELEV. (MSL.)________ DATE COMPLETED **11-07-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					Asphalt Parking lot			
				ML	**ALLUVIUM** Soft, saturated, gray, SILT with sand			
2								
	B1-1					3		56.5
4								
	B1-2					7		59.3
				ML	Medium stiff, saturated, gray with reddish brown, SILT			
6								
	B1-3					0		129.3
8					-Very soft with wood			
	B1-4					3		169.2
				ML/CL	Soft, saturated, gray, Silty CLAY/Clayey SILT with minor organics			
10								
	B1-5				-With organics and decayed wood	3		237.1
12								
	B1-6				-No recovery, wood cuttings	0		
14								
	B1-7					3		68.2
16					-Very soft and gray, clayey silt to silty clay			
	B1-8					0		69.6
18					-Same			
	B1-9			GP	Dense, saturated, gray rounded GRAVEL with sandy site, silty sand matrix.	68		31.5
20					REFUSAL ON GRAVEL AT 20 FEET			

Figure A-1,
Log of Boring B 1, Page 1 of 1

P1910-05-01.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL

▨ ... DISTURBED OR BAG SAMPLE

◧ ... STANDARD PENETRATION TEST

◩ ... CHUNK SAMPLE

■ ... DRIVE SAMPLE (UNDISTURBED)

▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. P1910-05-01

BORING B 2

ELEV. (MSL.)______ DATE COMPLETED **11-08-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					ROAD and CULVERT FILL across drainage			
2								
4								
6								
8								
10	B2-1			ML	**COLLUVIUM** Medium stiff, wet, reddish brown with gray, Clayey SILT with sand and gravel	7		42.0
12								
	B2-2				-No recovery	9		
14								
	B2-3				-No recovery	8		
16								
				ML	**ASTORIA FORMATION** Stiff, wet, gray, Clayey SILT			
18	B2-4					12		36.8
20	B2-5					18		
22								
24								
	B2-6					14		42.3
26								
28								
				SM/ML	Medium dense, stiff, wet, gray and brown, Silty SAND to Sandy SILT with visible mica			

Figure A-2,
Log of Boring B 2, Page 1 of 2

P1910-05-01.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
▨ ... DISTURBED OR BAG SAMPLE
◧ ... STANDARD PENETRATION TEST
◩ ... CHUNK SAMPLE
■ ... DRIVE SAMPLE (UNDISTURBED)
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. P1910-05-01

BORING B 2

ELEV. (MSL.)_______ DATE COMPLETED **11-08-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B2-7			SM/ML	Medium dense to stiff, wet, gray with brown, Silty SAND to Sandy SILT with visible mica	27		41.8
32								
34								
	B2-8					27		33.7
36								
38								
40	B2-9					35		32.3
					BORING TERMINATED AT 41.5 FEET			

Figure A-2,
Log of Boring B 2, Page 2 of 2

P1910-05-01.GPJ

SAMPLE SYMBOLS
- ☐ ... SAMPLING UNSUCCESSFUL
- ▨ ... DISTURBED OR BAG SAMPLE
- ◧ ... STANDARD PENETRATION TEST
- ◣ ... CHUNK SAMPLE
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. P1910-05-01

BORING B 3/SI 3

ELEV. (MSL.)______ DATE COMPLETED **11-09-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					6" GRAVEL ROAD			
				GP	**FILL/DOZER TRAIL** Soft, wet, reddish brown wit yellow and gray, CLAY with sand, some gravel and wood			
2	B3-1				-No recovery, gravel and wood in shoe	7		
4	B3-2					5		67.8
6								
8	B3-3			CL	**COLLUVIUM** Soft, wet, reddish brown, gray and yellow CLAY with sand and some gravel to coarse sand with clay and some gravel	4		68.7
10	B3-4				-No recovery, becomes medium stiff	7		
12	B3-5					7		72.7
14								
	B3-6			SM	Soft to loose, wet, reddish brown, yellow and gray, coarse SAND with silt, clay and some gravel	3		65.1
16								
18	B3-7				-Stiffer/denser with less silt and clay	16		56.9
20	B3-8				-Becomes softer, looser	5		72.5
22	B3-9					8		68.5
24					-Becomes stiffer, approximately 8" recovery, yellow, clayey silt with 1" single gray gravel			
	B3-10			ML/CL	**ASTORIA FORMATION** Stiff, wet, yellowish brown, SILT with clay	14		32.2
26								
28	B3-11					14		30.6

P1910-05-01.GPJ

Figure A-3,
Log of Boring B 3/SI 3, Page 1 of 4

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL

▨ ... DISTURBED OR BAG SAMPLE

◨ ... STANDARD PENETRATION TEST

◪ ... CHUNK SAMPLE

■ ... DRIVE SAMPLE (UNDISTURBED)

▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. P1910-05-01

BORING B 3/SI 3

ELEV. (MSL.)______ DATE COMPLETED **11-09-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B3-12			ML/CL		14		30.2
					-Becomes gray			
32	B3-13					21		29.8
34	B3-14					13		30.9
36								
38				CL	Stiff, wet, gray, Silty CLAY			
40	B3-15					21		32.8
42								
44	B3-16					26		34.7
46								
48								
50	B3-17					35		30.9
52								
54								
56								
58								

Figure A-3,
Log of Boring B 3/SI 3, Page 2 of 4

P1910-05-01.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
◧ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
▨ ... DISTURBED OR BAG SAMPLE
◩ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. P1910-05-01

BORING B 3/SI 3

ELEV. (MSL.)______ DATE COMPLETED **11-09-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	B3-18			CL	Stiff, wet, gray, CLAY	75	95.3	29.0
62								
64								
66								
68								
70	B3-19					28		34.5
72								
74								
76								
78								
80	B3-20				-Very hard, wet, gray clay	76	78.7	45.5
82								
84								
86								
88								

P1910-05-01.GPJ

Figure A-3,
Log of Boring B 3/SI 3, Page 3 of 4

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL

▨ ... DISTURBED OR BAG SAMPLE

◧ ... STANDARD PENETRATION TEST

◪ ... CHUNK SAMPLE

■ ... DRIVE SAMPLE (UNDISTURBED)

▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. P1910-05-01

BORING B 3/SI 3

ELEV. (MSL.)_______ DATE COMPLETED **11-09-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90	B3-21 ■	▨		CL	-Becomes very hard, wet, gray, clay	95		45.5
92		▨						
94		▨						
96		▨						
98		▨						
100					BORING TERMINATED AT 100 FEET Install inclinometer to 100 feet Install vibrating wire piezometer at 25 feet, 55 feet, 85 feet			

Figure A-3,
Log of Boring B 3/SI 3, Page 4 of 4

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◣ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. P1910-05-01

BORING B 4

ELEV. (MSL.)______ DATE COMPLETED **11-12-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					12" GRAVEL ROAD			
				SM	**COLLUVIUM** Loose, wet, brown and gray, coarse SAND with gravel, silt and clay and boulders			
2	B4-1					8		
4								
	B4-2				-20" basalt boulder	50/0"		
6				ML/MH	Stiff, moist, light brown, SILT with fine sand and silt			
8	B4-3					13		40.0
10	B4-4				-Becomes medium stiff, reddish brown, visible mica	5		62.4
12	B4-5					14		62.4
14					-No recovery			
	B4-6				-Becomes silt	10		43.5
16								
18	B4-7					10		34.4
20	B4-8					16		35.4
22								
24				CL	**ASTORIA FORMATION** Very stiff, gray to dark gray, CLAY -Very stiff			
	B4-9					26		32.1
26								
28								

Figure A-4,
Log of Boring B 4, Page 1 of 2

P1910-05-01.GPJ

SAMPLE SYMBOLS	☐ ... SAMPLING UNSUCCESSFUL	◨ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	▨ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. P1910-05-01

BORING B 4

ELEV. (MSL.)________ DATE COMPLETED **11-12-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B4-10			ML/MH	-Becomes stiff and gray, mica visible with some fine gravel	12		23.4
32								
34				CL	Stiff, moist to wet, gray, CLAY with silt			
	B4-11					17		31.7
36								
38								
40	B4-12					9		37.4
42								
44								
	B4-13				-Becomes more granular	26		37.9
46								
48								
50	B4-14					31		31.4
					BORING TERMINATED AT 51.5 FEET			

P1910-05-01.GPJ

Figure A-4, Log of Boring B 4, Page 2 of 2

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL

▨ ... DISTURBED OR BAG SAMPLE

◧ ... STANDARD PENETRATION TEST

◪ ... CHUNK SAMPLE

■ ... DRIVE SAMPLE (UNDISTURBED)

▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. P1910-05-01

BORING B 5

ELEV. (MSL.)_______ DATE COMPLETED **11 -12 -2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					6" GRAVEL ROAD			
				CL/CH	**COLLUVIUM** Soft, wet, dark gray, CLAY with 3" woody organic layer			
2								
4								
	B5-1					7		78.6
6								
8								
				MH	Very soft, wet, light gray and light brown, Clayey SILT with some fine sand			
10	B5-2					0		55.3
12								
				ML/MH	Stiff, wet, light brown/gray/reddish brown, SILT with clay			
14								
	B5-3					12		48.9
16								
18								
				CH	Very soft, wet, gray, Silty CLAY			
20	B5-4					0		57.4
22								
				CL	**ASTORIA FORMATION** Stiff, wet, light gray to dark gray, CLAY			
24								
	B5-5					12		54.8
26								
28								

Figure A-5,
Log of Boring B 5, Page 1 of 2

P1910-05-01.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL
▨ ... DISTURBED OR BAG SAMPLE
◨ ... STANDARD PENETRATION TEST
◪ ... CHUNK SAMPLE
■ ... DRIVE SAMPLE (UNDISTURBED)
▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. P1910-05-01

BORING B 5

ELEV. (MSL.)_______ DATE COMPLETED **11 -12 -2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B5-6	▨		CL	-Becomes hard	34		38.4
					BORING TERMINATED AT 31.5 FEET			

Figure A-5,
Log of Boring B 5, Page 2 of 2

P1910-05-01.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	▩ ... DISTURBED OR BAG SAMPLE	◣ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. P1910-05-01

BORING B 6

ELEV. (MSL.)________ DATE COMPLETED 11-12-2012

EQUIPMENT CME 75 TRACK w/MUD BY: S. DIXON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					6" GRAVEL ROAD			
				CL-ML	**COLLUVIUM** Stiff, wet, light brown with gray, Clayey SILT to Silty CLAY			
2								
4								
	B6-1					9		
6								
8								
10	B6-2				-Becomes gray	9		58.5
12								
14								
	B6-3				-Becomes soft	3		51.8
16								
18				CL	**ASTORIA FORMATION** Stiff, wet, gray, CLAY			
20	B6-4					12		41.0
22								
24								
	B6-5				-Becomes very stiff	23		36.0
26								
					BORING TERMINATED AT 26.5 FEET			

P1910-05-01.GPJ

Figure A-6,
Log of Boring B 6, Page 1 of 1

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	▩ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. P1910-05-01

BORING B 7

ELEV. (MSL.)_______ DATE COMPLETED **11-16-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				ML	**COLLUVIUM** Hard, moist, reddish brown, Sandy SILT with clay			
2								
4								
6								
8								
10	B7-1					25	82.0	38.8
12								
14								
16								
18								
20	B7-2				-Becomes gray	15	81.1	42.7
					BORING TERMINATED AT 21.5 FEET			

P1910-05-01.GPJ

Figure A-7,
Log of Boring B 7, Page 1 of 1

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	▩ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. P1910-05-01

BORING SI 1

ELEV. (MSL.)_______ DATE COMPLETED **11-06-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				CL	GRASS SURFACE			
2					**COLLUVIUM**			
	SI1-1				Medium stiff, wet, light reddish brown with light and dark gray, CLAY	6		56
4								
	SI1-2				-Same with 2" decayed wood and organics in shoe	5		103.9
6								
8	SI1-3					1		125.3
					-Becomes soft with 2'x1" layers of wood/roots			
10	SI1-4				-Becomes medium stiff without wood	6		66.4
12								
14								
	SI1-5				-Becomes stiff, gray	9		36.2
16								
18								
					ASTORIA FORMATION			
20	SI1-6				-Becomes hard	40		37.7
22					Hard, wet, gray, CLAY			
24								
	SI1-7					47		37.2
26								
28								

Figure A-9,
Log of Boring SI 1, Page 1 of 3

P1910-05-01.GPJ

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
▩ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. P1910-05-01

BORING SI 1

ELEV. (MSL.)_______ DATE COMPLETED **11-06-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	SI1-8	▨		CL	Hard, wet, gray, CLAY	33		29.7
32		▨						
34		▨						
36		▨						
38		▨						
40	SI1-9	▨				50		28.2
42		▨						
44		▨						
46		▨						
48		▨						
50	S1-10	▨				65		29.3
52		▨						
54		▨						
56		▨						
58		▨						

Figure A-9,
Log of Boring SI 1, Page 2 of 3

P1910-05-01.GPJ

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ▨ ... DISTURBED OR BAG SAMPLE
- ◧ ... STANDARD PENETRATION TEST
- ◪ ... CHUNK SAMPLE
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. P1910-05-01

BORING SI 1

ELEV. (MSL.)_______ DATE COMPLETED **11-06-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	SI1-11			CL	Hard, wet, gray, CLAY	38		33.8
62								
64								
66								
68								
70	SI1-11					50		30.9
72								
74								
76								
78								
80	SI1-12					65		32.3

BORING TERMINATED AT 81.5 FEET
Vibrating wire piezometer at 25 feet and 55 feet

Figure A-9,
Log of Boring SI 1, Page 3 of 3

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	◨ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	⊠ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. P1910-05-01

BORING SI 2

ELEV. (MSL.)______ DATE COMPLETED **11-13-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					18" GRAVEL ROAD			
2				ML	**COLLUVIUM** Medium stiff, moist, light reddish brown, Clayey SILT with sand and gravel			
4								
6								
8								
10	SI2-1			CL	Medium stiff, moist to wet, dark gray, CLAY with sand and some gravel	6		47.3
12								
14								
16	SI2-2				-Becomes stiff without gravel	18		33.8
18								
20	SI2-3					18		31.6
22								
24				CL	**ASTORIA FORMATION** Very stiff to hard, wet, gray, CLAY			
26	SI2-4					20		31.4
28								

Figure A-10,
Log of Boring SI 2, Page 1 of 4

P1910-05-01.GPJ

SAMPLE SYMBOLS	☐ ... SAMPLING UNSUCCESSFUL	◨ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	▩ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. P1910-05-01

BORING SI 2

ELEV. (MSL.)______ DATE COMPLETED **11-13-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	SI2-5			CL		12		35.1
32					-Stiff, wet, gray, CLAY			
34								
	SI2-6					30		32.2
36								
					-Very stiff to hard			
38								
40	SI2-7					30		34.0
42								
44								
	SI2-8					29		35.5
46								
48								
50	SI2-9					25		35.5
52								
	SI2-10					33		31.8
54								
	SI2-11					30		32.3
56								
58	SI2-12					34		34.2

Figure A-10,
Log of Boring SI 2, Page 2 of 4

P1910-05-01.GPJ

SAMPLE SYMBOLS	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	▨ ... DISTURBED OR BAG SAMPLE	◩ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. P1910-05-01

BORING SI 2

ELEV. (MSL.)_______ DATE COMPLETED 11-13-2012

EQUIPMENT CME 75 TRACK w/MUD BY: S. DIXON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60	SI2-13				-Continued 11-14-2012	28		37.6
62								
	SI2-14					39		32.6
				CL	Very stiff to hard, wet, gray, CLAY			
64								
	SI2-15					28		33.3
66								
68	SI2-16					35		28.3
					-Becomes hard			
70	SI2-17					44		29.2
72								
74								
	SI2-18					36		28.3
76								
78								
80	SI2-19			CL		35		33.3
82								
84								
86								
88								

Figure A-10,
Log of Boring SI 2, Page 3 of 4

P1910-05-01.GPJ

SAMPLE SYMBOLS	☐ ... SAMPLING UNSUCCESSFUL	◧ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	▩ ... DISTURBED OR BAG SAMPLE	◪ ... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. P1910-05-01

BORING SI 2

ELEV. (MSL.)_______ DATE COMPLETED **11-13-2012**

EQUIPMENT **CME 75 TRACK w/MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
90	SI2-20	▨				51		25.3
92		▨		CL	Hard to very hard, wet, gray, CLAY			
94		▨						
96		▨						
98		▨						
100					BORING TERMINATED AT 100 FEET Install inclinometer Install vibrating wire piezometer at 25 feet, 55 feet, 85 feet			

Figure A-10,
Log of Boring SI 2, Page 4 of 4

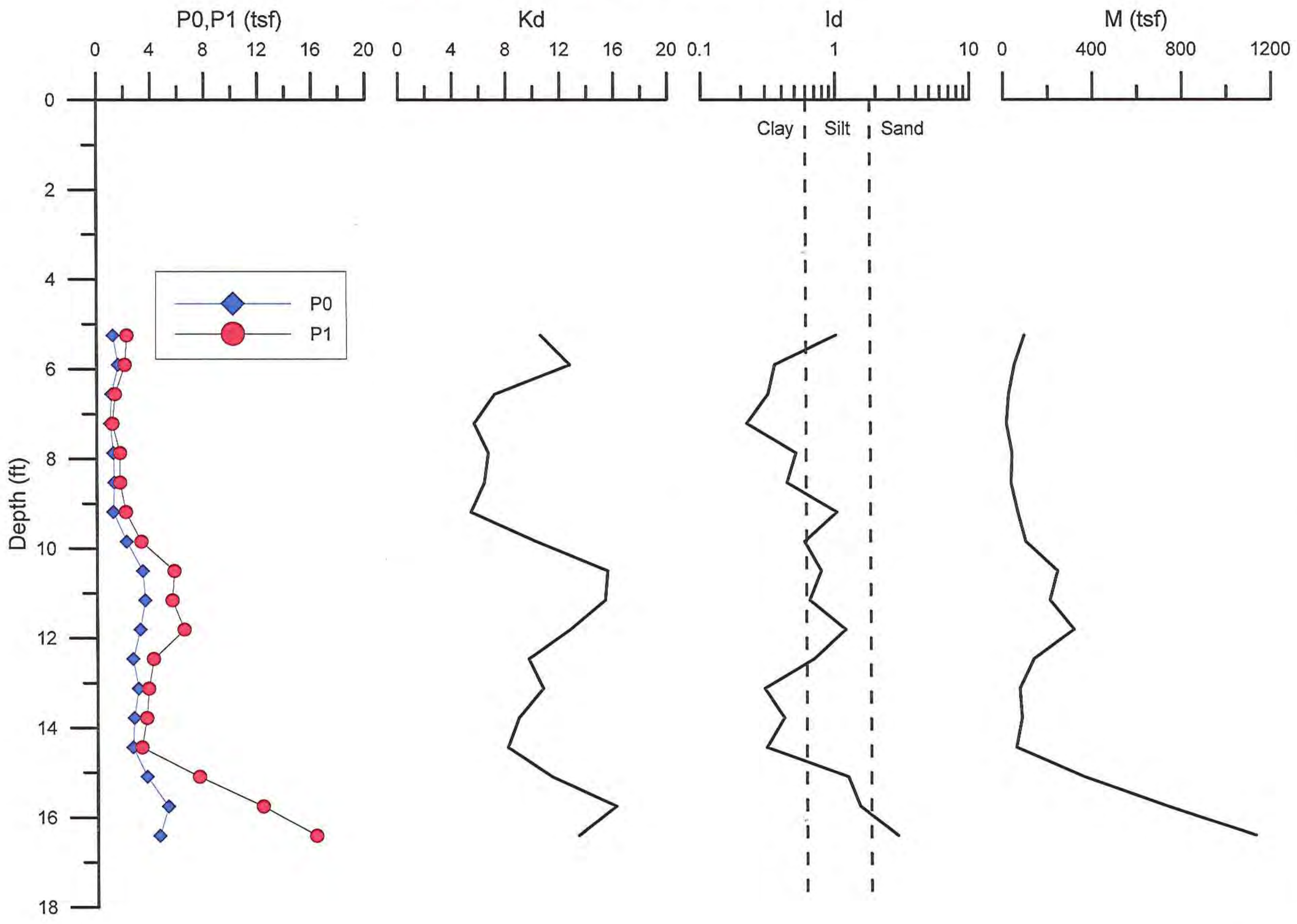
P1910-05-01.GPJ

SAMPLE SYMBOLS
☐ ... SAMPLING UNSUCCESSFUL
▥ ... STANDARD PENETRATION TEST
■ ... DRIVE SAMPLE (UNDISTURBED)
▩ ... DISTURBED OR BAG SAMPLE
◣ ... CHUNK SAMPLE
▼ ... WATER TABLE OR SEEPAGE

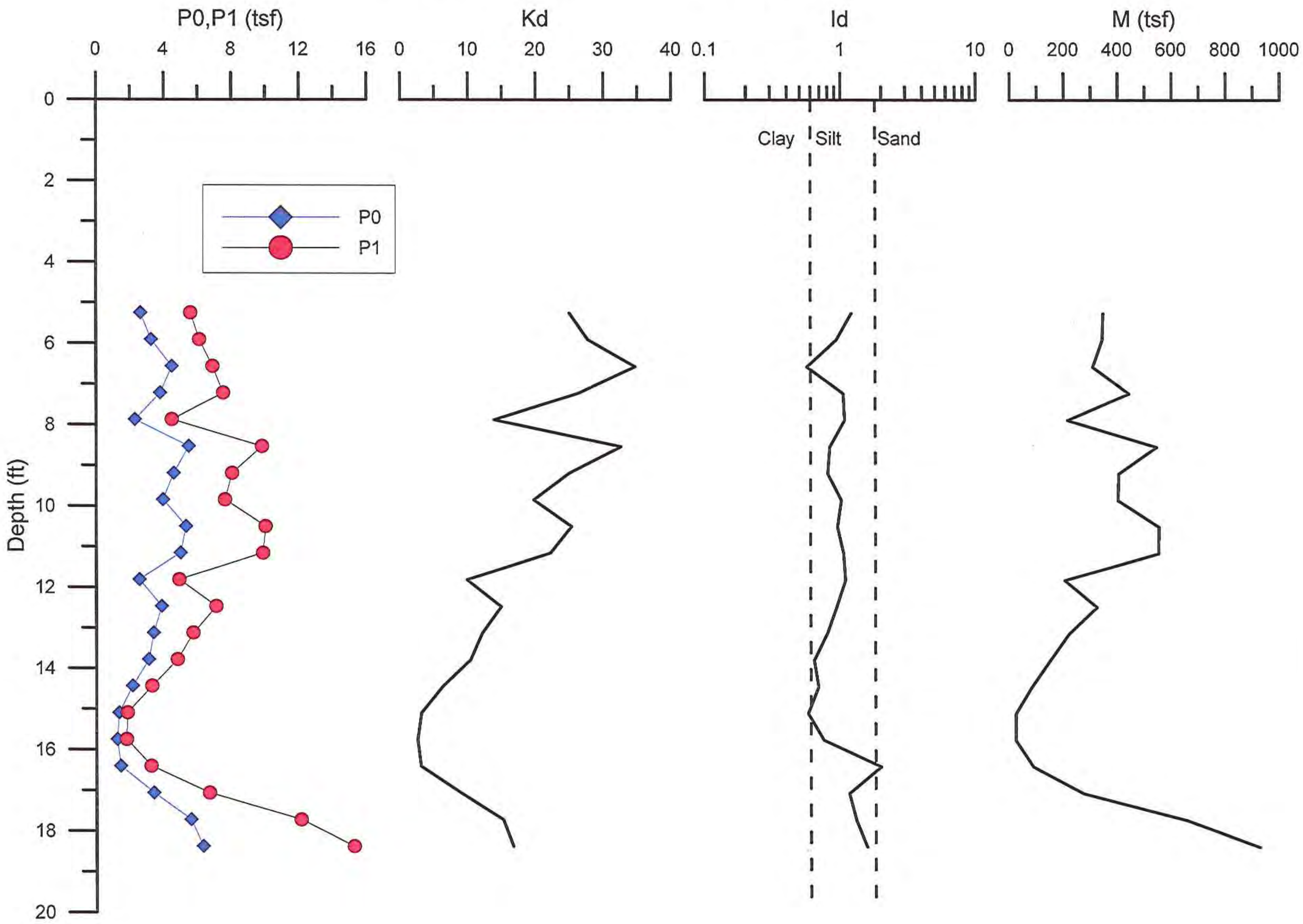
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

Seaside Campus Dilatometer Sounding 1



Seaside Campus Dilatometer Sounding 2



APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected soil samples were tested for their in situ moisture content, plasticity index, grain size distribution, compressibility and shear strength. Moisture contents from the borings are indicated on the boring logs in Appendix A. The results of the remaining laboratory tests performed are summarized in the following tables and graphs. Appendix C presents the results of ring shear residual strength testing.

TABLE B-1
SUMMARY OF PLASTICITY INDEX TEST RESULTS
ASTM D4318

Sample Number	*Depth (ft)*	*Liquid Limit*	*Plastic Limit*	*Plasticity Index*	*USCS Classification*
B1-4	8.5	191	78	113	MH
B1-8	16.5	65	33	32	MH/CH
B2-1	10	55	44	11	MH
B3-13	32.5	41	31	10	ML
B3-15	40	58	24	34	CH
B4-9	25	52	22	30	CH
B4-12	40	70	21	49	CH
B5-2	10	59	41	18	MH
B5-4	20	53	33	20	MH
B5-5	25	78	31	47	CH
B6-4	20	65	34	31	MH/CH
SI 1-3	7.5	91	40	51	MH/CH
SI 1-11	60	103	26	77	CH
SI 2-1	10	66	27	39	CH

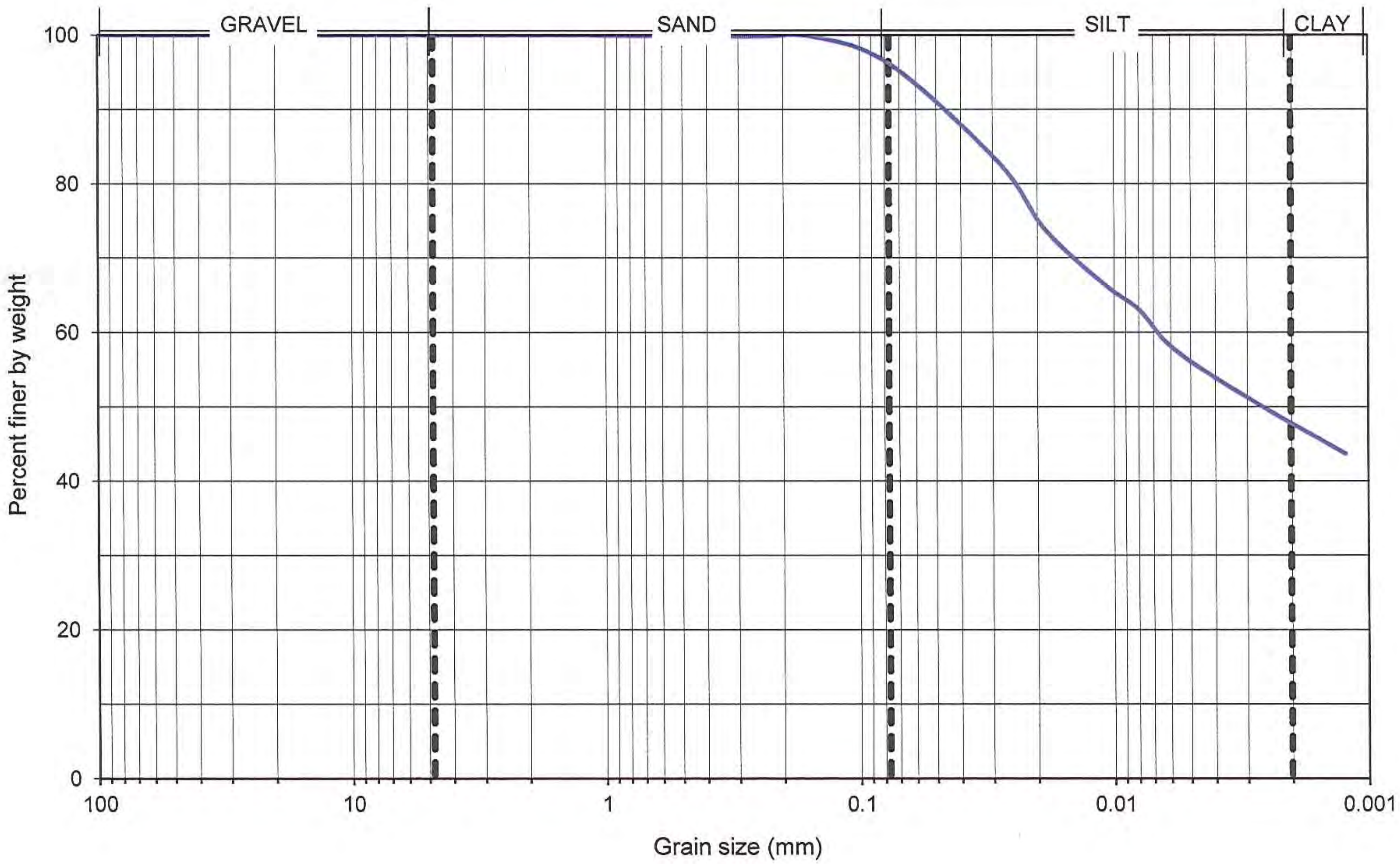
TABLE B-1
SUMMARY OF PLASTICITY INDEX TEST RESULTS
ASTM D4318

Sample Number	***Depth (ft)***	***Liquid Limit***	***Plastic Limit***	***Plasticity Index***	***USCS Classification***
SI 2-3	20	54	20	34	CH
SI 2-5	30	60	20	40	CH
SI 2-7	40	65	23	42	CH
SI 2-11	55	72	22	50	CH
SI 2-14	62.5	70	23	47	CH
SI 2-18	75	69	23	46	CH
SI 2-19	80	79	23	56	CH
SI 2-20	90	71	20	51	CH
SI 3-6	15	56	42	14	MH
SI 3-10	25	36	30	6	ML
SI 3-17	50	50	26	24	CL/CH
SI 3-19	70	76	27	49	CH
SI 3-20	90	88	24	64	CH

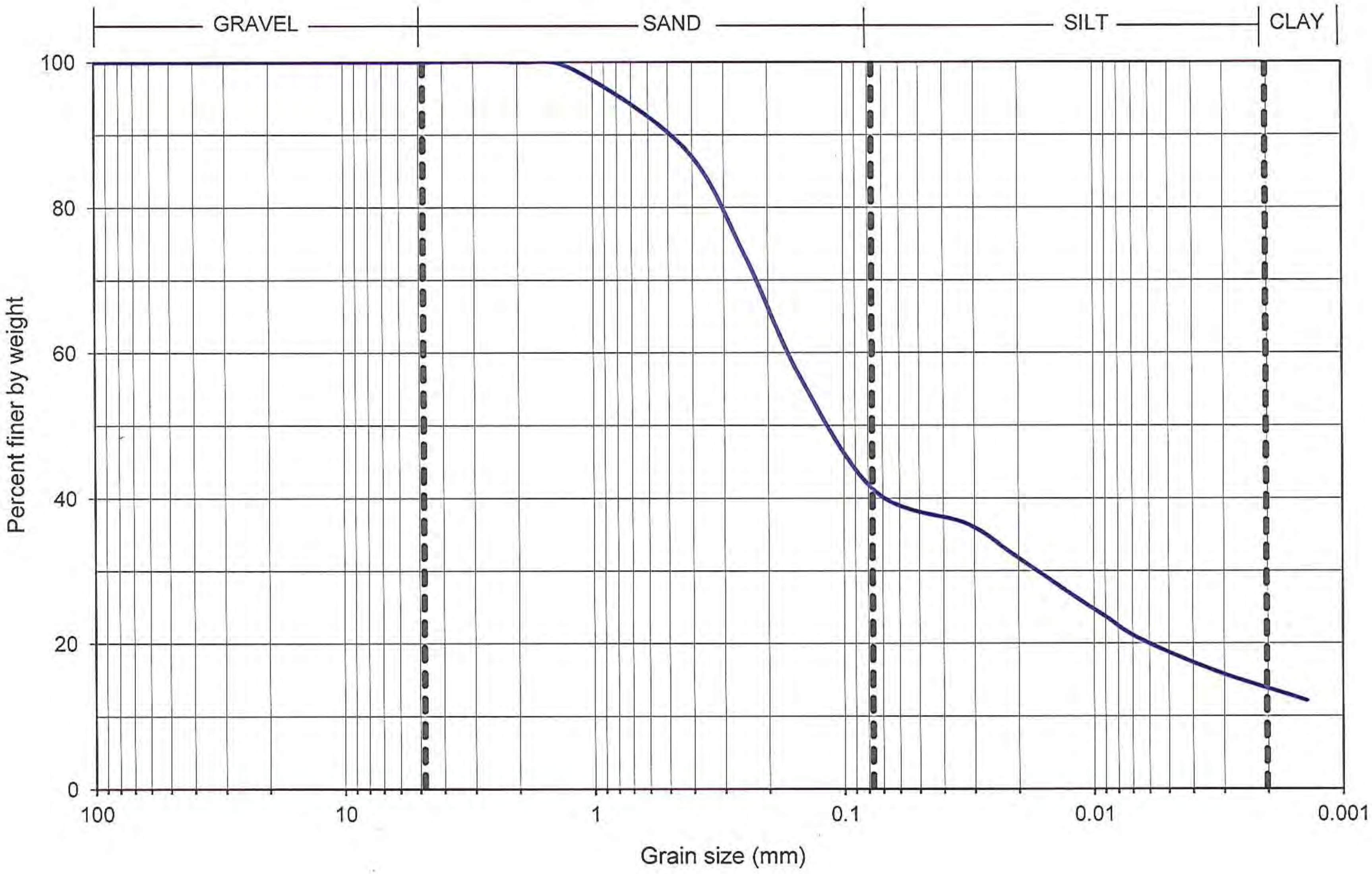
TABLE B-2
SUMMARY OF SPECIFIC GRAVITY TEST RESULTS
AASHTO T100

Sample Number	*Depth (ft)*	*Specific Gravity*
SI 3-18	60	2.80
B 7-1	10	2.71

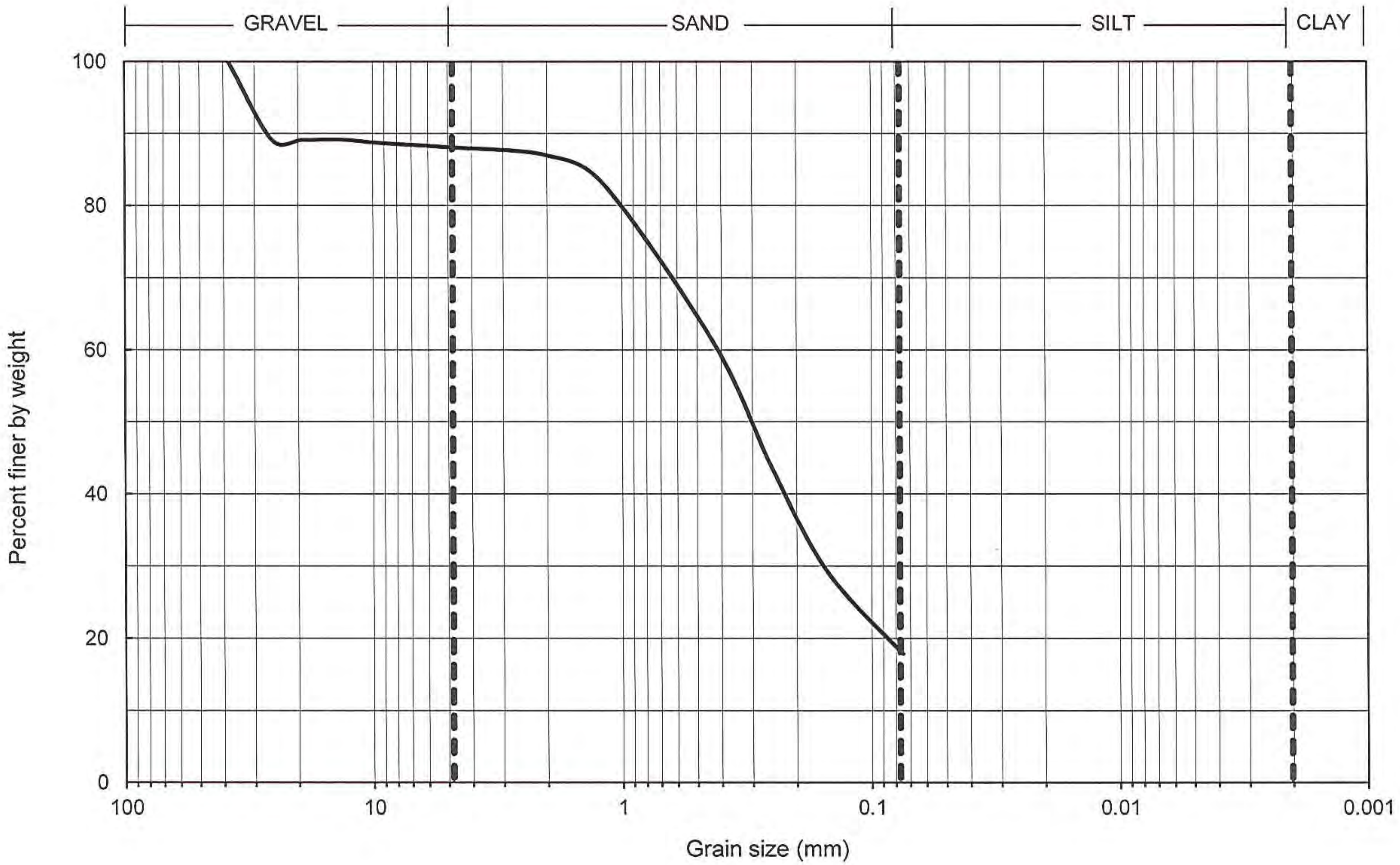
GRAIN SIZE DISTRIBUTION
Seaside Campus
P1910-05-01
Sample: B2-5 Depth = 30 feet



GRAIN SIZE DISTRIBUTION
Seaside Campus
P1910-05-01
Sample: B3-6 Depth = 15 feet

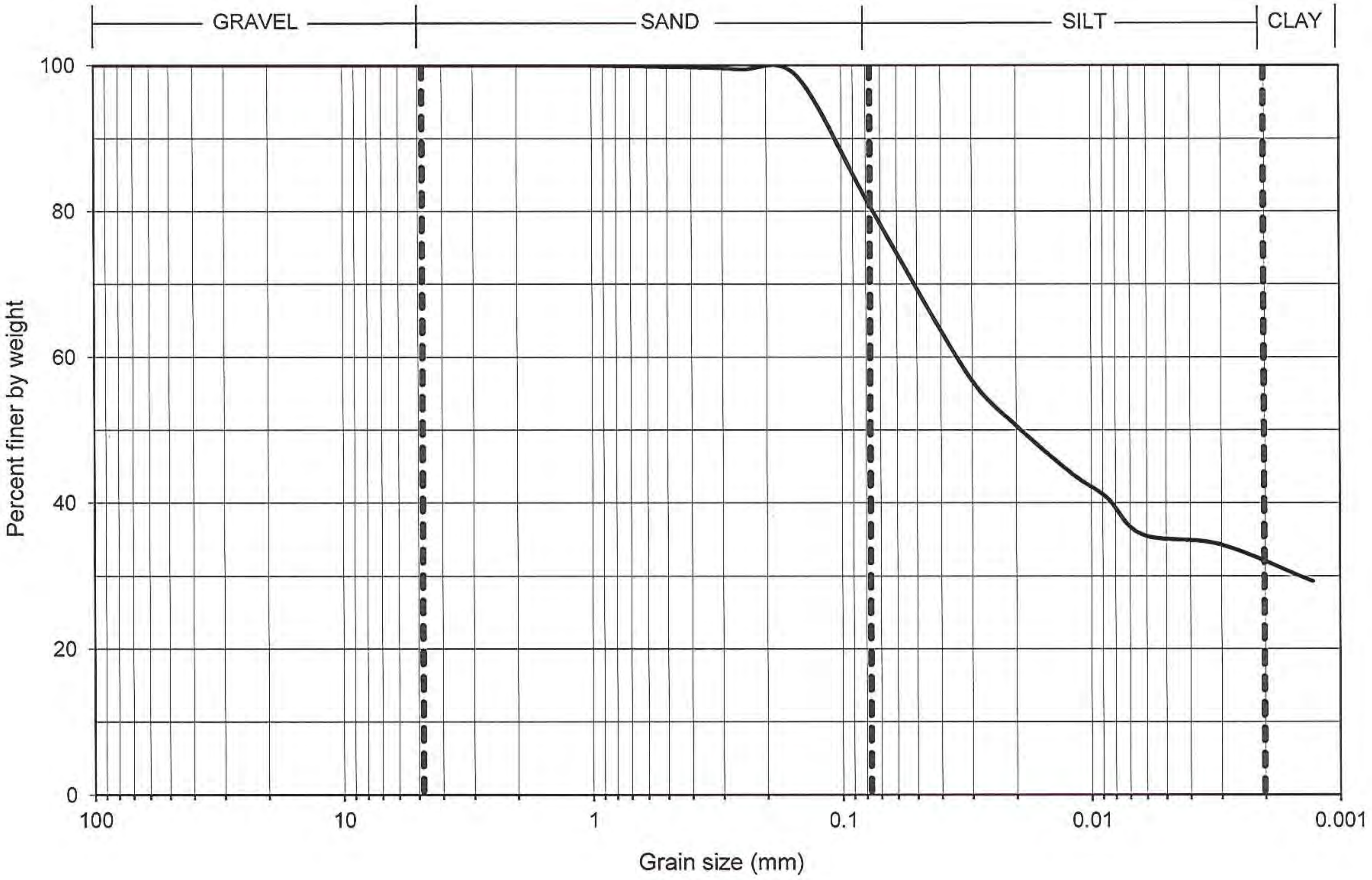


Grain Size Distribution (ASTM D1140 and D 422)
Seaside Campus
Sample B 3-8   Depth = 20 feet

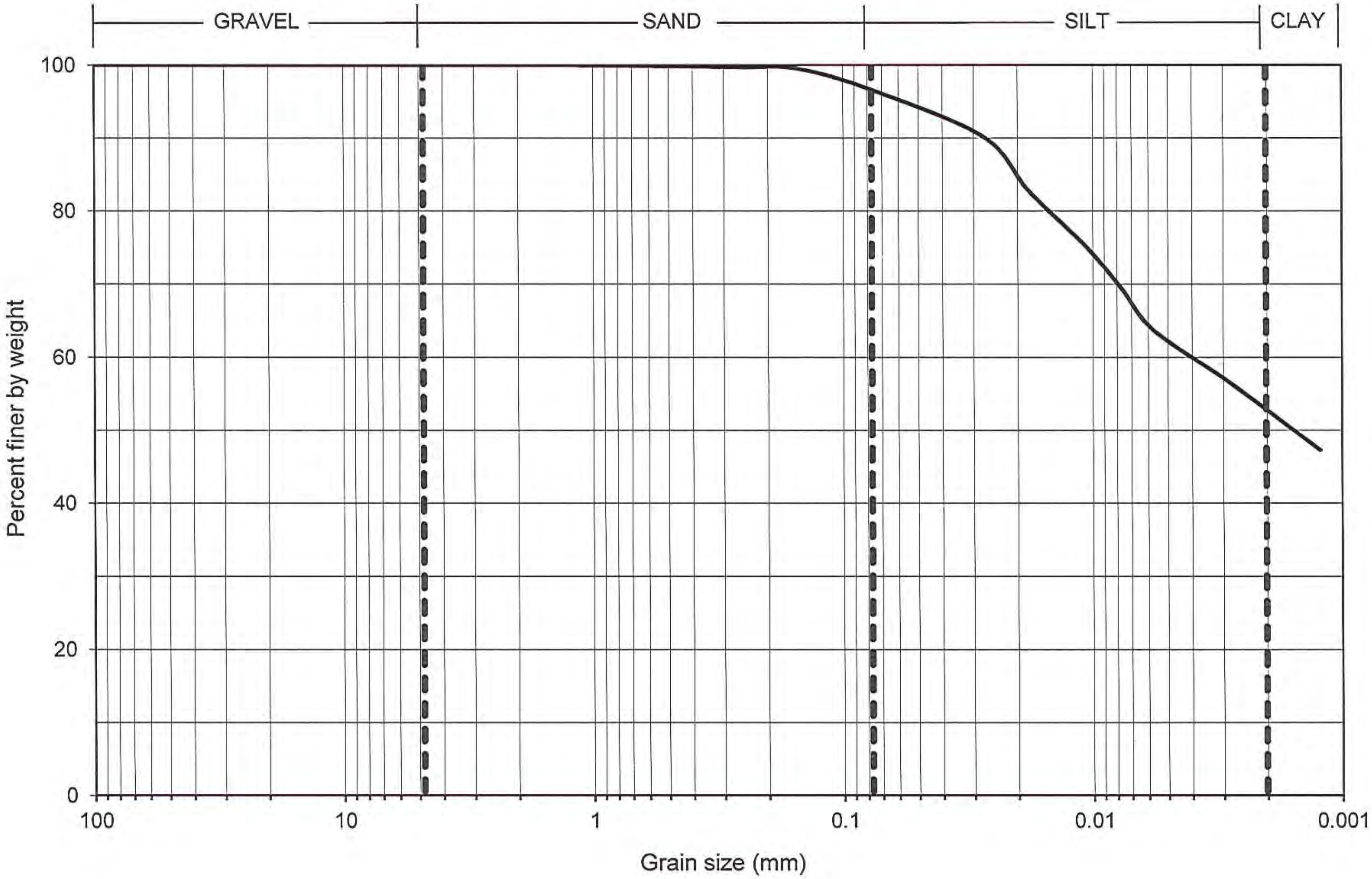


GRAIN SIZE DISTRIBUTION

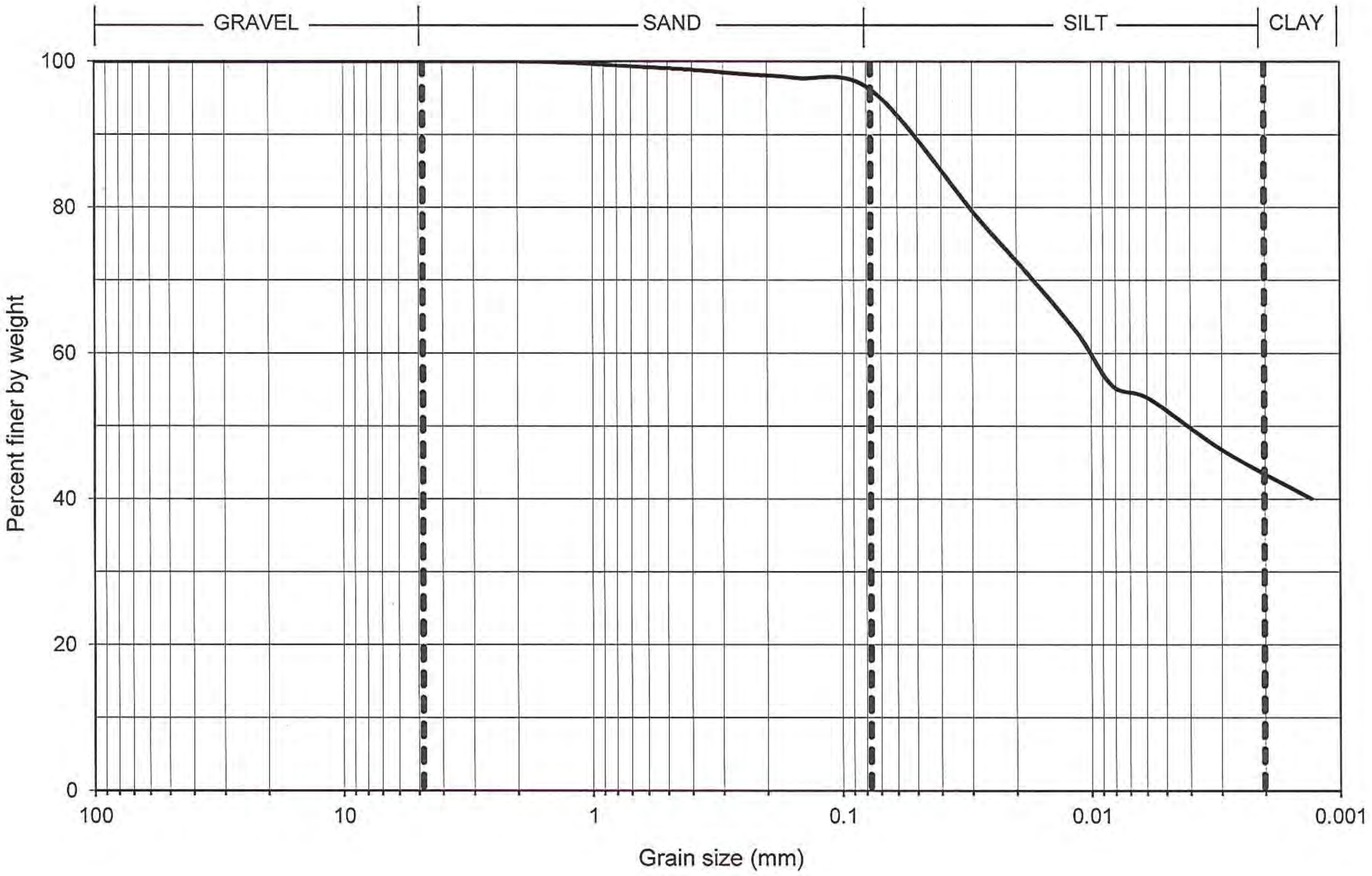
Seaside Campus
P1910-05-01
Sample: B3-13 Depth = 32.5 feet



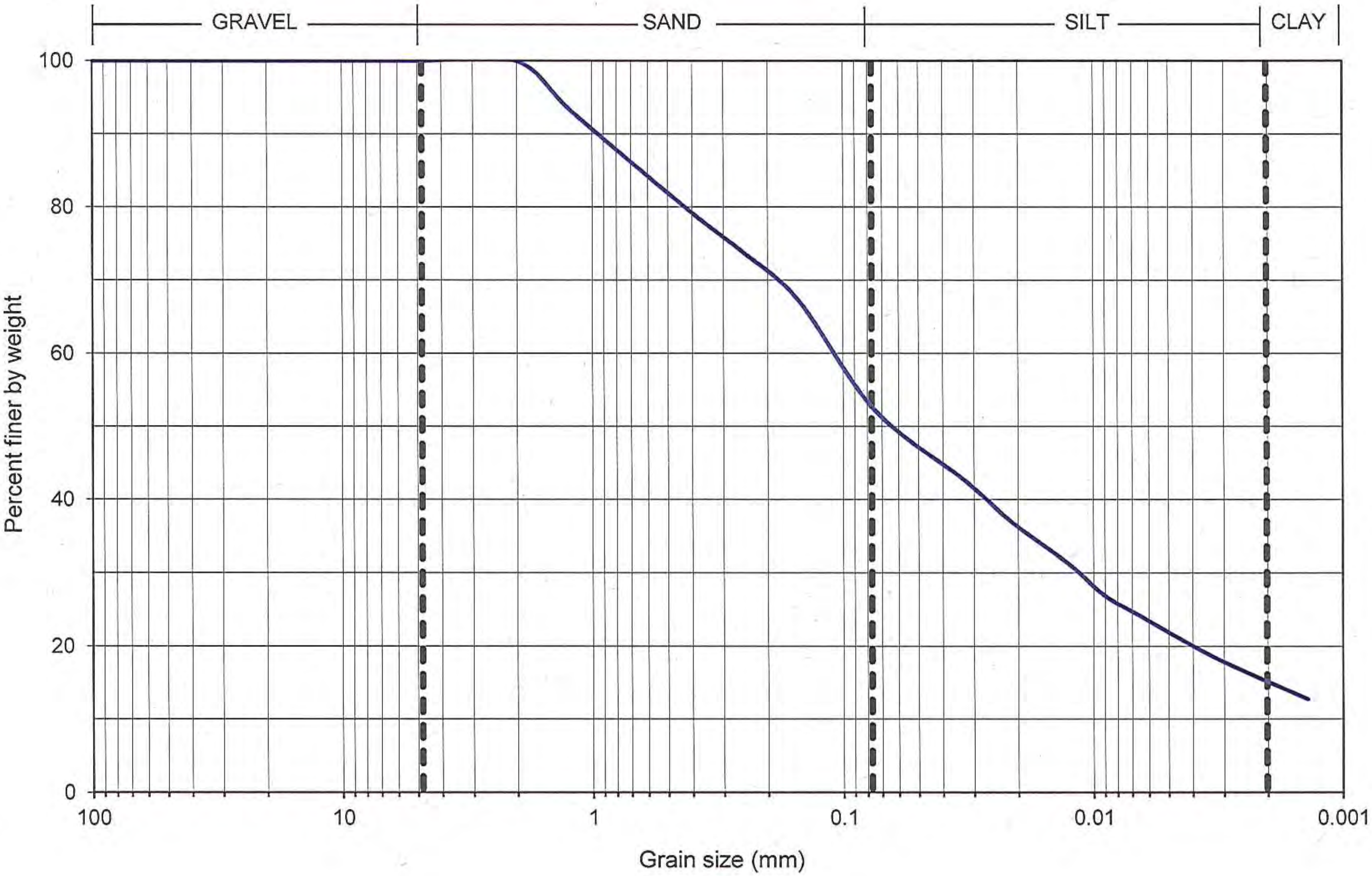
GRAIN SIZE DISTRIBUTION
Seaside Campus
P1910-05-01
Sample: B3-15 Depth = 40 feet



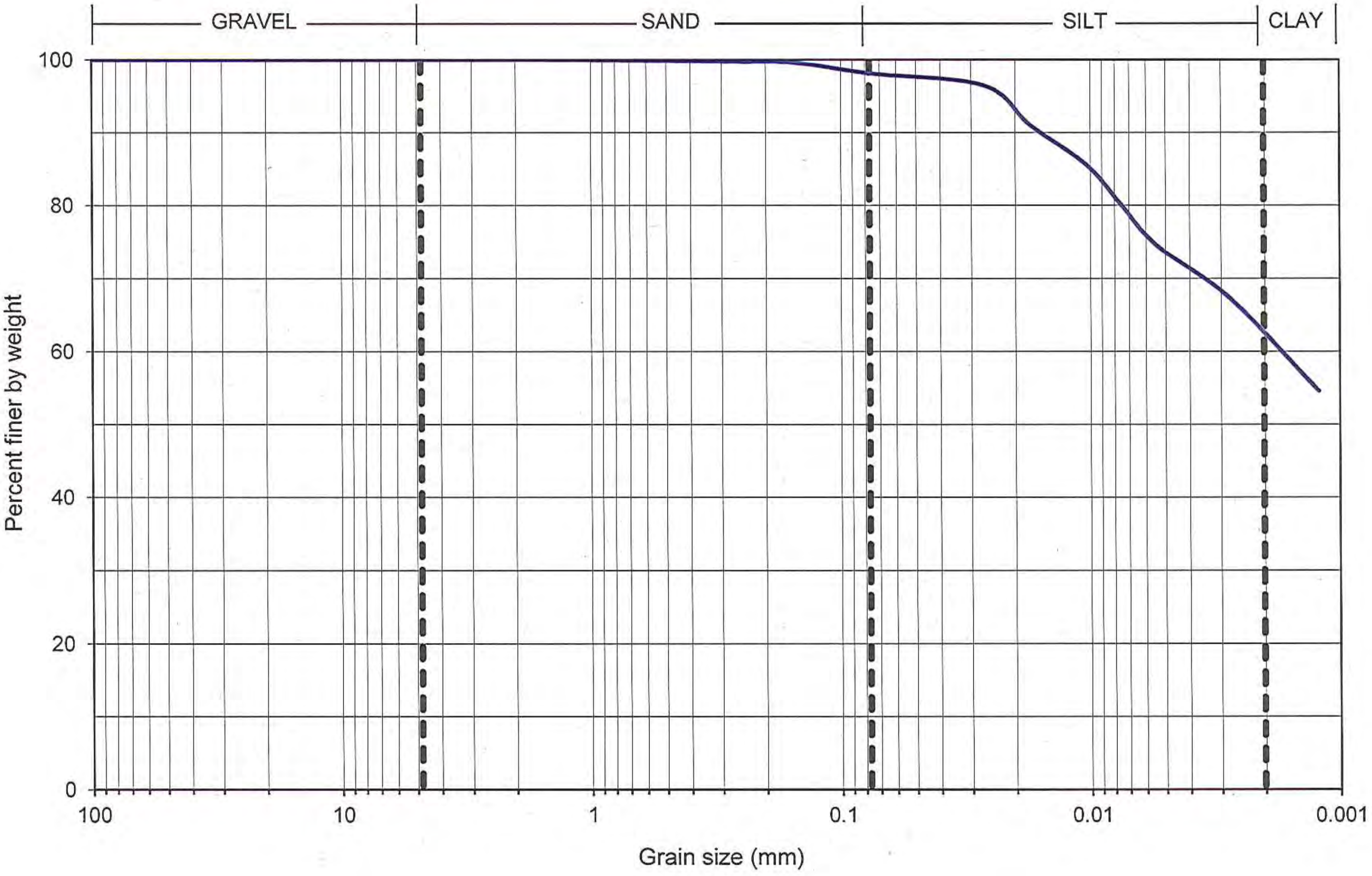
GRAIN SIZE DISTRIBUTION
Seaside Campus
P1910-05-01
Sample: B3-17 Depth = 50 feet



GRAIN SIZE DISTRIBUTION
Seaside Campus
P1910-05-01
Sample: B4-4 Depth = 10 feet



GRAIN SIZE DISTRIBUTION
Seaside Campus
P1910-05-01
Sample: B4-12 Depth = 40 feet

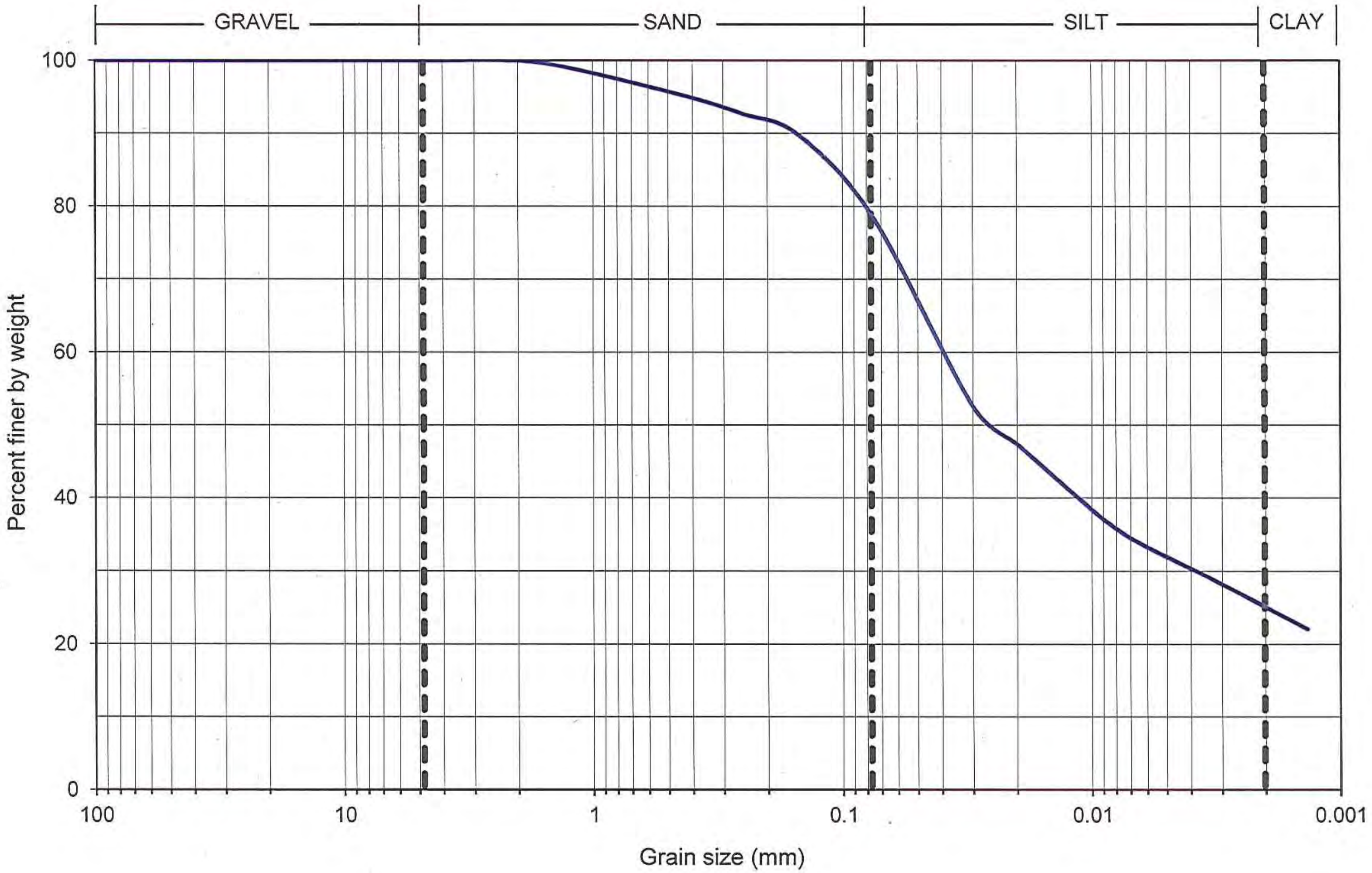


GRAIN SIZE DISTRIBUTION

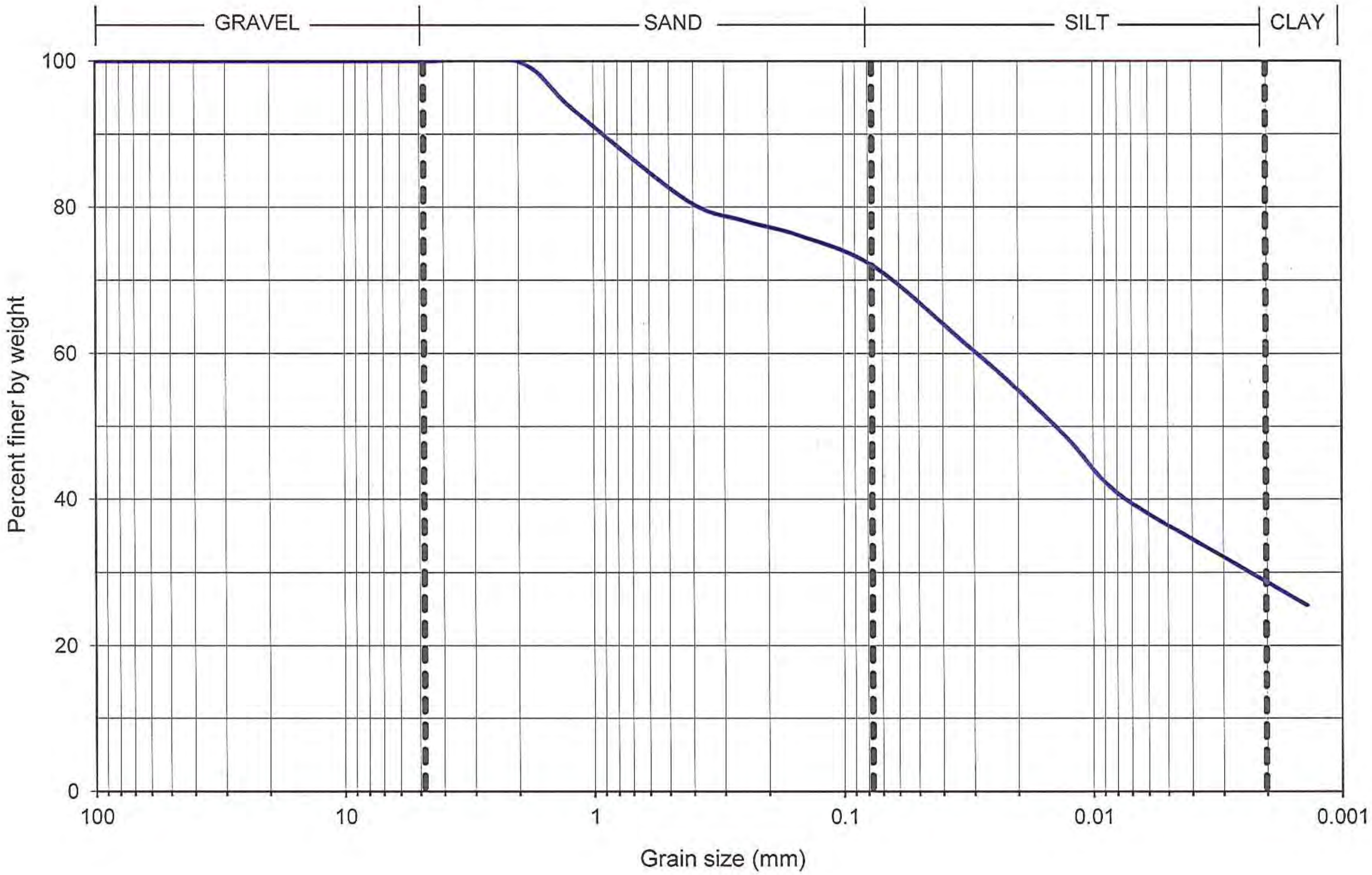
Seaside Campus

P1910-05-01

Sample: B5-2 Depth = 10 feet

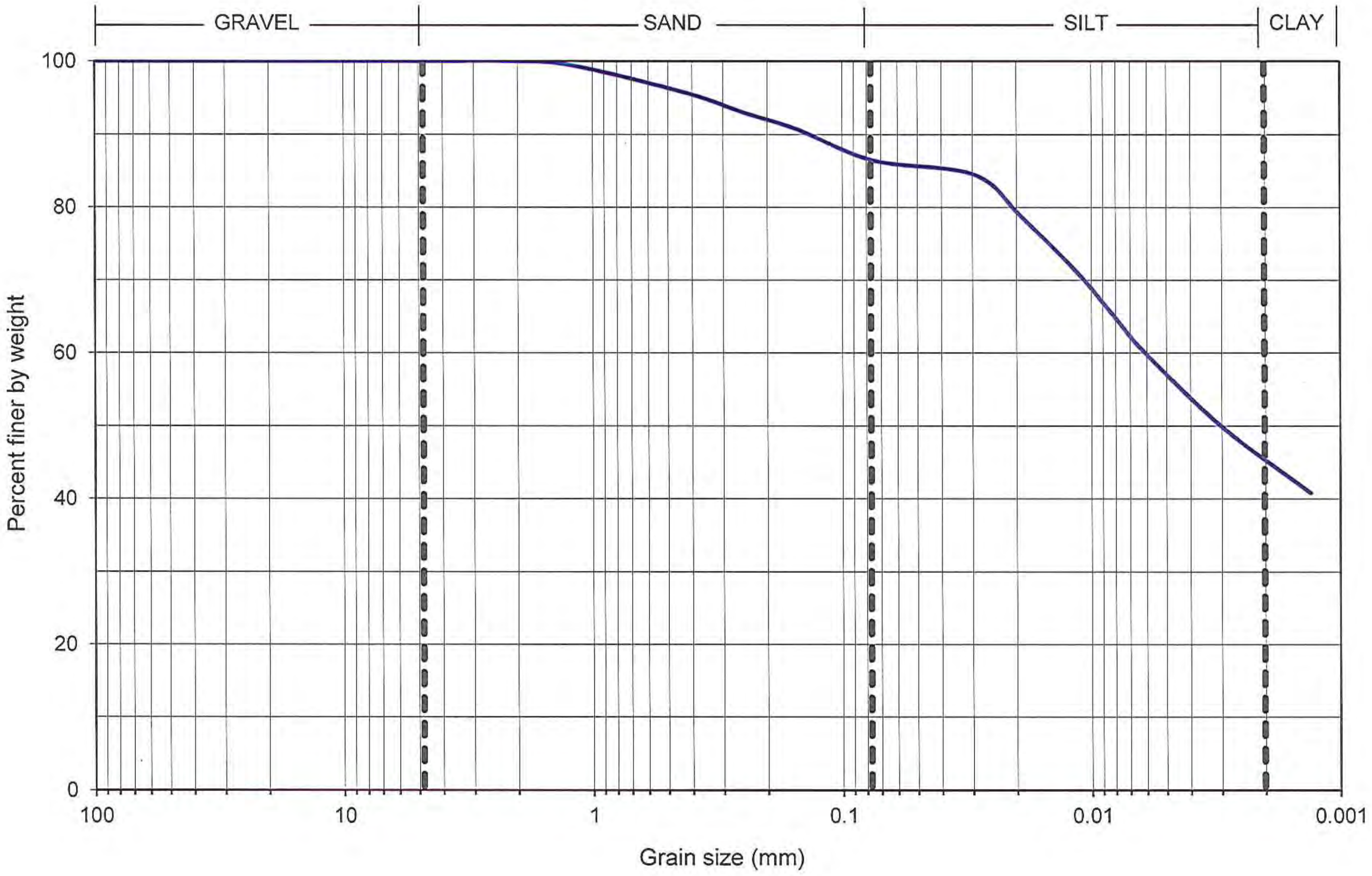


GRAIN SIZE DISTRIBUTION
Seaside Campus
P1910-05-01
Sample: B5-4   Depth = 20 feet

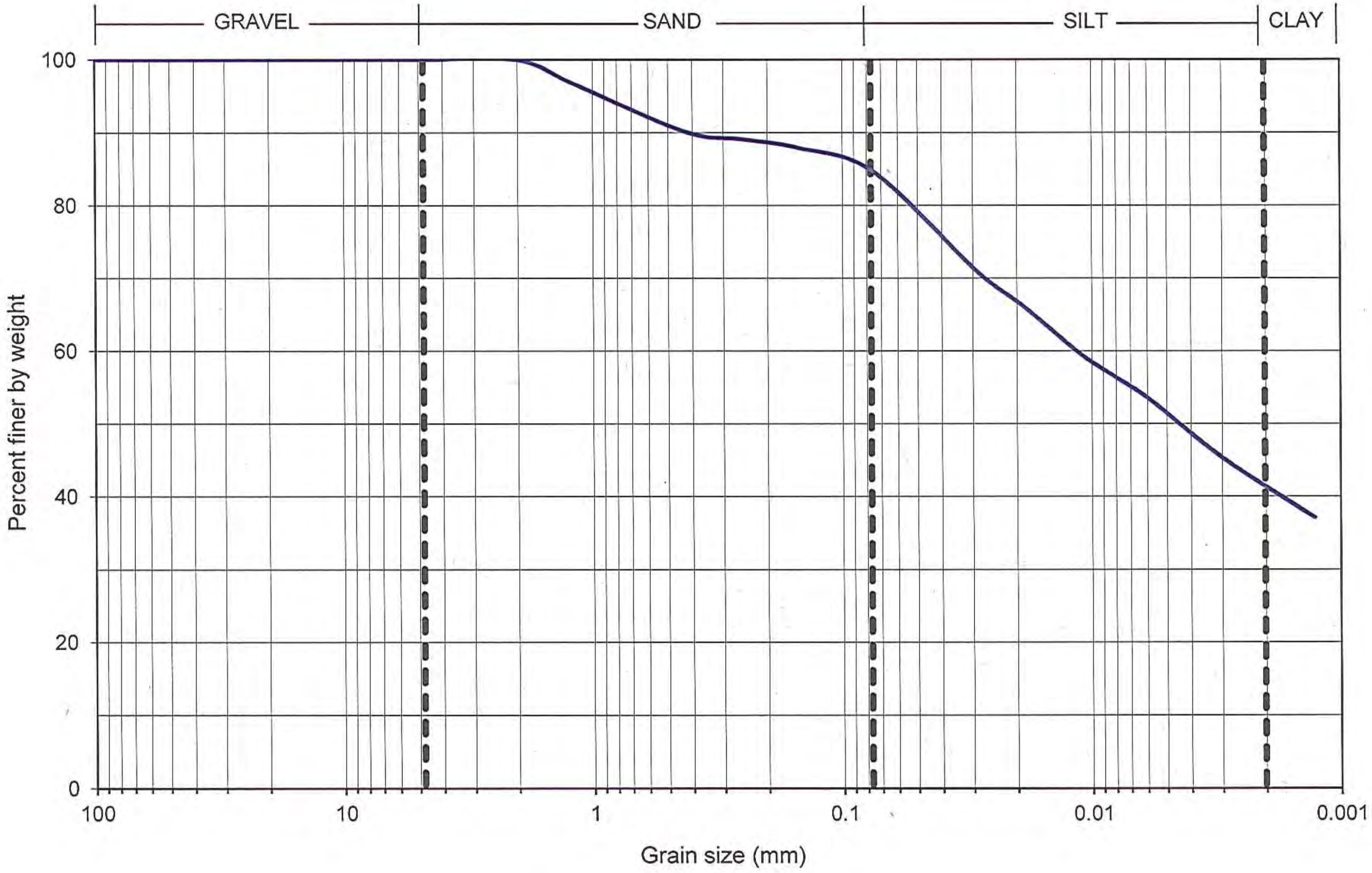


GRAIN SIZE DISTRIBUTION

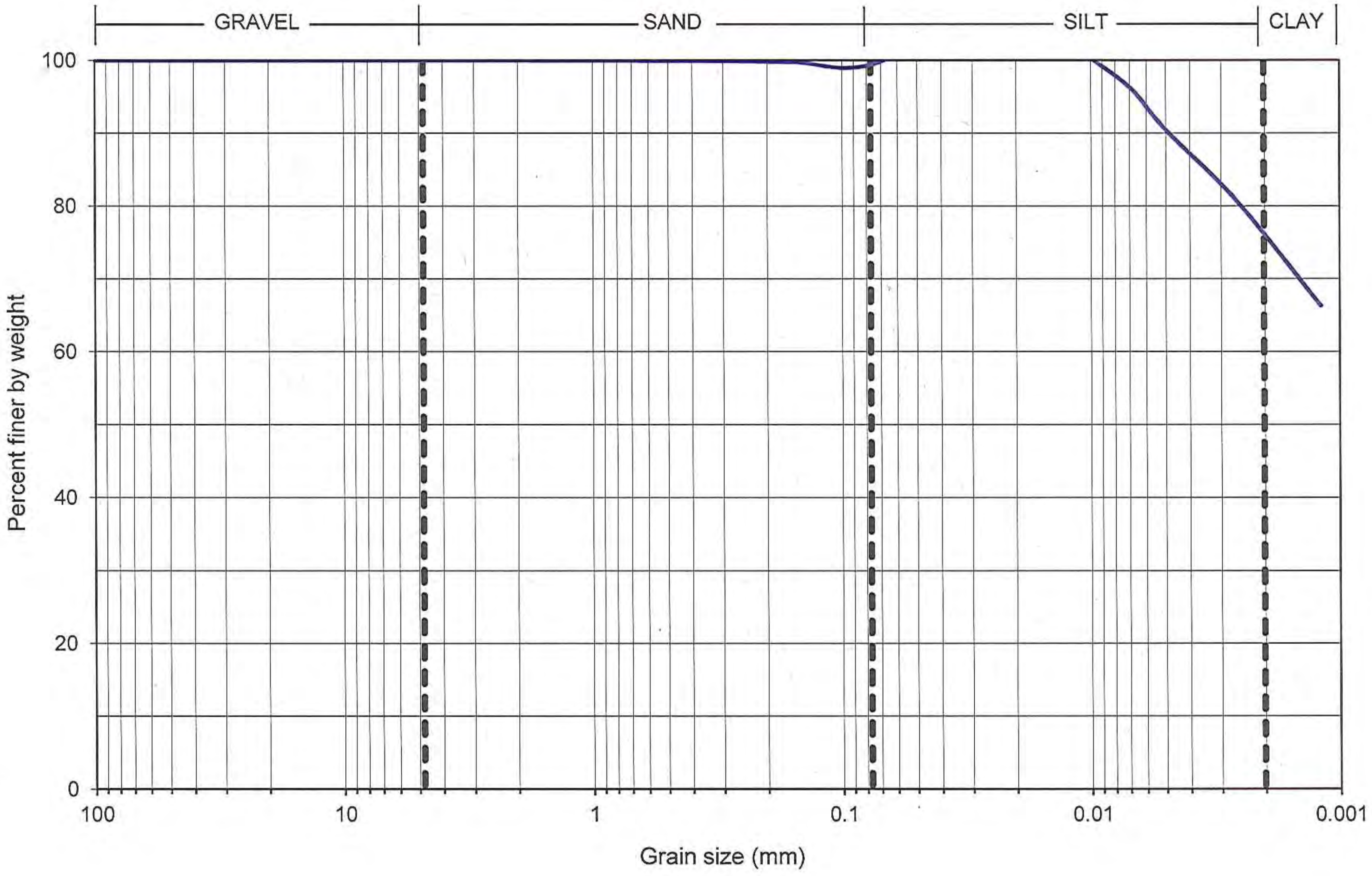
Seaside Campus
P1910-05-01
Sample: B6-3 Depth = 15 feet



GRAIN SIZE DISTRIBUTION
Seaside Campus
P1910-05-01
Sample: SI1-3 Depth = 7.5 feet

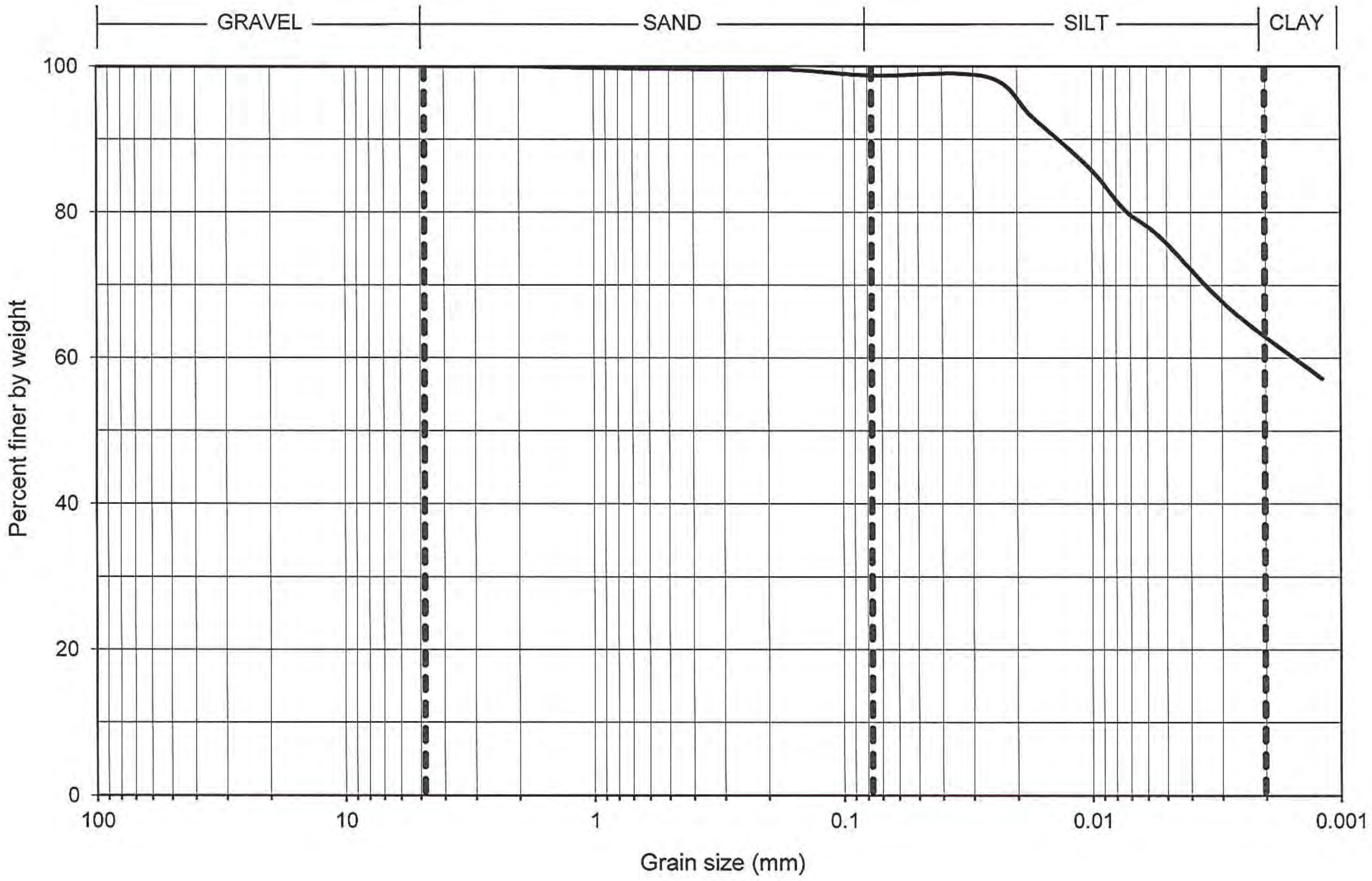


GRAIN SIZE DISTRIBUTION
Seaside Campus
P1910-05-01
Sample: SI1-11 Depth = 60 feet



GRAIN SIZE DISTRIBUTION

Seaside Campus
P1910-05-01
Sample: SI 2-10   Depth = 52.5 feet

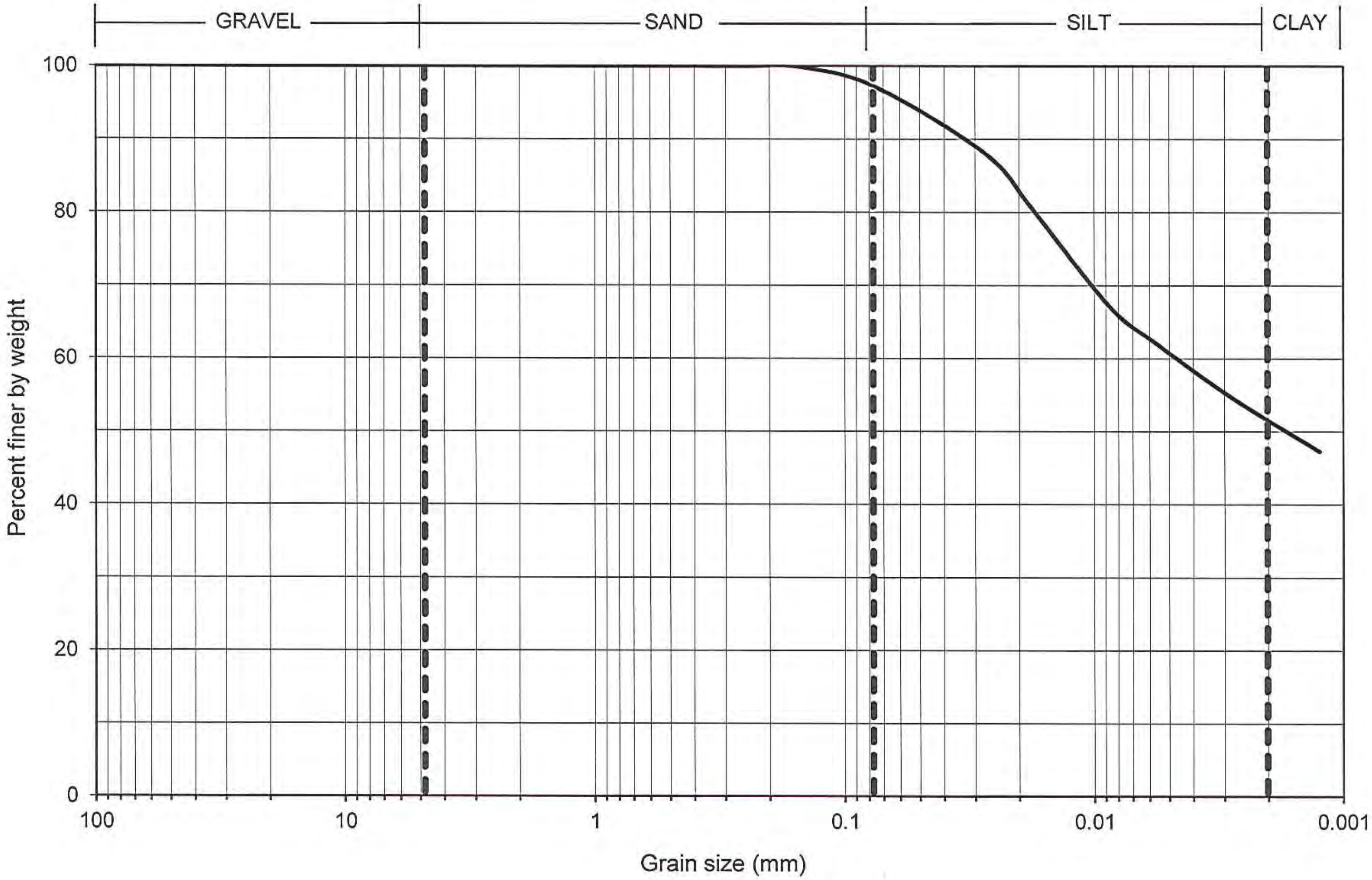


GRAIN SIZE DISTRIBUTION

Seaside Campus

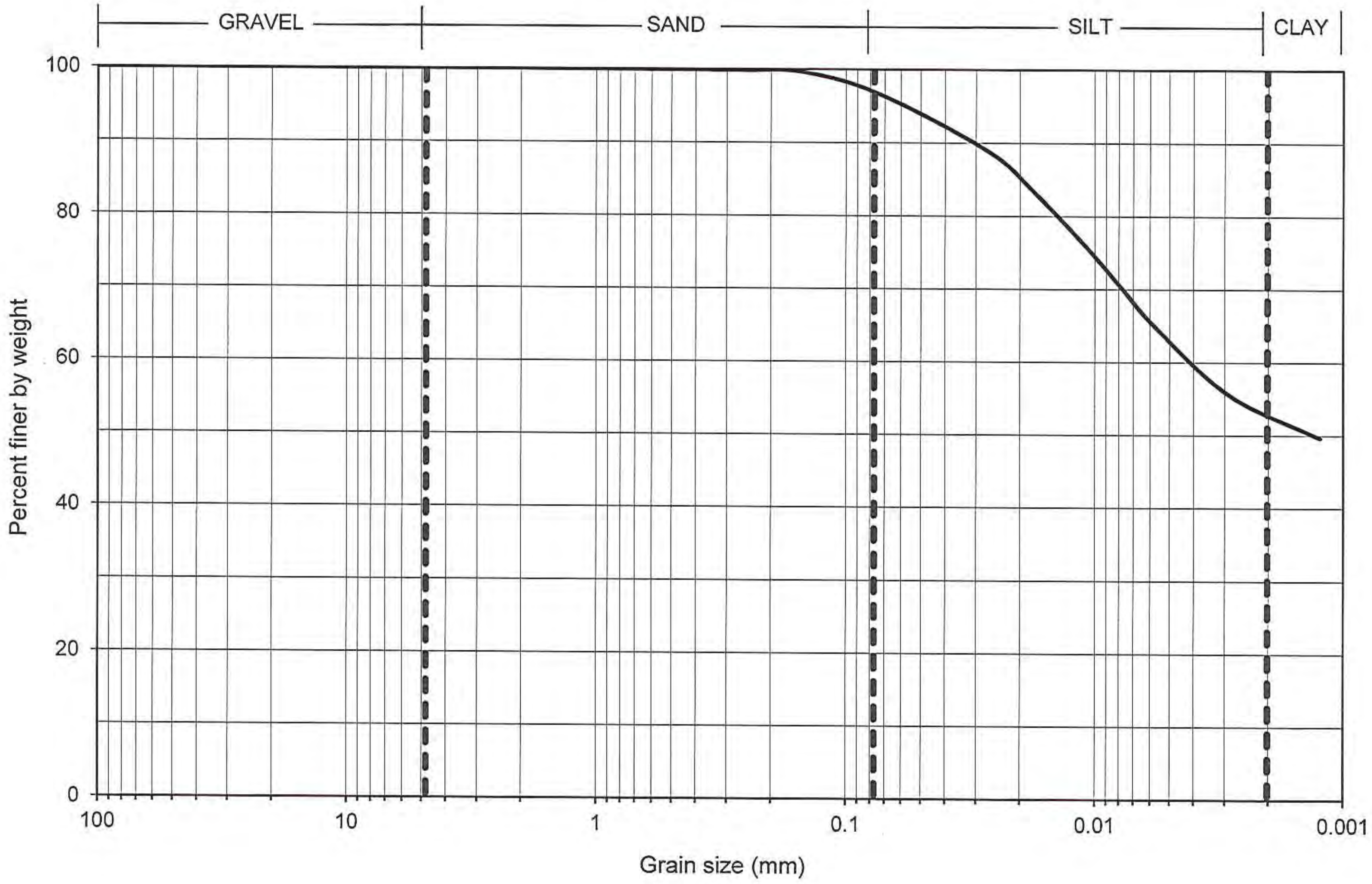
P1910-05-01

Sample: SI2-18 Depth = 75 feet



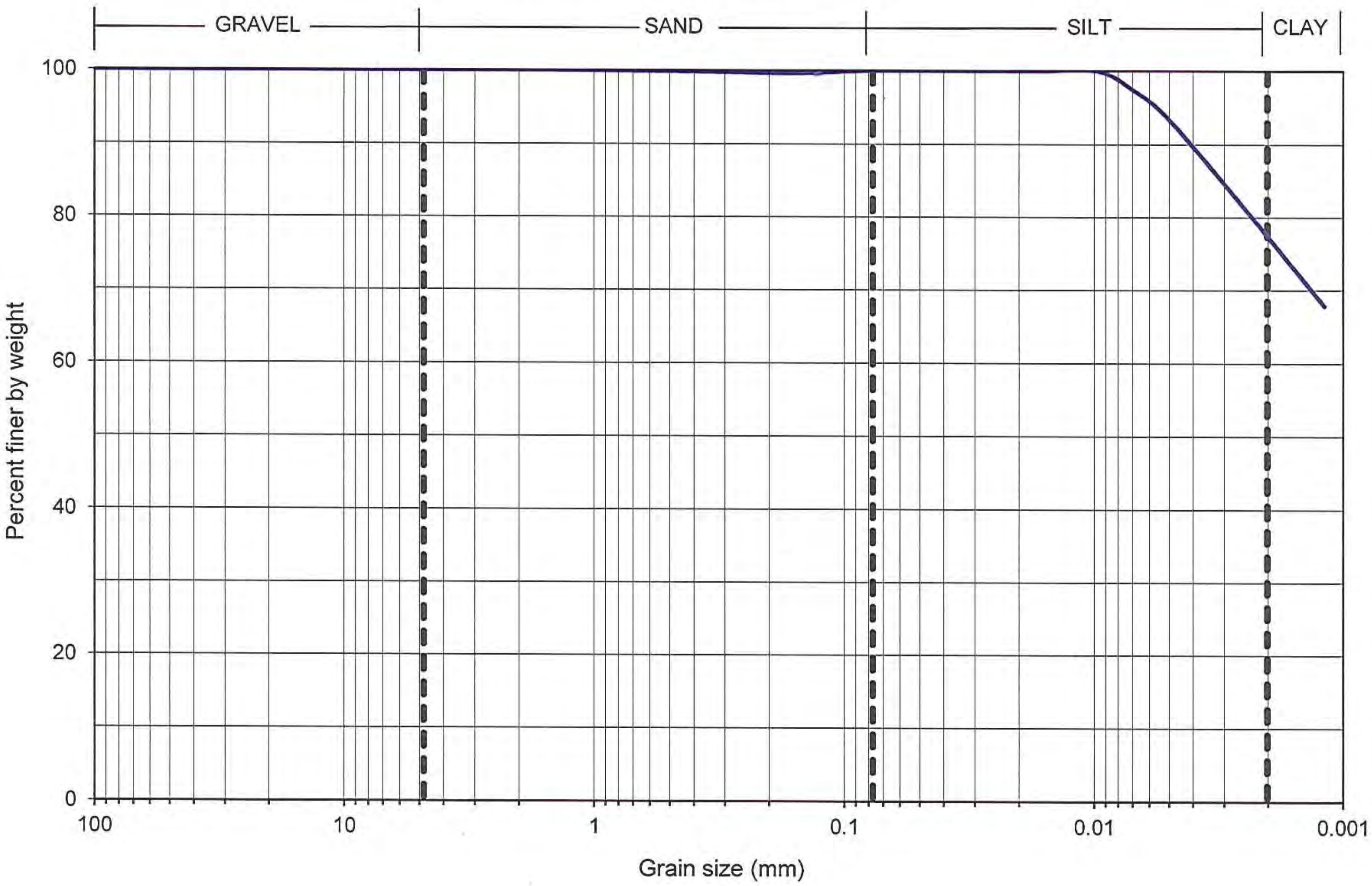
GRAIN SIZE DISTRIBUTION

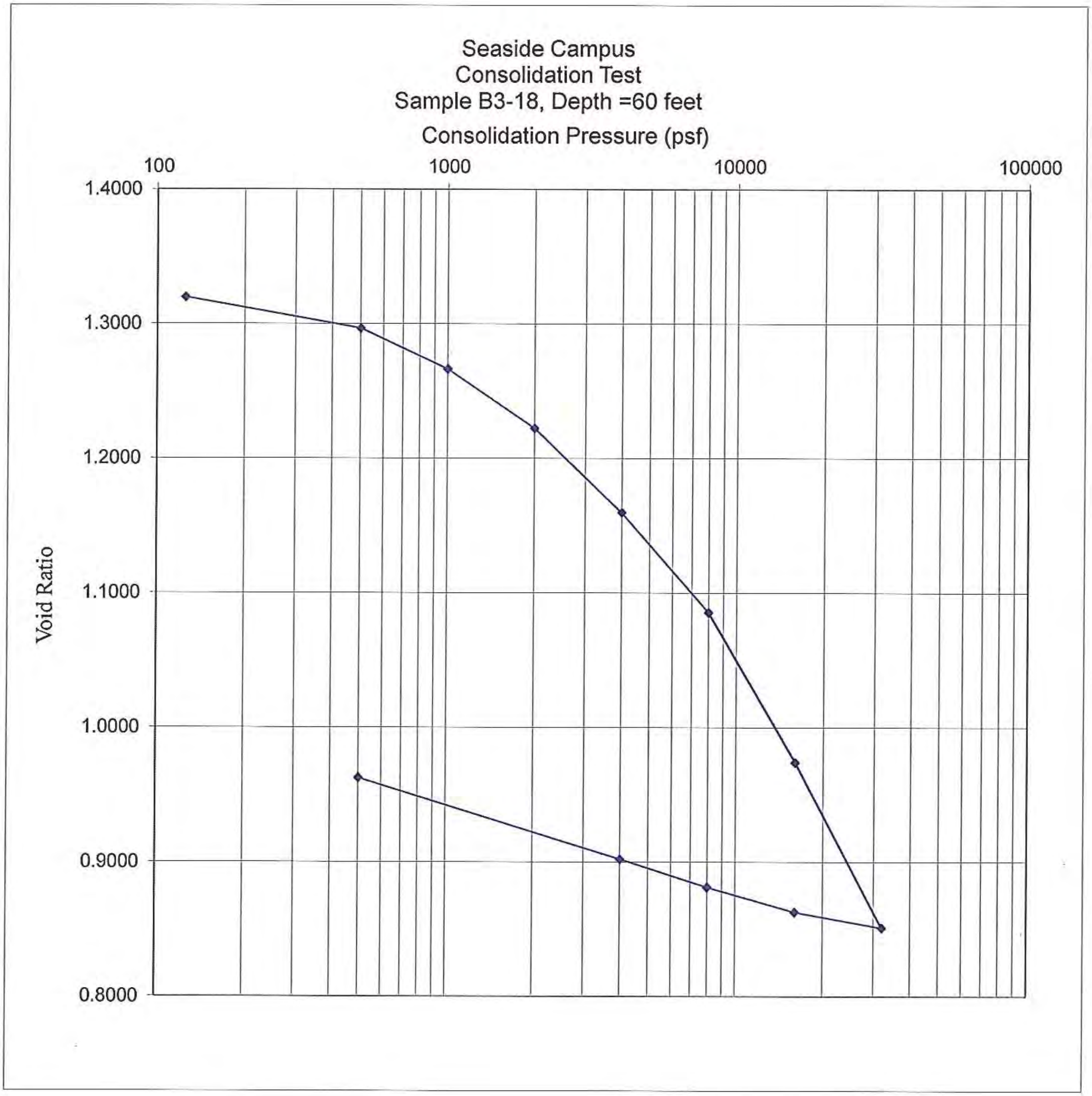
Seaside Campus
P1910-05-01
Sample: SI2-20 Depth = 90 feet

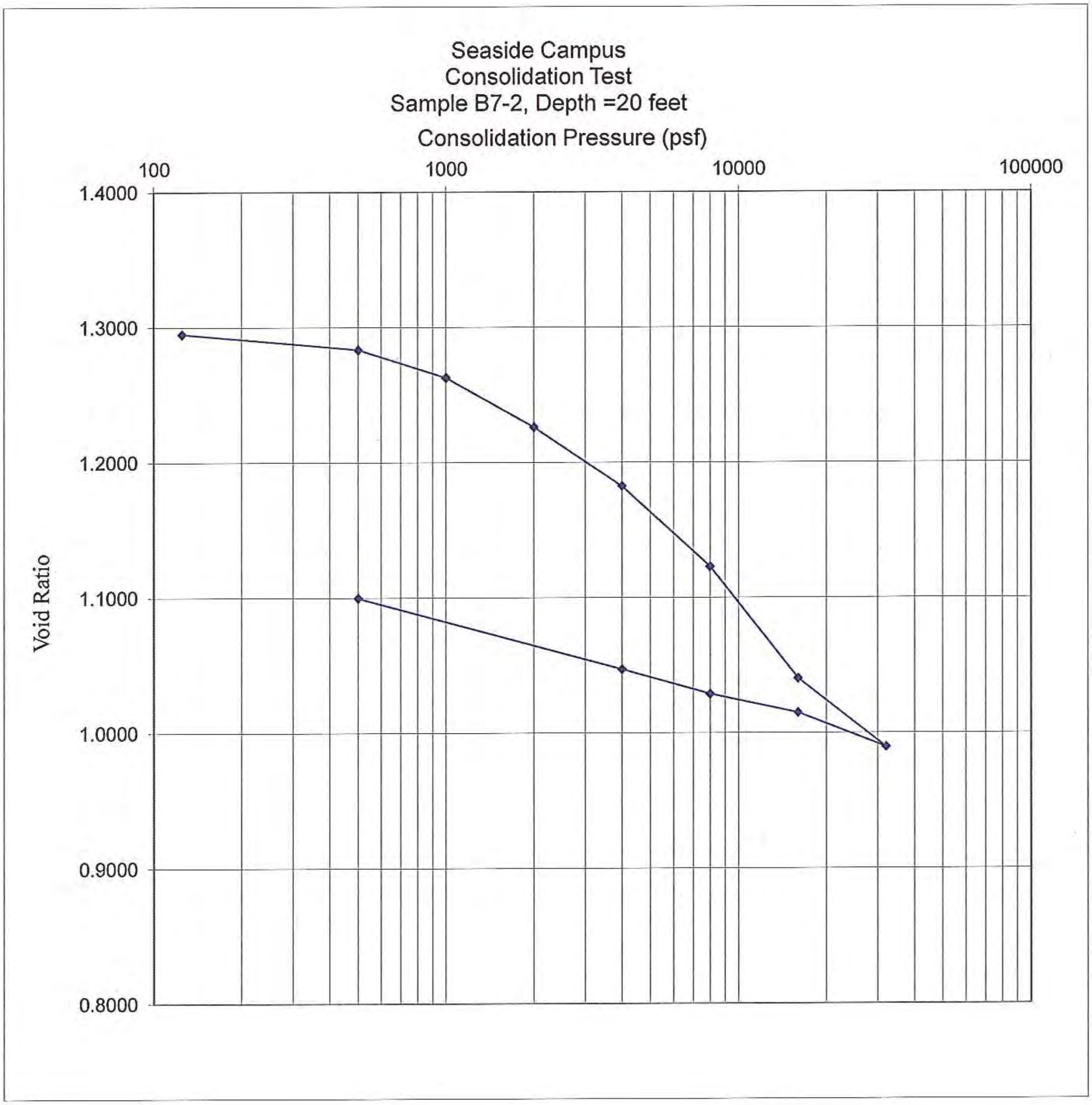


GRAIN SIZE DISTRIBUTION

Seaside Campus
P1910-05-01
Sample: SI3-19 Depth = 70 feet




Seaside Campus
Consolidation Test
Sample B3-18, Depth =60 feet
Consolidation Pressure (psf)
100
1000
10000
100000
1.4000
1.3000
1.2000
1.1000
1.0000
0.9000
0.8000
Void Ratio



Seaside Campus
Consolidation Test
Sample B7-2, Depth =20 feet

GEOCON
INCORPORATED

DIRECT SHEAR TEST REPORT

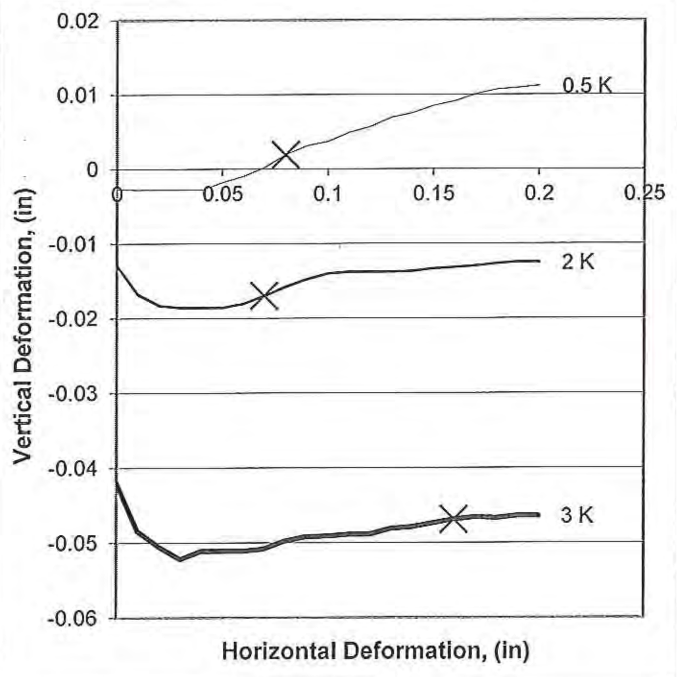
Seaside Campus

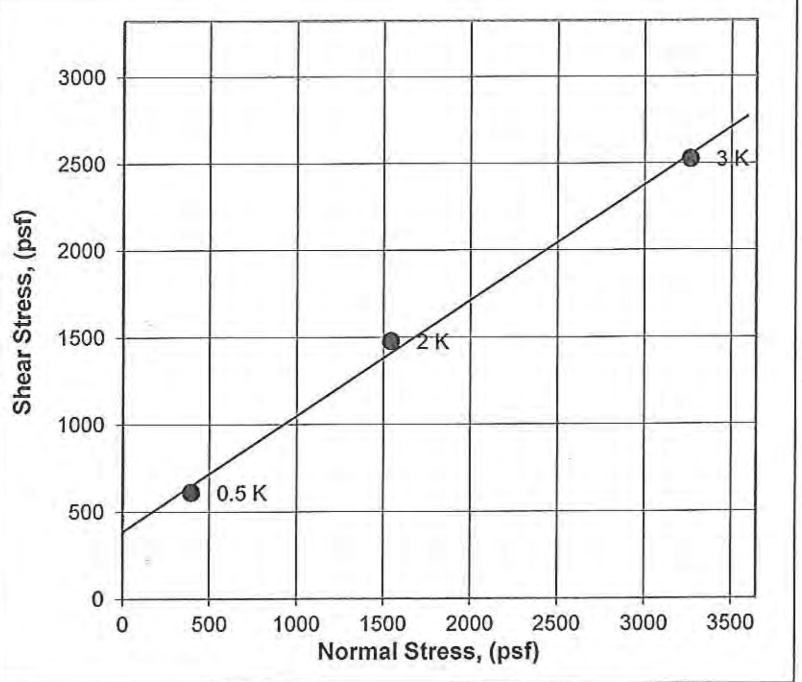
P1910-05-01 **Date:** Wednesday, November 21, 2012 **By:** NJ

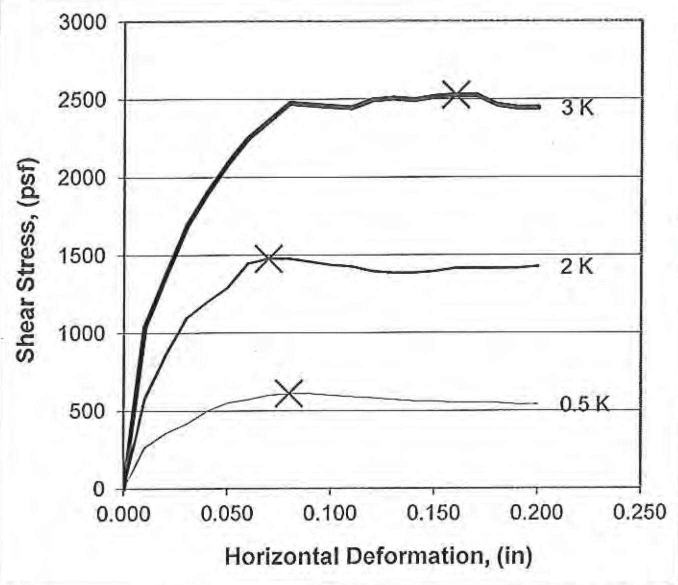
Sample No.: B7-1 **Natural or Remold:** Natural

Description: ML-Dark yellowish brown(f-m0sandy silt w/little clay.

Remarks:







φ (Degrees)	**33.5**
c (psf)	**389**
Tan φ	**0.663**
Method	**Calc**

Load	0.5 K	2 K	3 K
INITIAL			
Water Content	36.0%	35.6%	35.7%
Dry Density (pcf)	84.5	83.7	83.9
Saturation*	99.7%	96.4%	97.3%
Height (inches)	1.00	1.00	1.00
AFTER TEST			
Water Content	36.5%	36.1%	36.4%
Dry Density (pcf)	83.6	84.7	88.0
FAILURE			
Normal Stress (psf)	395	1543	3264
Failure Stress (psf)	611	1477	2525
Failure Deffinition	Max	Max	Max
Displacement (in)	0.08	0.07	0.16
Rate (in/min)	0.0050	0.0050	0.0050

* Degree of saturation calculated with a specific gravity of 2.65

GEOCON
INCORPORATED

DIRECT SHEAR TEST REPORT

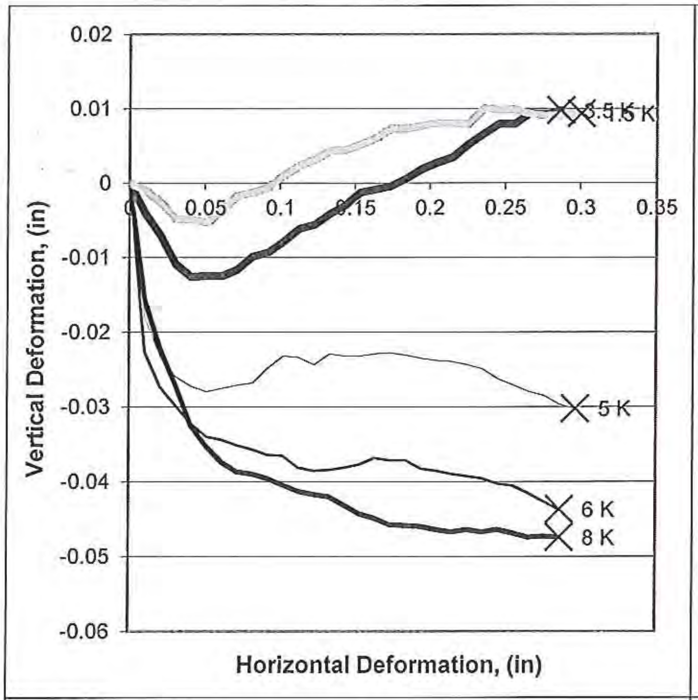
SEASIDE CAMPUS

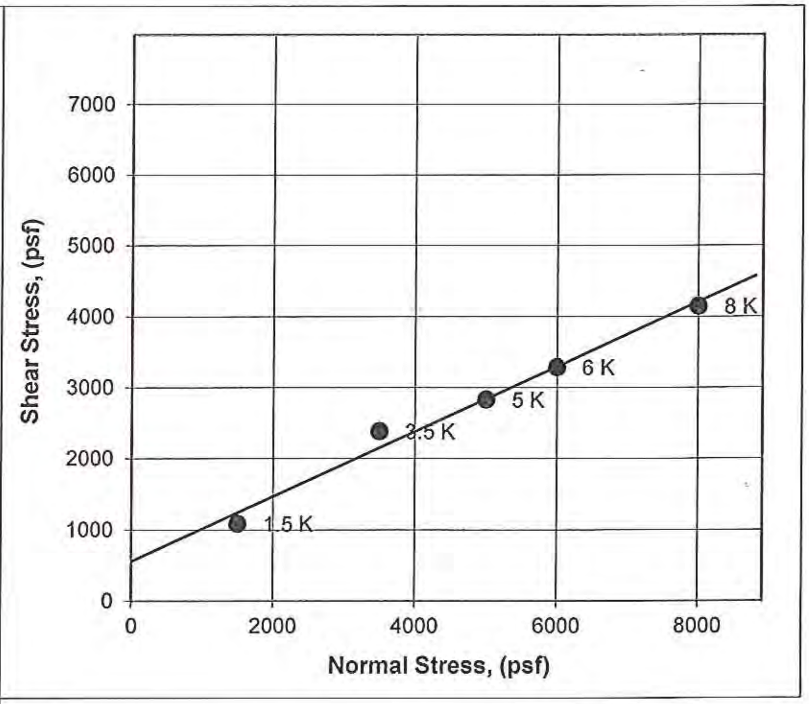
P1910-05-01 **Date:** Wednesday, November 21, 2012 **By:** TG

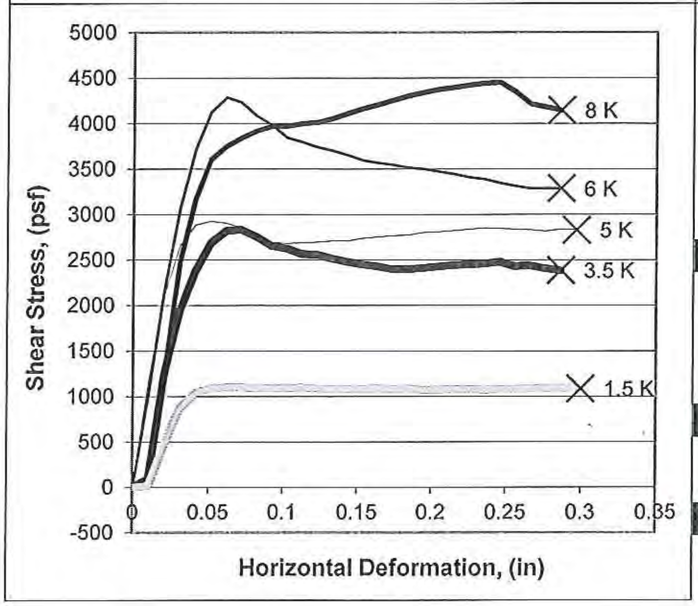
Sample No.: B3-18 **Natural or Remold:** Natural

Description: CL-DARK GRAY

Remarks:







ϕ (Degrees)	24.5
c (psf)	556
Tan ϕ	0.457
Method	Calc

Load	5 K	6 K	8 K	3.5 K	1.5 K
INITIAL					
Water Content	29.1%	27.5%	28.8%	28.0%	30.1%
Dry Density (pcf)	93.6	93.6	91.9	95.3	91.8
Saturation*	100.5%	94.8%	95.5%	100.7%	99.3%
Height (inches)	1.00	1.00	1.00	1.00	1.00
AFTER TEST					
Water Content	32.1%	30.3%	31.2%	30.4%	32.1%
Dry Density (pcf)	96.5	97.8	96.5	94.4	90.9
FAILURE					
Normal Stress (psf)	5000	6000	8000	3500	1500
Failure Stress (psf)	2831	3287	4147	2386	1086
Failure Deffinition	User	User	User	User	User
Displacement (in)	0.30	0.29	0.29	0.29	0.30
Rate (in/min)	0.0010	0.0010	0.0010	0.0011	0.0011

* Degree of saturation calculated with a specific gravity of 2.65

APPENDIX C

COOPER LABORATORY TESTING

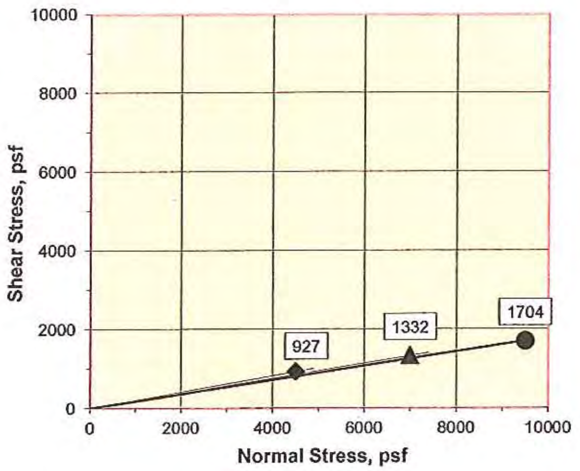
Cooper Laboratory, a specialized geotechnical engineering testing laboratory, performed ring shear residual strength testing on 3 soil specimens. This testing determines effective stress friction angles that represent the lower-bound shear strength of a clay soil that has undergone horizontal displacement. Their results are presented in the following figures.

COOPER

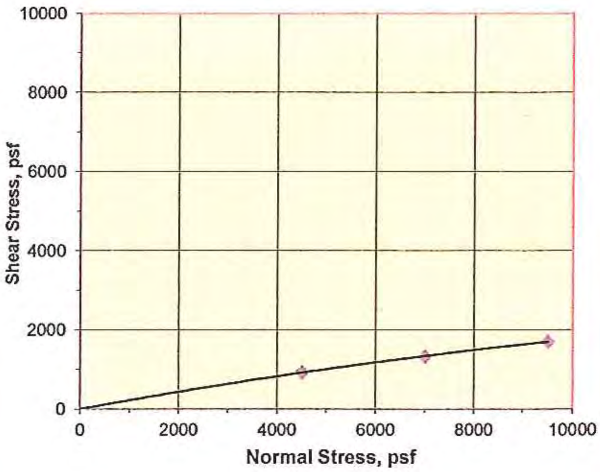
Drained, Residual Torsional Ring Shear

Job No.:	379-016a	**Boring:**	B3-19	**Date:**	12/15/2012	**Undisturbed:**	
Client:	Geocon	**Sample:**		**By:**	PJ	**Peak:**	
Project:	Seaside Campus	**Depth:**		**Checked:**	DC	**Residual:**	
Soil Type:	Dark Gray CLAY			**Clay, %:**		**Fully Softened:**	X
Remarks:	A small friction correction was applied to each point.			**LL:**		**Peak:**	
Normal Stress, psf	4500	7000	9500	**PL:**		**Residual:**	X
Secant Phi, deg.	12	11	10				

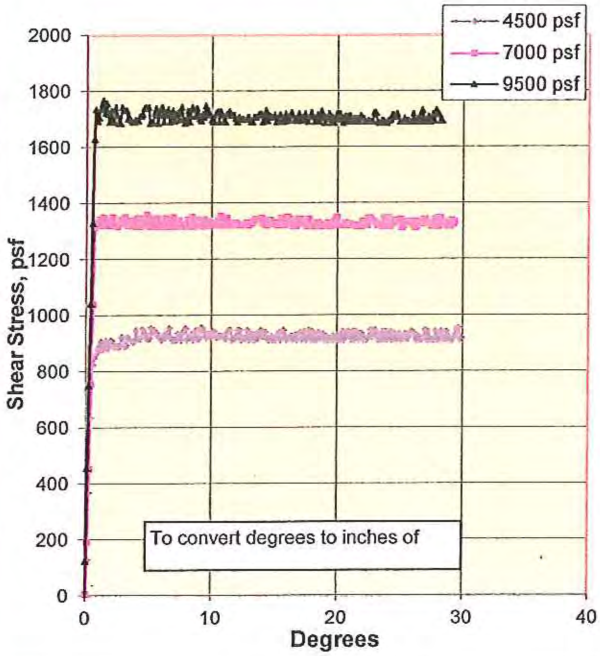
Secant Residual Friction Angles



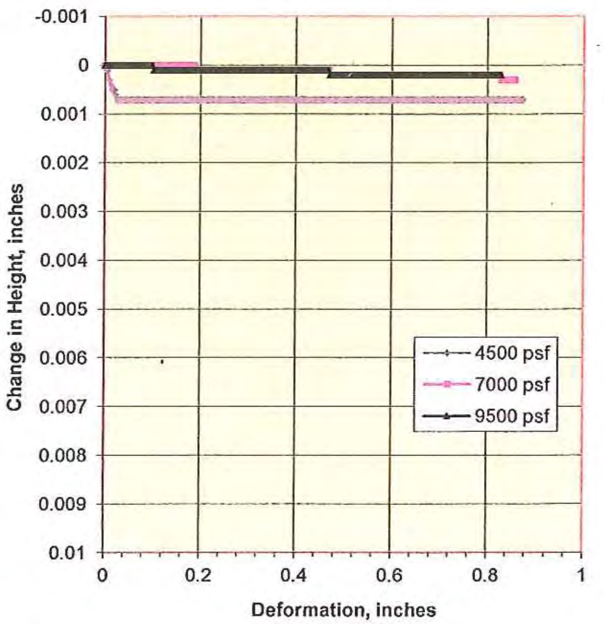
Strength Envelope



Deformation Curves


To convert degrees to inches of

Vertical Deformation



COOPER
TESTING LABORATORY

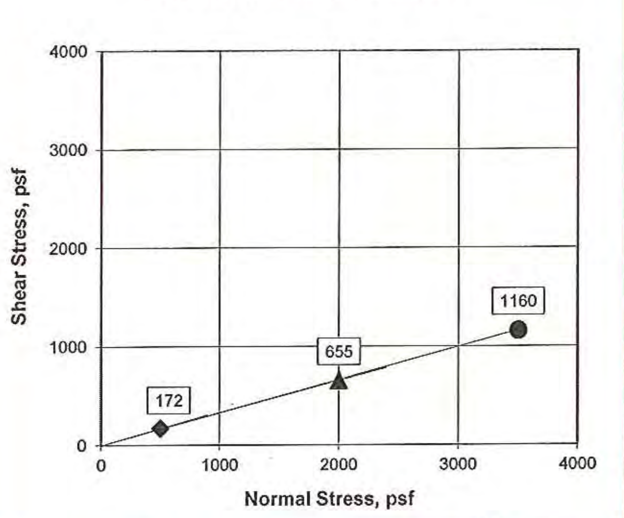
Drained, Residual Torsional Ring Shear Test ASTM D 6467

Job No.: 379-016
Client: Geocon NW
Project: Seaside Campus - P1910-05-1
Soil Type: Dark Gray CLAY
Remarks: A small friction correction was applied to each point.
Boring: B6-3
Sample:
Depth:
Date: 12/13/2012
By: PJ
Checked: DC
Clay, %:
LL:
PL:

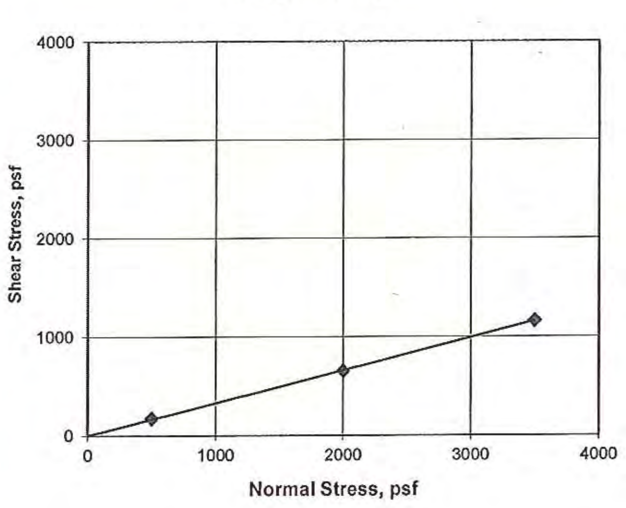
Undisturbed:
Peak:
Residual:
Fully Softened: X
Peak:
Residual: X

Normal Stress, psf	500	2000	3500
Secant Phi, deg.	19	18	18

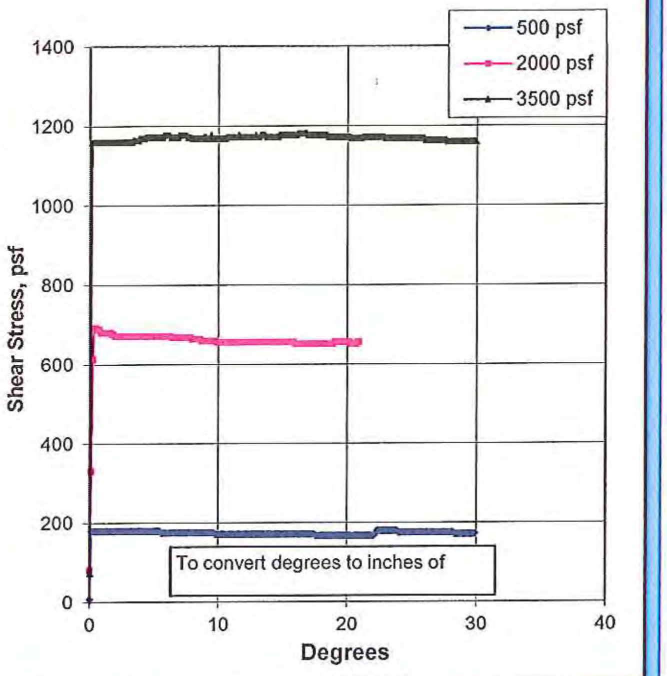
Secant Residual Friction Angles



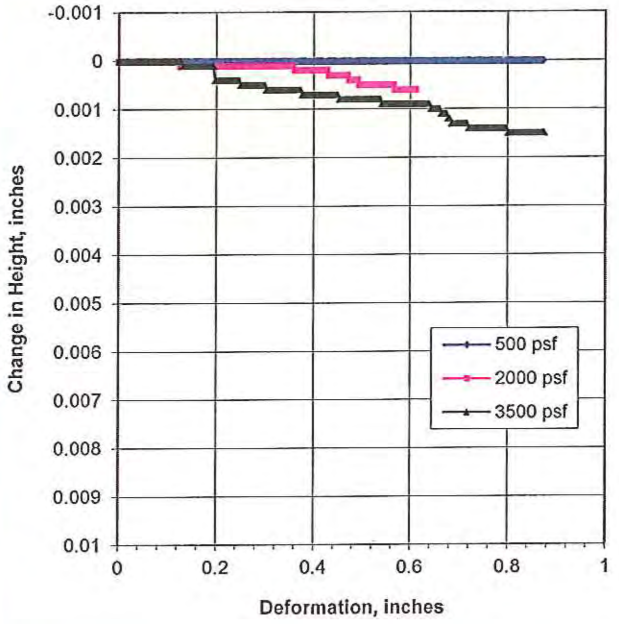
Strength Envelope



Deformation Curves



Vertical Deformation

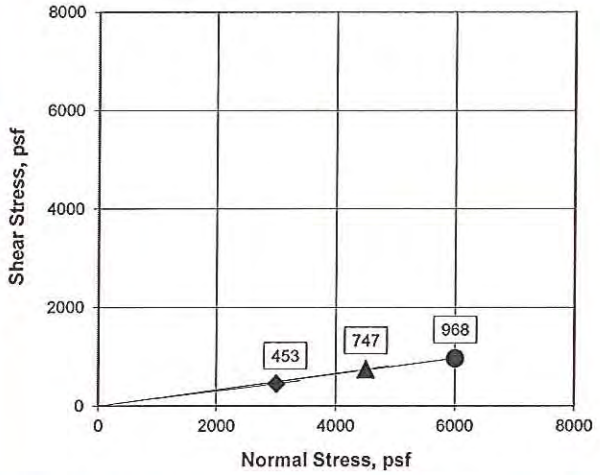


COOPER TESTING LABORATORY

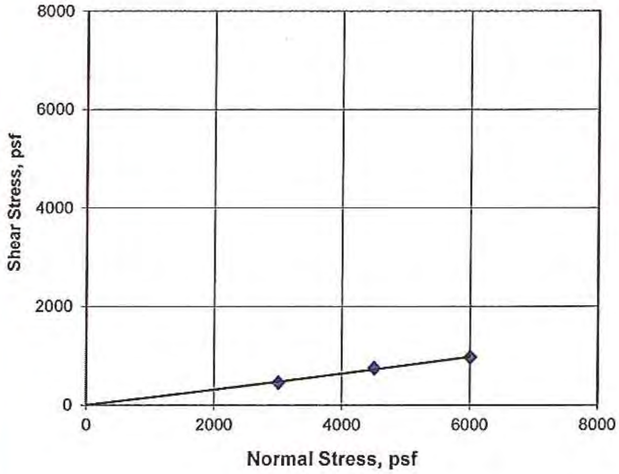
Drained, Residual Torsional Ring Shear Test ASTM D 6467

Job No.:	379-017	**Boring:**	S12-11	**Date:**	12/27/2012	**Undisturbed:**	
Client:	Geocon	**Sample:**		**By:**	PJ	**Peak:**	
Project:	Seaside Campus	**Depth:**	55	**Checked:**	DC	**Residual:**	
Soil Type:	Dark Brown CLAY			**Clay, %:**		**Fully Softened:**	X
Remarks:	A small friction correction was applied to each point.			**LL:**		**Peak:**	
Normal Stress, psf	3000	4500	6000	**PL:**		**Residual:**	X
Secant Phi, deg.	9	9	9				

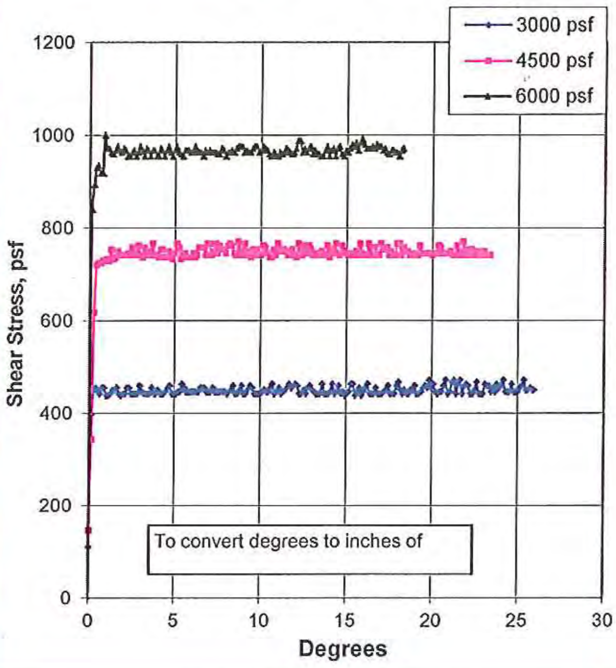
Secant Residual Friction Angles



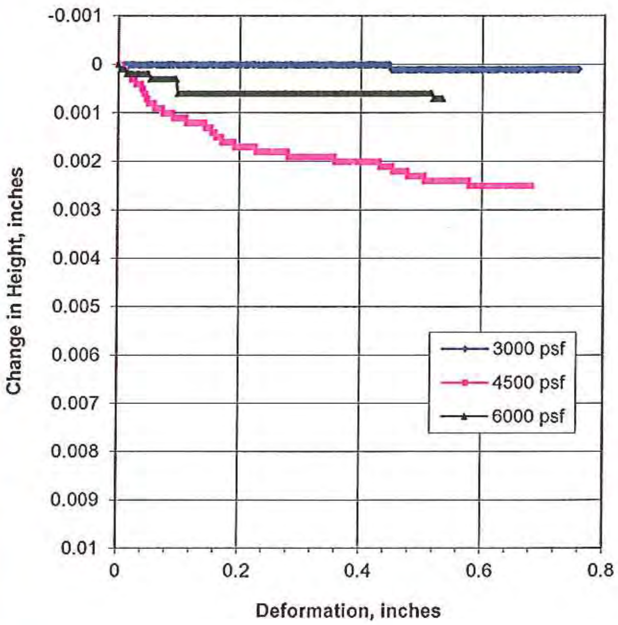
Strength Envelope



Deformation Curves



Vertical Deformation

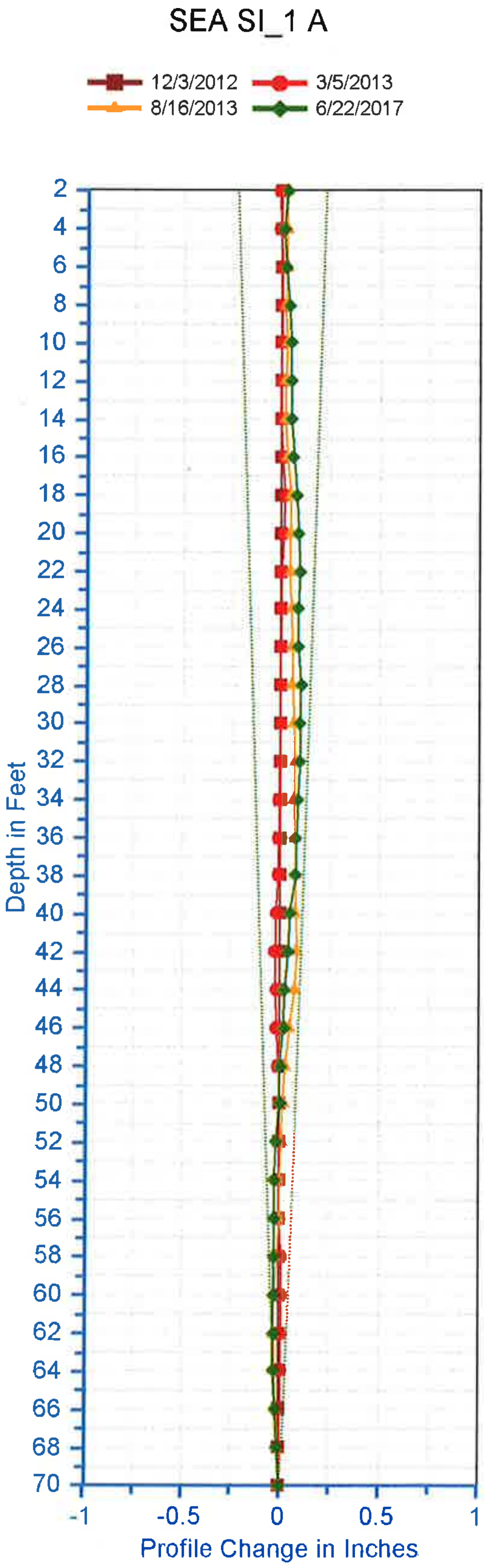


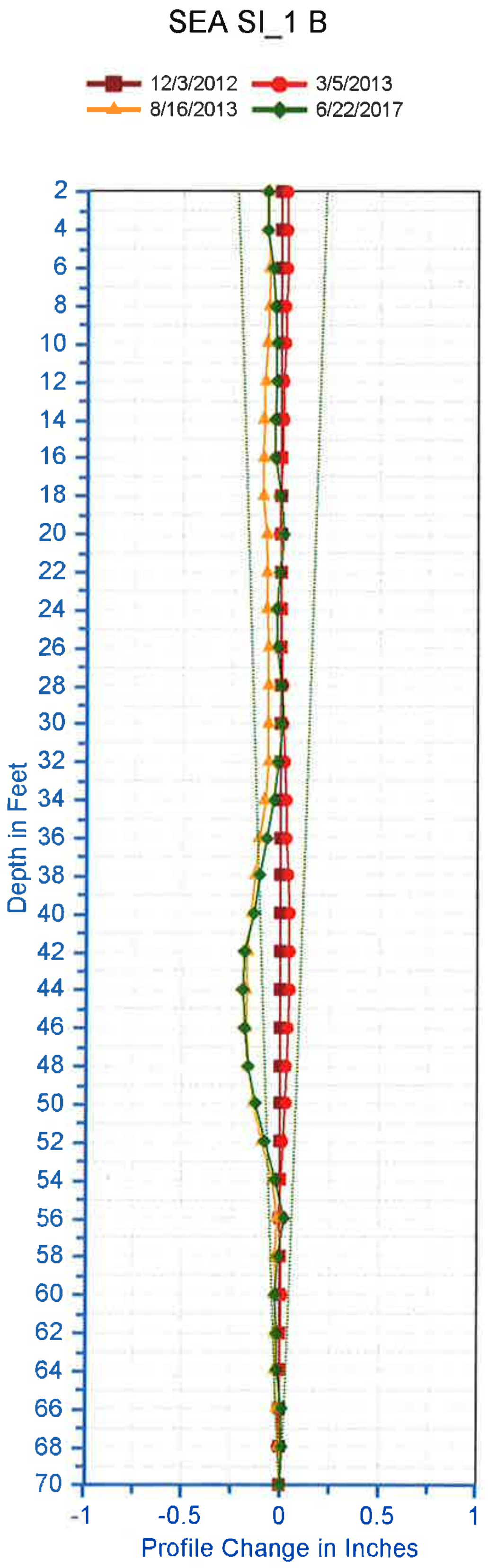
APPENDIX F

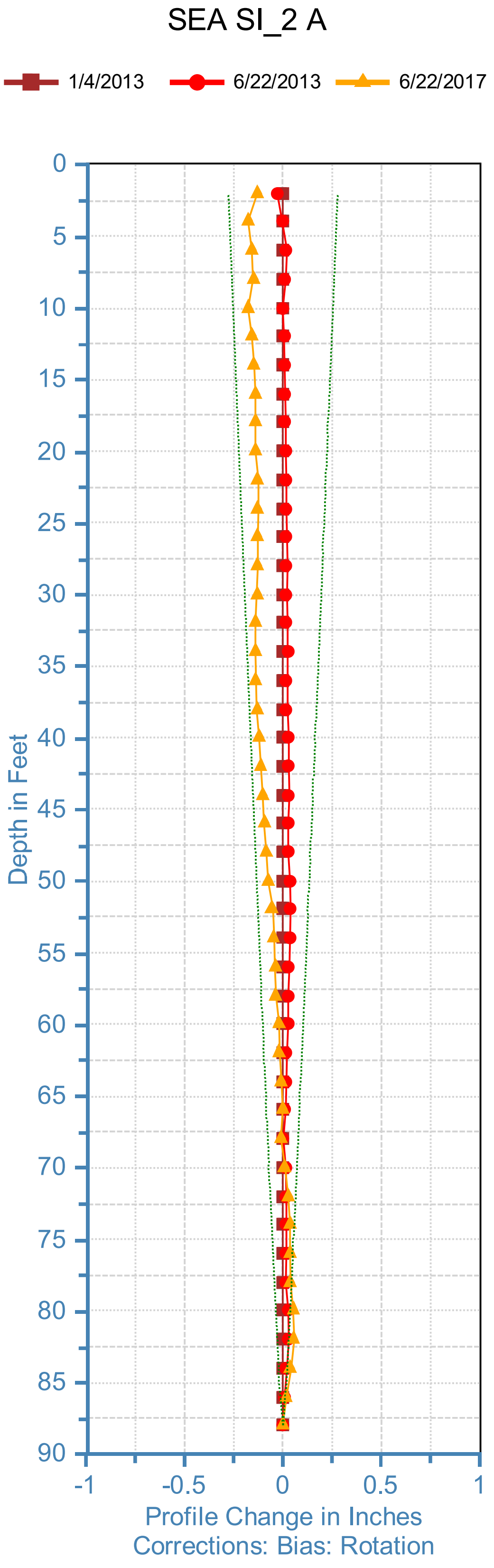
APPENDIX F

SLOPE INCLINOMETER PLOTS

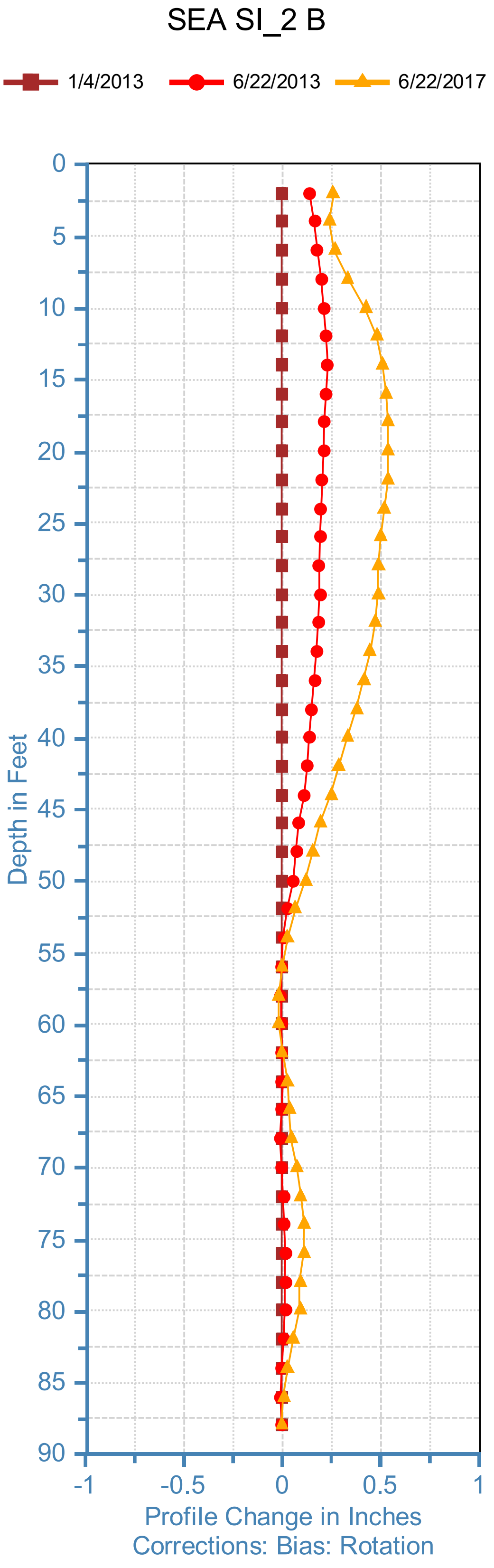
Four slope inclinometers were previously installed at the site (SI-1 through SI-3 by Geocon in 2012 and SI-4 by GRI in 2013). GeoDesign obtained additional readings on the slope inclinometers on June 22, 2017. Plots presenting the inclinometer data are presented in this appendix.


SEA SI_1 A
12/3/2012
3/5/2013
8/16/2013
6/22/2017
Depth in Feet
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
54
56
58
60
62
64
66
68
70
-1
-0.5
0
0.5
1
Profile Change in Inches

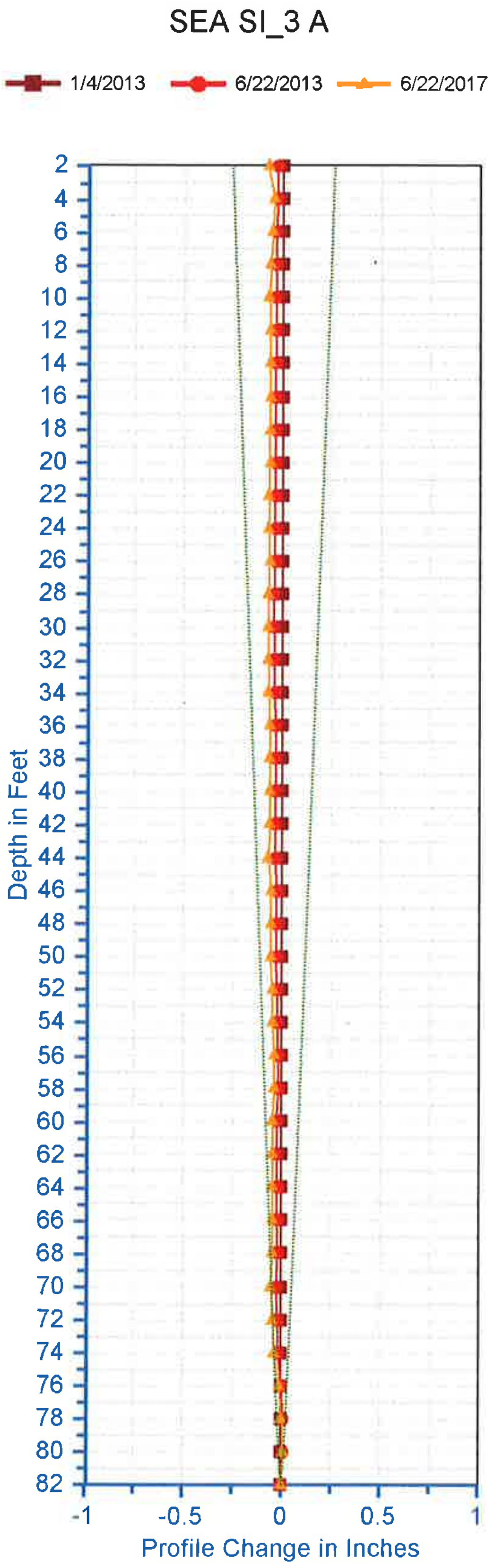

SEA SI_1 B
12/3/2012
3/5/2013
8/16/2013
6/22/2017
Depth in Feet
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
54
56
58
60
62
64
66
68
70
-1
-0.5
0
0.5
1
Profile Change in Inches

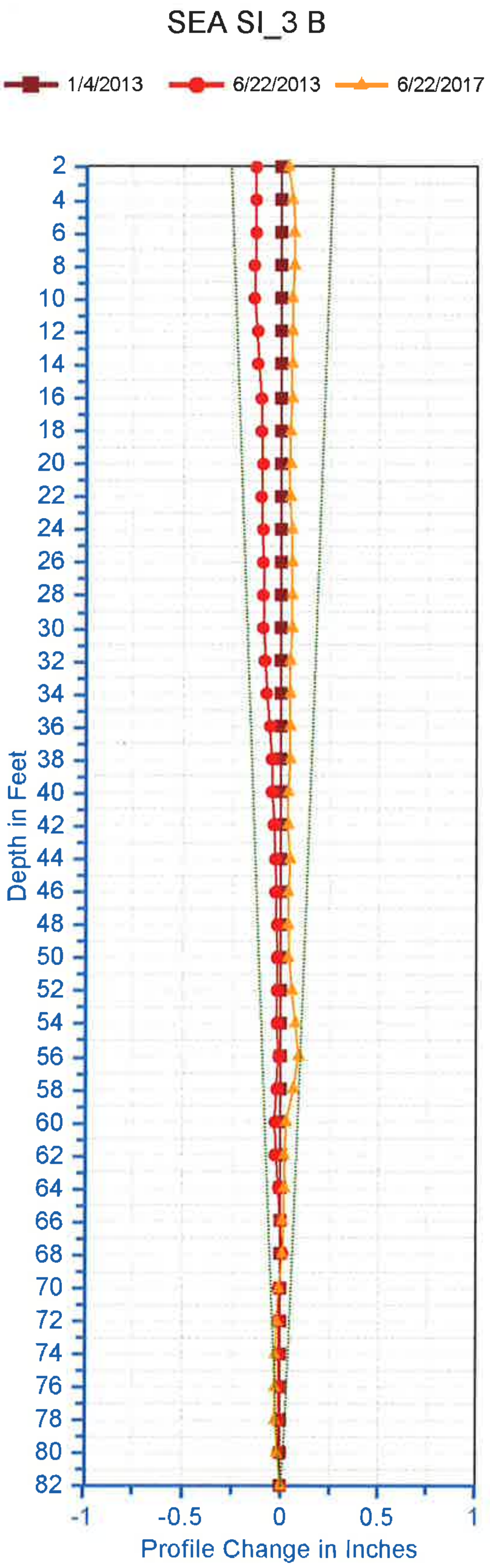


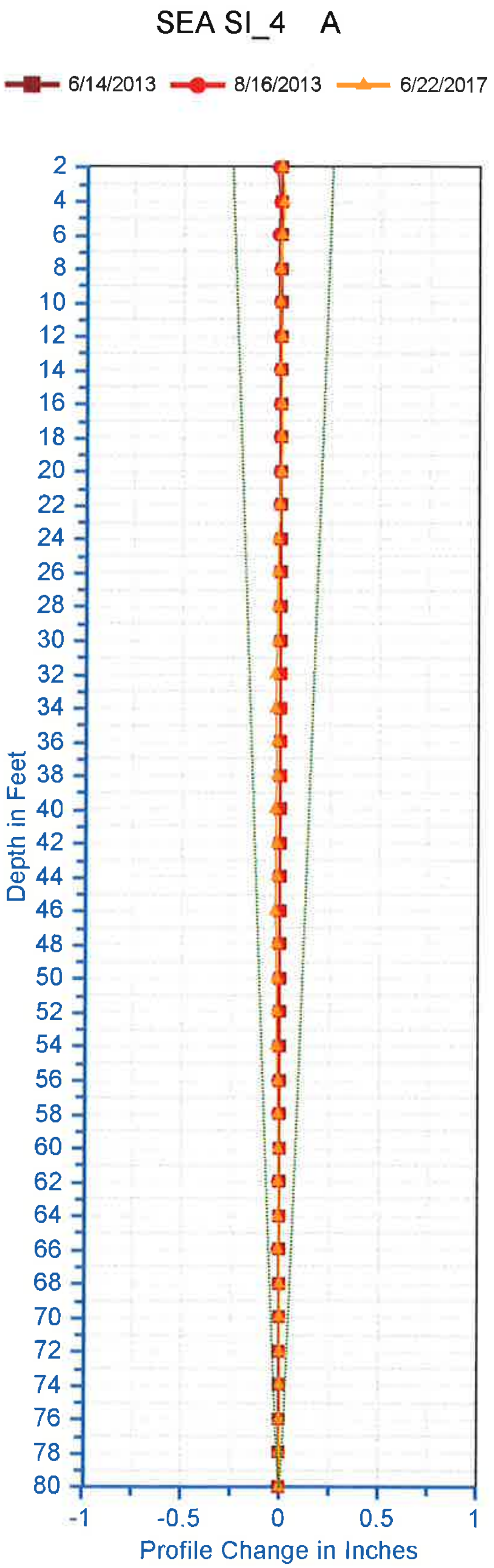
SEA SI_2 A

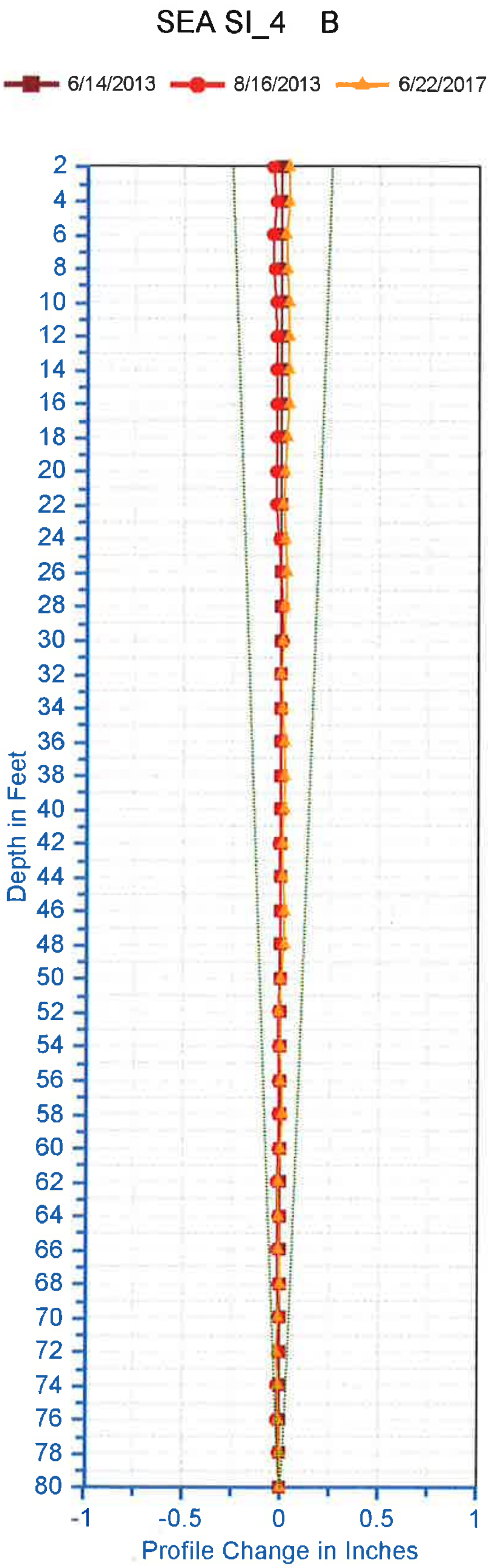


SEA SI_2 B


SEA SI_3 A
1/4/2013
6/22/2013
6/22/2017
Depth in Feet
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
54
56
58
60
62
64
66
68
70
72
74
76
78
80
82
-1
-0.5
0
0.5
1
Profile Change in Inches


SEA SI_3 B
1/4/2013
6/22/2013
6/22/2017
Depth in Feet
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
54
56
58
60
62
64
66
68
70
72
74
76
78
80
82
-1
-0.5
0
0.5
1
Profile Change in Inches


SEA SI_4 A
6/14/2013
8/16/2013
6/22/2017
Depth in Feet
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
54
56
58
60
62
64
66
68
70
72
74
76
78
80
-1
-0.5
0
0.5
1
Profile Change in Inches


SEA SI_4 B
6/14/2013
8/16/2013
6/22/2017
Depth in Feet
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
42
44
46
48
50
52
54
56
58
60
62
64
66
68
70
72
74
76
78
80
-1
-0.5
0
0.5
1
Profile Change in Inches

APPENDIX G

APPENDIX G

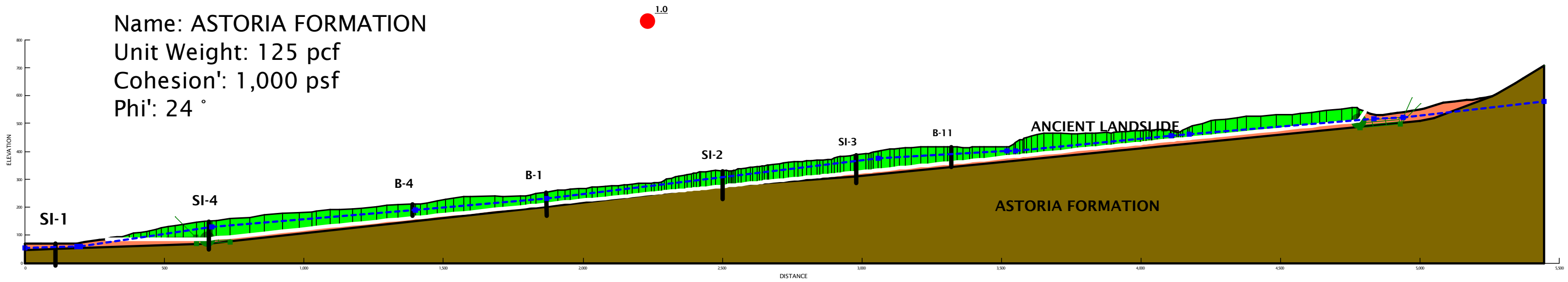
SLOPE/W SLOPE STABILITY ANALYSIS PLOTS

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

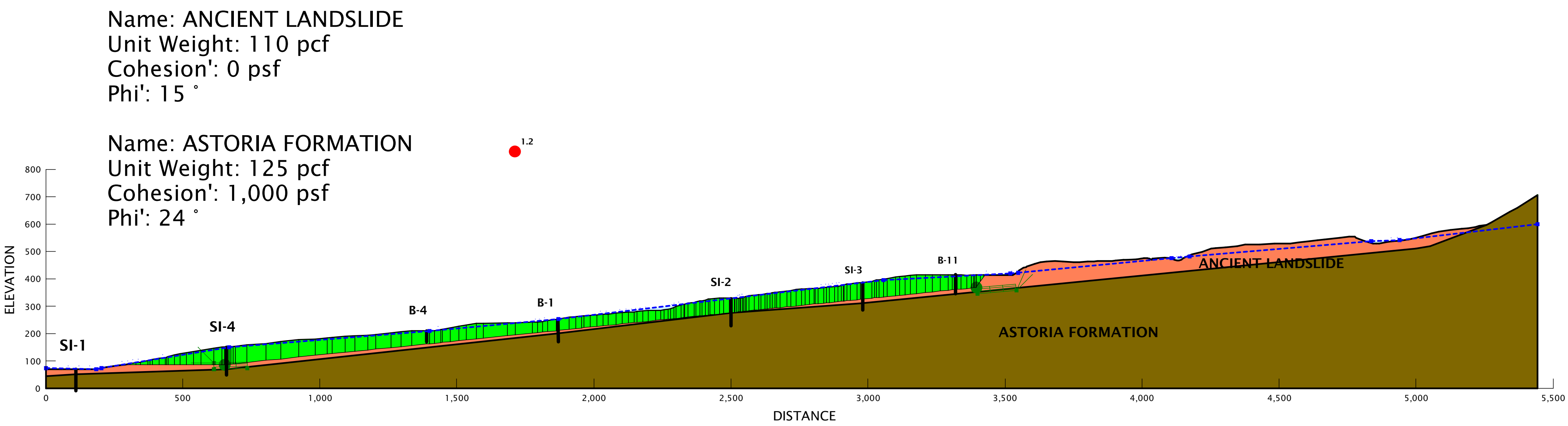
ANCIENT LANDSLIDE MODEL
BACKCALCULATED DYNAMIC FRICTION ANGLE
GROUNDWATER AVG 20 FEET BGS
SEISMIC LOAD: 0.2g

Name: ANCIENT LANDSLIDE
Unit Weight: 110 pcf
Cohesion': 0 psf
Phi': 22.5 °

Name: ASTORIA FORMATION
Unit Weight: 125 pcf
Cohesion': 1,000 psf
Phi': 24 °



ANCIENT LANDSLIDE MODEL
SIESMIC EFFECTIVE STRESS CONDITION
GROUNDWATER NEAR GROUND SURFACE
STATIC



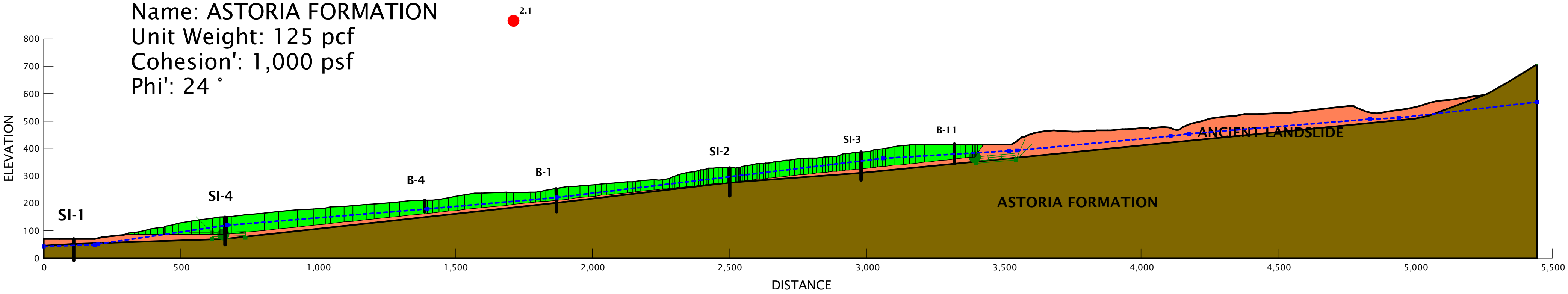
Name: ANCIENT LANDSLIDE
Unit Weight: 110 pcf
Cohesion': 0 psf
Phi': 15 °

Name: ASTORIA FORMATION
Unit Weight: 125 pcf
Cohesion': 1,000 psf
Phi': 24 °

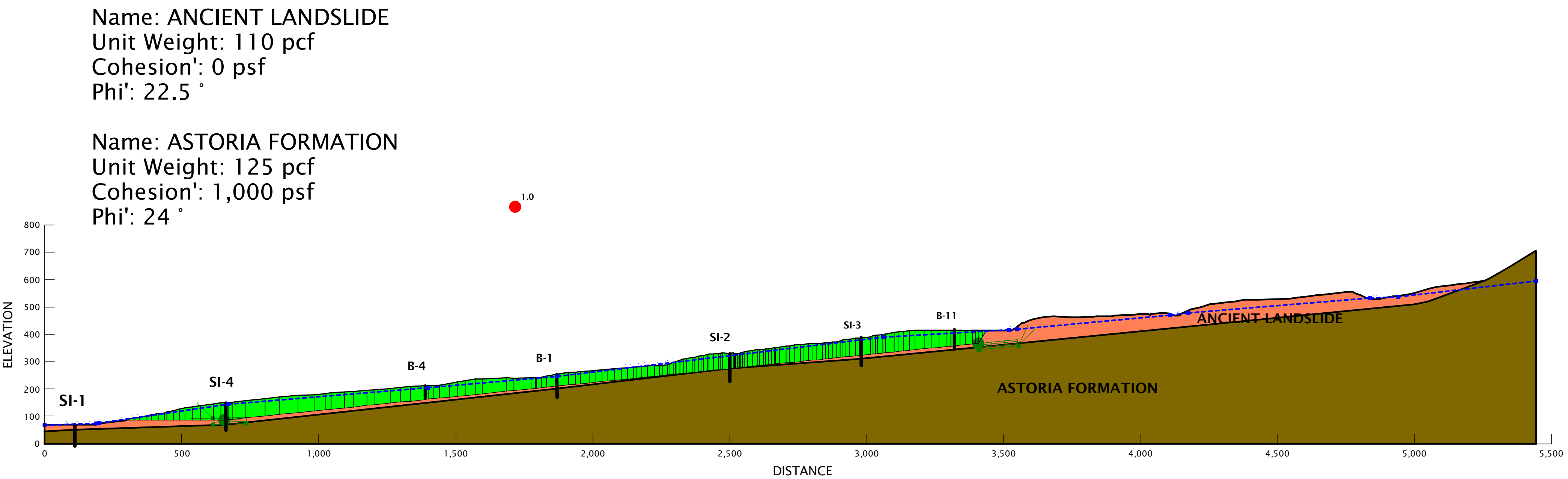
ANCIENT LANDSLIDE MODEL
SIESMIC EFFECTIVE STRESS CONDITION
GROUNDWATER AVG 30 FEET BGS
STATIC

Name: ANCIENT LANDSLIDE
Unit Weight: 110 pcf
Cohesion': 0 psf
Phi': 15 °

Name: ASTORIA FORMATION
Unit Weight: 125 pcf
Cohesion': 1,000 psf
Phi': 24 °



ANCIENT LANDSLIDE MODEL
GROUNDWATER ~5' FEET BGS
SIESMIC YIELD ACCELERATION 0.11G



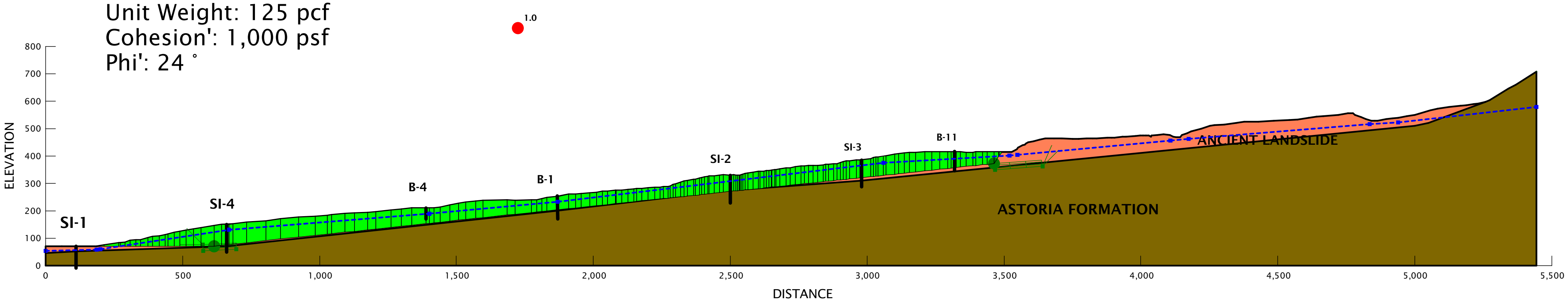
Name: ANCIENT LANDSLIDE
Unit Weight: 110 pcf
Cohesion': 0 psf
Phi': 22.5 °

Name: ASTORIA FORMATION
Unit Weight: 125 pcf
Cohesion': 1,000 psf
Phi': 24 °

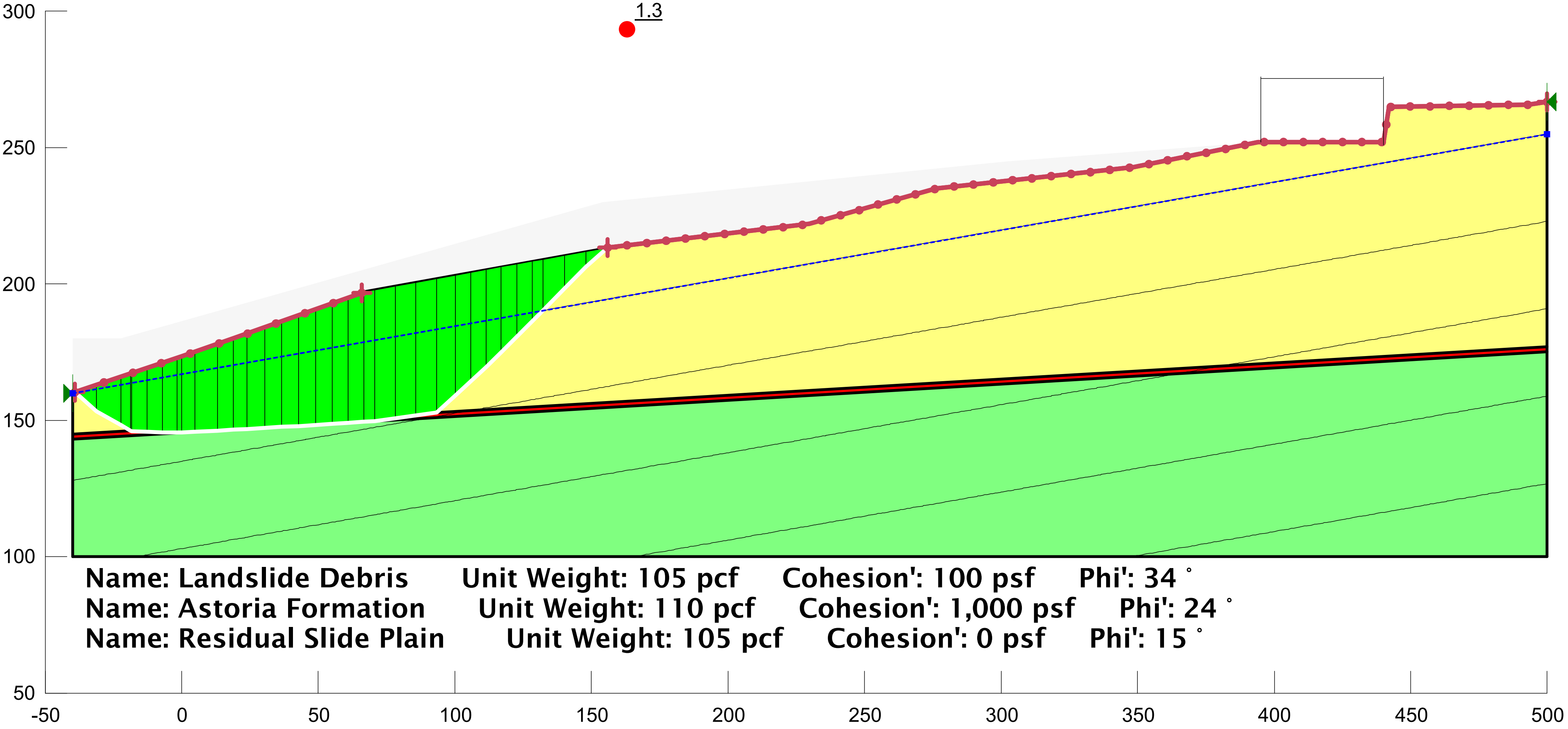
ANCIENT LANDSLIDE MODEL
GROUNDWATER ~20 FEET bgs
SEISMIC YIELD ACCELERATION 0.2g

Name: ANCIENT LANDSLIDE
Unit Weight: 110 pcf
Cohesion': 0 psf
Phi': 22.5 °

Name: ASTORIA FORMATION
Unit Weight: 125 pcf
Cohesion': 1,000 psf
Phi': 24 °

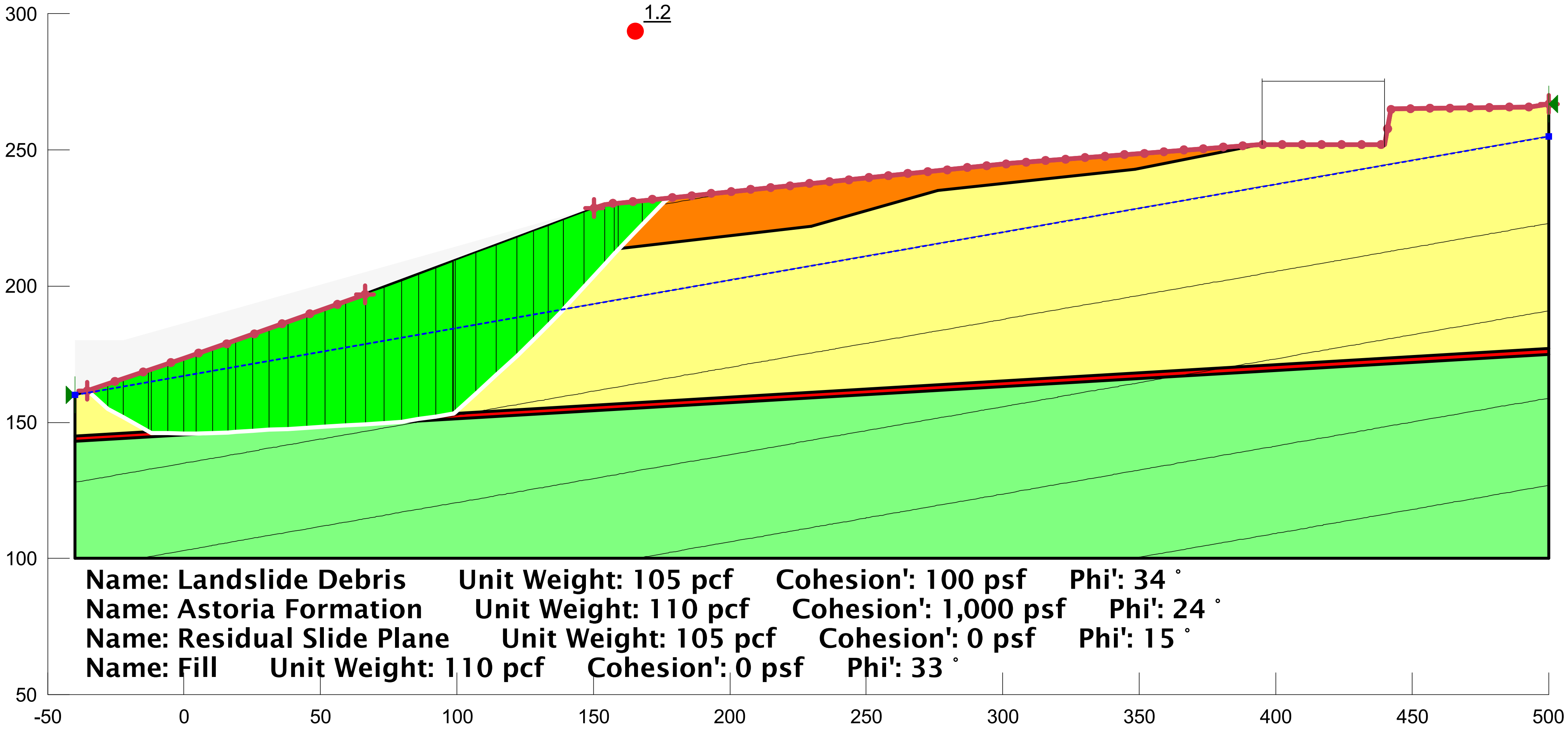


SLOPE BELOW PARKING AREA
STATIC



Name: Landslide Debris **Unit Weight: 105 pcf** **Cohesion': 100 psf** **Phi': 34 °**
Name: Astoria Formation **Unit Weight: 110 pcf** **Cohesion': 1,000 psf** **Phi': 24 °**
Name: Residual Slide Plain **Unit Weight: 105 pcf** **Cohesion': 0 psf** **Phi': 15 °**

PROPOSED SLOPE BELOW PARKING AREA
STATIC



Name: Landslide Debris **Unit Weight: 105 pcf** **Cohesion': 100 psf** **Phi': 34 °**
Name: Astoria Formation **Unit Weight: 110 pcf** **Cohesion': 1,000 psf** **Phi': 24 °**
Name: Residual Slide Plane **Unit Weight: 105 pcf** **Cohesion': 0 psf** **Phi': 15 °**
Name: Fill **Unit Weight: 110 pcf** **Cohesion': 0 psf** **Phi': 33 °**

APPENDIX H

APPENDIX H

SITE-SPECIFIC SEISMIC HAZARD EVALUATION

INTRODUCTION

The information in this appendix summarizes the results of a site-specific seismic hazard evaluation for the planned Seaside middle/high school in Seaside, Oregon. This seismic hazard evaluation was performed in accordance with the requirements of the 2014 SOSSC and ASCE 7-10.

SITE CONDITIONS

REGIONAL GEOLOGY

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

SUBSURFACE CONDITIONS

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

SEISMIC SETTING

Earthquake Source Zones

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

Regional Events

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

Two types of subduction zone earthquakes are possible and considered in this study:

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

An earthquake could occur on a local fault near the site within the design life of the facility. Figure H-1 shows the locations of faults with potential Quaternary movement within a 20-mile radius of the site. Figure H-2 shows the interpreted locations of seismic events that occurred between 1833 and 2014 (USGS, 2016). The most significant faults in the site vicinity are the Gales Creek fault and Tillamook fault. Table H-1 presents the closest mapped distance and mapped length of these faults.

Table H-1. Significant Crustal Faults

Source	Closest Mapped Distance[1] (km)	Mapped Length[1] (km)
Gales Creek	17.7	50
Tillamook	47	31

[1] Reported by USGS (USGS, 2016)

SEISMIC DESIGN PARAMETERS

Seismic design is prescribed by the 2014 SOSSC and 2015 IBC. Table H-2 presents the site design parameters prescribed by the 2015 IBC for the site. The building codes require that seismic design parameters associated with a percent probability of being exceeded in a 50-year period be used in design.

Table H-2. Seismic Design Parameters

Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
Spectral Acceleration, S (MCE)	S_s = 1.326 g	S_1 = 0.680 g
Site Class	D	
Site Coefficient, F	F_a = 1.000	F_v = 1.500
Spectral Acceleration Parameters, S_M (MCE)	S_{MS} = 1.326 g	S_{M1} = 1.020 g
Design Spectral Acceleration Parameters, S_D	S_{DS} = 0.884 g	S_{D1} = 0.680 g

Table H-2 represents the code-based requirements for the site. Because the building is described as an essential facility, a site-specific seismic evaluation is required for the project. The evaluation is described below. The seismic parameters provided below should be used for the design of the structures.

SITE-SPECIFIC SEISMIC HAZARD ANALYSIS

SITE AND ATTENUATION RELATIONSHIPS

Site Parameters

As described in “Subsurface Conditions” section, a CPT with seismic shear wave velocity was completed at the site. The CPT encountered refusal in the sedimentary rock beneath the site.

We estimated the shear wave velocity of the sedimentary rock using DOGAMI IMS-10 (1999). Based on the results of testing, a Vs_{30} of 745 ft/s (Site Class D in ASCE 7-10) was used for the site.

Because shear wave velocities were not directly measured in the sedimentary rock, three profiles were analyzed to capture the site sensitivity. Profile 1 used the assumed Vs_{30} of 745 ft/s. Profile 2 reduced the assumed Vs_{30} by 20 percent (600 ft/s). Profile 3 increased the assumed Vs_{30} by 25 percent (845 ft/s). A weighted average of the results of the site response (Profile 1 = 0.6, Profile 2 = 0.2, and Profile 3 = 0.2) were taken as the site response spectra for the site.

Attenuation Relationships

The level of seismic shaking at the site was determined using Next Generation Attenuation West 2 (NGA-West2). The values represent the average horizontal component considering 5 percent damping. We note that Idriss (2014) was not used in the analyses because it is not valid for sites with Vs_{30} less than approximately 1,300 ft/s. The attenuation relationships and weighting used in analysis is presented in Table H-3. In our opinion, the use of multiple attenuation relationships addresses epistemic uncertainty.

Table H-3. Attenuation Relationships Weights for Seismic Sources

Faulting Type	Ground Motion Prediction Equation	Weights
Shallow Faults and Shallow Crustal Background Seismicity	Abrahamson et al. (2014)	0.25
	Boore et al. (2014)	0.25
	Campbell and Bozorgnia (2014)	0.25
	Chiou and Youngs (2014)	0.25
	Idriss (2014)	0.0
Subduction (CSZ)	Zhao et al. (2006)	0.3
	BC Hydro (Abrahamson et al., 2016)	0.3
	Atkinson-Macias (2009)	0.1
	Atkinson and Boore (2003) Global Model	0.3
Deep Intraslab	Atkinson and Boore (2003) Cascadia Model	0.1667
	Zhao et al. (2006)	0.33
	BC Hydro (Abrahamson et al., 2016)	0.33
	Atkinson and Boore (2003) Global Model	0.1667

PROBABILISTIC SEISMIC HAZARD ANALYSIS

A site-specific PSHA was completed to produce hazard curves and uniform hazard spectra for the site using the software program EZ-FRISK 8.0 and fault source parameters described in the "Seismic Setting" section.

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 7-10 (C21.2). According to ASCE 7-10 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 (with averaging in between 0.2 and 1 second and between 1 and 1.5 second).

The results of the site response were also modified with risk coefficients using Method 1 outlined in ASCE 7-10 Section 21.2.1.2. A risk coefficient of C_{RS} = 0.827 was applied to the spectrum at periods of 0.2 second or less and a risk coefficient of C_{R1} = 0.824 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.

Figure H-3 shows the PSHA MCE_R for the three profiles analyzed as well as the weighted average MCE_R.

DETERMINISTIC MCE_R RESPONSE SPECTRUM

Per ASCE 7-10 Section 21.2.2, the deterministic MCE is the envelope of the 84th percentile spectral ordinates of the DSHA faults considered; however, the ordinates of the response spectrum must not be taken as lower than the corresponding ordinates of the response spectrum determined in accordance with Figure 21.2-1 of ASCE 7-10.

A DSHA was completed using the same ground motion models and site parameters described in the PSHA. The DSHA ordinates were modified to represent the MRC using the methodology described above. We note that risk coefficients are not included in the DSHA. Figure H-4 shows the DSHA from analysis and the deterministic lower limit.

SITE-SPECIFIC MCE_R RESPONSE SPECTRUM

As outlined in ASCE 7-10 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. Figure H-4 shows the site-specific MCE_R for the site.

DESIGN RESPONSE SPECTRUM

ASCE 7-10 Section 21.3 states that the site-specific MCE_R response spectrum is reduced to two-thirds of the acceleration at any period. However, the lower bound for design ground motions is 80 percent of the generalized response spectrum as outlined in ASCE 7-10 Section 11.4.5. The site-specific response spectrum and generalized response spectrum are shown on Figure H-5.

DESIGN ACCELERATION PARAMETERS

The parameter S_{DS} is taken from the site-specific response spectrum at a period of 0.2 second but shall not be smaller than 90 percent of the peak spectral acceleration taken at any period larger than 0.2 second. The parameter S_{D1} is taken as the greater of the spectral acceleration at 1 second or 2 times the acceleration at 2 seconds. Figure H-5 shows the design response spectrum for the project. The values of S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}. Based on this discussion, the site-specific design parameters are as follows:

- S_{DS} = 1.065 g
- S_{D1} = 0.898 g
- S_{MS} = 1.598 g
- S_{M1} = 1.347 g

FAULT SURFACE RUPTURE

The closest know active fault to the site is more than 10 miles away. Consequently, it is our opinion that the probability of surface fault rupture beneath the site is low. We note that there is the potential for other non-active and/or unknown faults in the area that may have the potential to rupture.

LIQUEFACTION AND LATERAL SPREADING

The main report provides a discussion of liquefaction and lateral spreading potential 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 explorations. 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 and drainage recommendations to limit the risk of landslides for the proposed development at the site. Based on the site conditions, geologic conditions, and our analyses, a design-level Magnitude 9.0 subduction zone earthquake could result in up to 2 feet of movement of the ancient massive landslide encompassing the site. Accordingly, we have recommended supporting the proposed middle/high school building on a reinforced mat foundation that will reduce the risk of differential movement, protecting life-safety and reducing the potential for structural damages to the building.

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. The site is located within the uplift portion of plate during strain accumulation. Upon coesiemic strain release the uplifted portion rapidly subsides.

The DOGAMI Coseismic Subsidence Map for Simulated Magnitude 9 Cascadia Earthquake: Clatsop County, Oregon, indicates up to 4 to 5 feet of subsidence is possible at the site. Based on our review of published subsidence estimates for the CSZ by Hawkes et al. (2011), the anticipated coastal subsidence during a large M9.0 CSZ event for the Nehalem River approximately 18 miles south of the site is estimated to be 1.6 feet with an error of +/- 1.0 foot.

Accordingly, subsidence is expected at the site for CSZ events, but the magnitude of subsidence is difficult to accurately estimate and will depend on the earthquake characteristics.

TSUNAMI

The proposed building, parking areas, and track and field locations are not within a tsunami inundation zone according to the Tsunami Inundation Map Clat-08 Plate 1 from DOGAMI.

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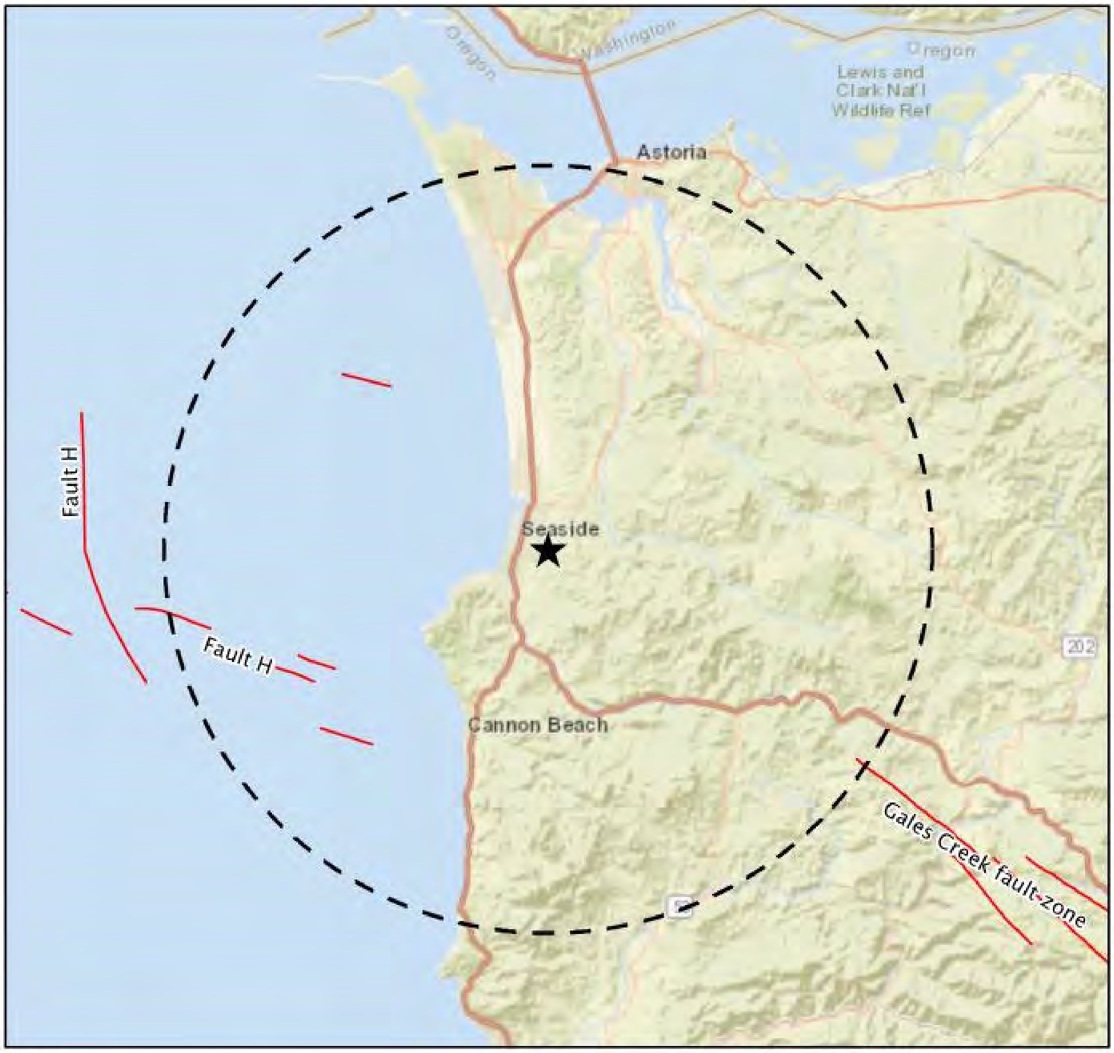
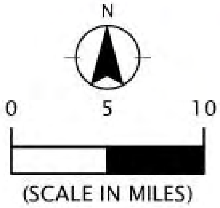


LEGEND

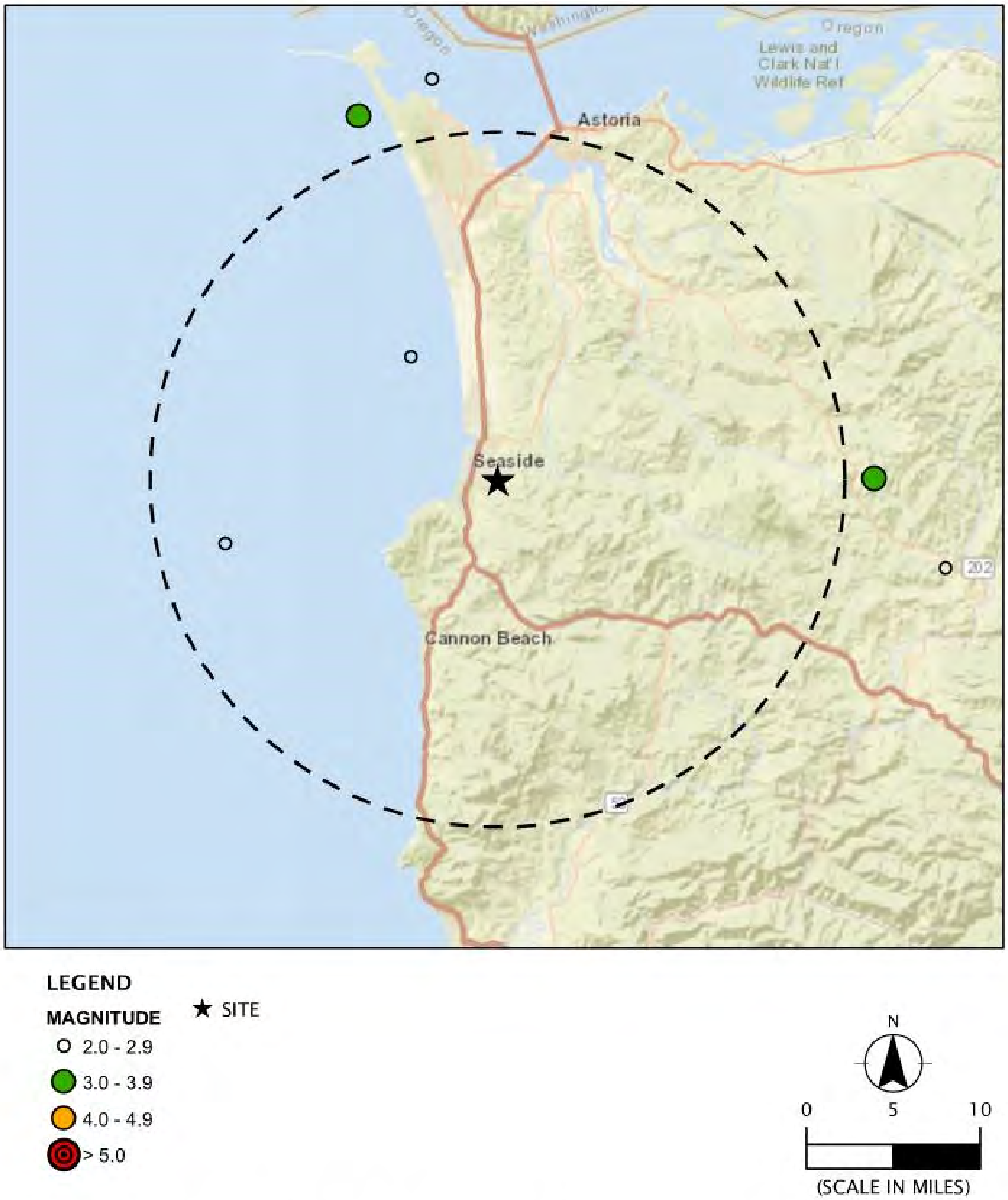
- QUATERNARY FAULT
- ★ SITE



FAULTS PROVIDED BY THE USGS (2006) QUATERNARY FAULT AND FOLD DATABASE, ACCESSED 9/21/17 FROM: http//earthquakes.usgs.gov/hazards/qfaults/

SeasideSD-1-03-01-FH1_H2.docx Print Date: 11/18/17

GEODESIGN INC 9450 SW Commerce Circle - Suite 300 Wilsonville OR 97070 503.968.8787 www.geodesigninc.com	SEASIDESD-1-03-01	**QUATERNARY FAULT MAP**	
	NOVEMBER 2017	SEASIDE SCHOOL DISTRICT CAMPUS SEASIDE, OR	**FIGURE H-1**



HISTORICAL EARTHQUAKES 1898–1969 PROVIDED BY THE ANSS COMPREHENSIVE CATALOG, NCEDC (2014)
HISTORICAL EARTHQUAKES 1969–2017 PROVIDED BY THE PNSN AUTHORATIVE CATALOG (2017)

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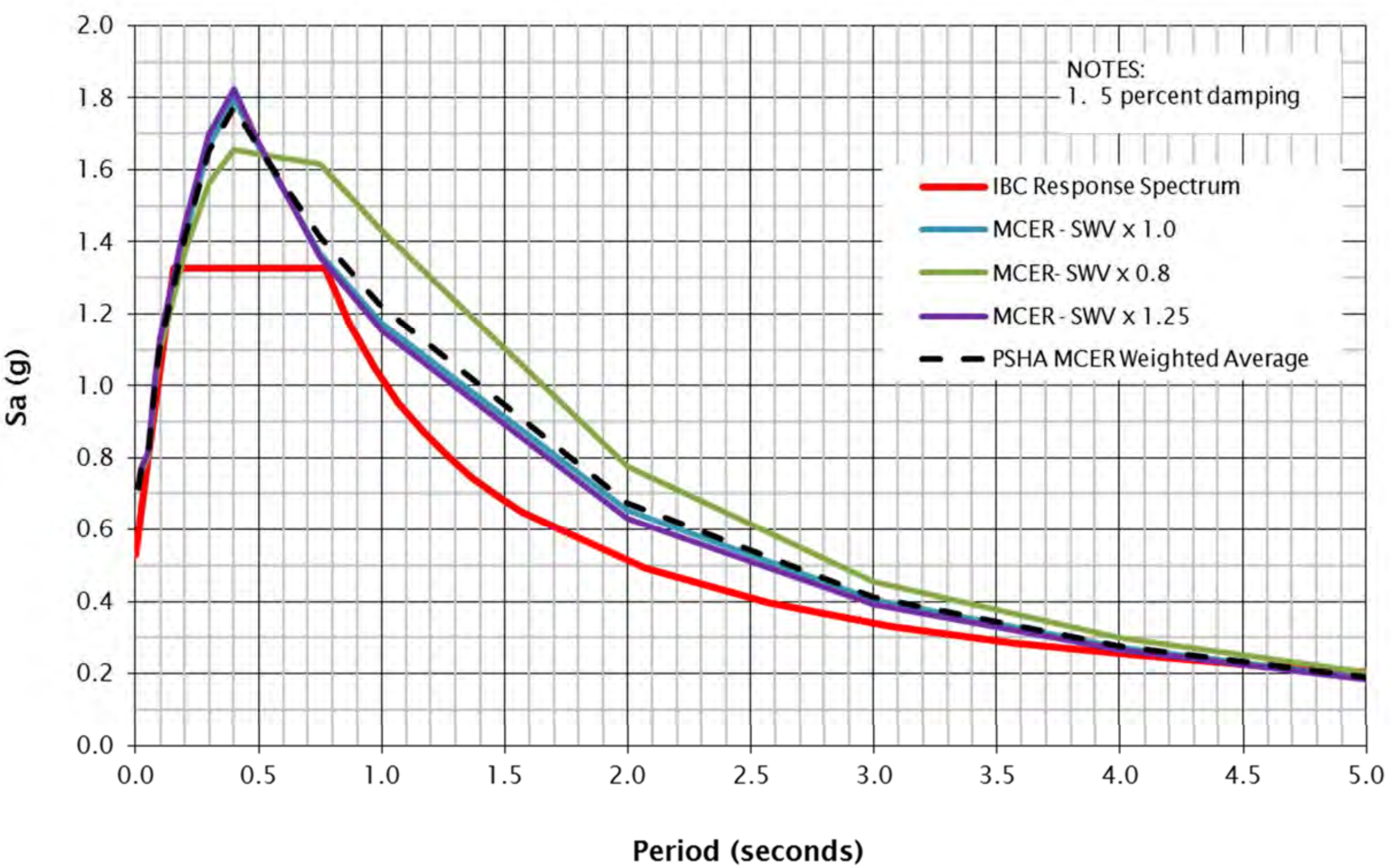
HISTORICAL SEISMICITY MAP

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE H-2



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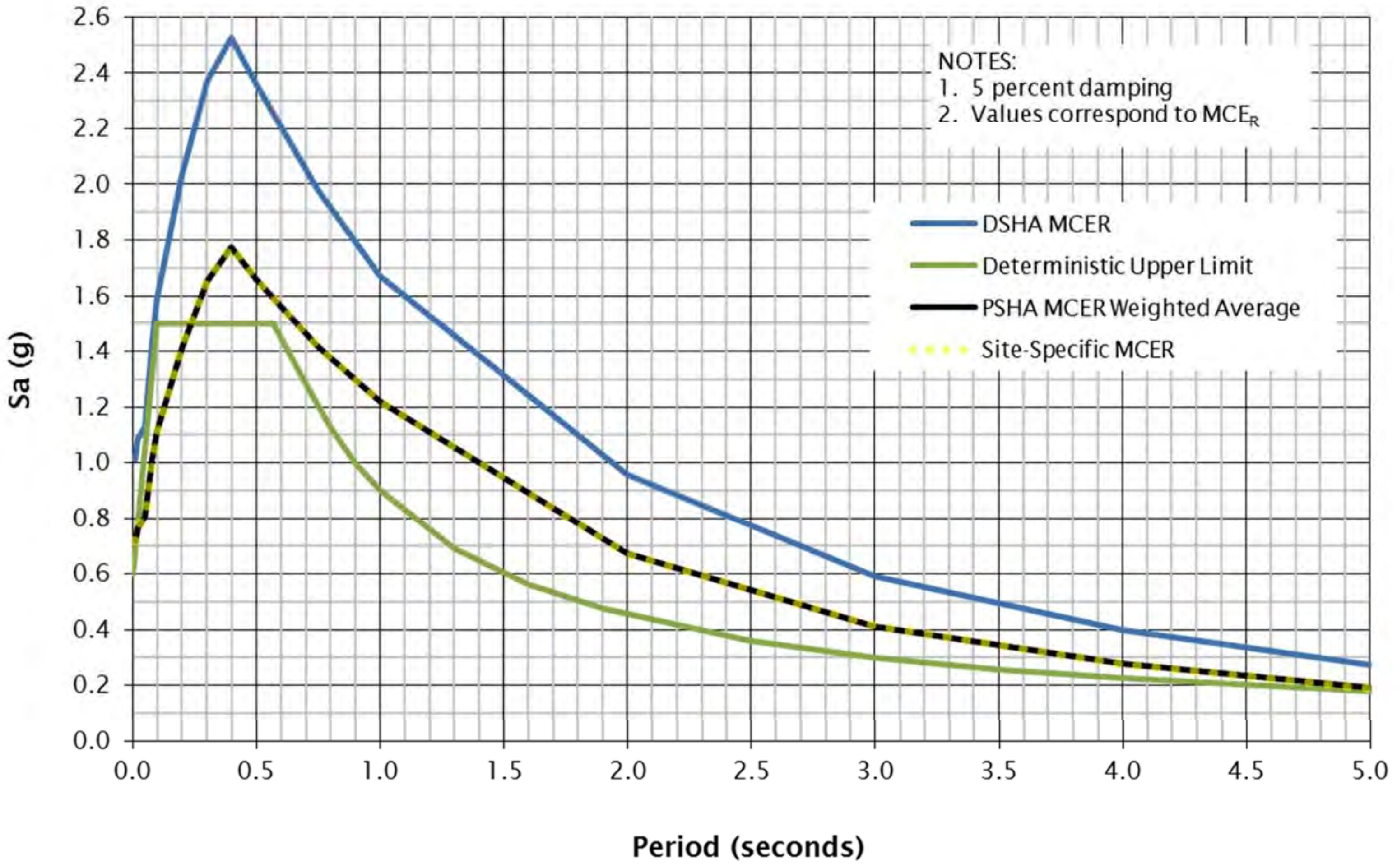
SITE RESPONSE SPECTRA

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE H-3



NOTES:
1. 5 percent damping
2. Values correspond to MCE_R

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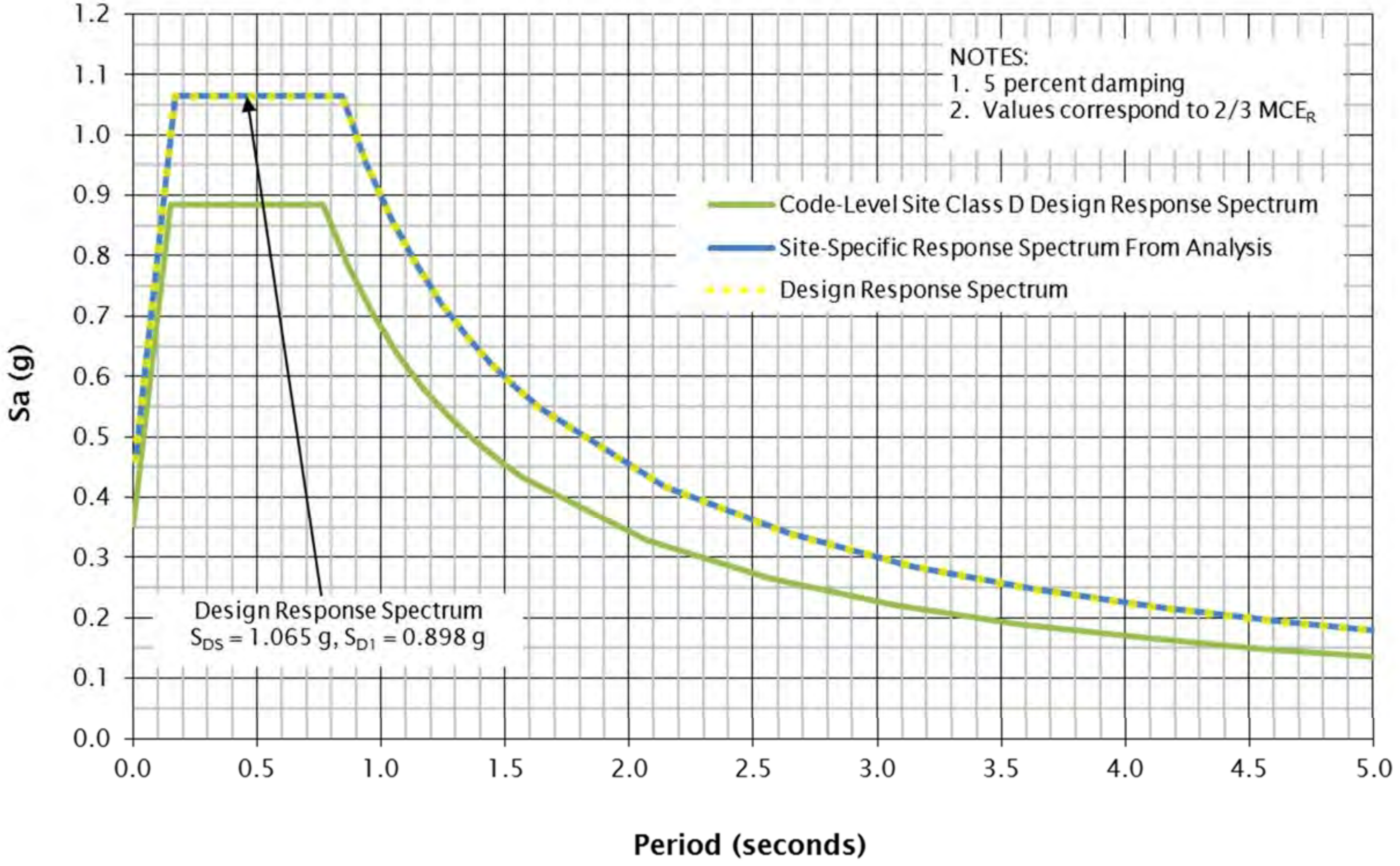
SITE-SPECIFIC RESPONSE SPECTRA

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE H-4



NOTES:
1. 5 percent damping
2. Values correspond to 2/3 MCE_R

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DESIGN RESPONSE SPECTRUM

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE H-5

ACRONYMS AND ABBREVIATIONS

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 penetrometer test
CRBG	Columbia River Basalt Group
CSZ	Cascadia Subduction Zone
DOGAMI	Oregon Department of Geology and Mineral Industries
DSHA	deterministic seismic hazard analysis
ESAL	equivalent single-axle load
ft/s	feet per second
g	gravitational acceleration (32.2 feet/second^2)
GPS	global positioning system
H:V	horizontal to vertical
IBC	International Building Code
km	kilometers
LiDAR	light detection and ranging
MCE	maximum considered earthquake
MCE_R	risk-targeted maximum considered earthquake
MRC	maximum rotated component
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Standard Specifications for Construction (2018)
PCC	Portland cement concrete
pcf	pounds per cubic foot
pci	pounds per cubic inch
PG	performance grade
PGA	peak ground acceleration
psf	pounds per square foot
PSHA	probabilistic seismic hazard analysis
psi	pounds per square inch
PVC	polyvinyl chloride
SOSSC	State of Oregon Structural Specialty Code
SPT	standard penetration test
USGS	U.S. Geological Survey

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