REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Seaside Heights Elementary School Expansion
2000 Spruce Drive
Seaside, Oregon

For
Seaside School District
November 6, 2017

GeoDesign Project: SeasideSD-1-03-02

November 6, 2017

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

Attention: Justine Hill

Report of Geotechnical Engineering Services
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Seaside, Oregon
GeoDesign Project: SeasideSD-1-03-02

GeoDesign, Inc. is pleased to submit this report of geotechnical engineering services for the proposed Seaside Heights Elementary School expansion at 2000 Spruce Drive in Seaside, Oregon. The proposed expansion includes a new two-story building northwest of the existing building. Our services for this project were conducted in accordance with our confirming agreement dated September 7, 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

cc: Mitali Kulkarni, Day CPM Services (via email only)
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1.0 INTRODUCTION

GeoDesign, Inc. is pleased to present this geotechnical engineering report for the proposed Seaside Heights Elementary School expansion. The proposed expansion includes a new two-story building northwest of the existing building, a cut wall for improvements northeast of the existing building, and a new driveway and roundabout in an existing pavement area on the southwest side of the existing building. Figure 1 shows the site location relative to existing topographic and physical features. The approximate location of the planned new building is 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 two-story building are anticipated to be less than 100 kips and 8 kips per foot, respectively. Most of the planned building will be near existing grades; however, we understand cuts of up to approximately 9 feet will be required for the northwest corner of the building and fills of up to 7 feet will be required for the southwest corner of the building. A cut wall ranging in height up to approximately 15 feet is also planned north of the existing school building to create relatively flat play areas.

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.
- Completed the following subsurface explorations at the site:
    - Three borings to depths between 16.5 and 46.5 feet BGS.
    - One CPT probe advanced to practical refusal at a depth of 48.1 feet BGS
    - Shear wave velocity testing at 2-meter intervals in the CPT probe
    - Five test pits to depths between 5.0 and 11.0 feet BGS near or on the slopes around the elementary school area
- Collected soil samples at select depths in the explorations.
- Classified the materials encountered in, and maintained a detailed log of, each exploration.
- Completed the following laboratory tests on selected soil samples:
    - Thirty-one moisture content determinations in general accordance with ASTM D 2216
    - Three dry density determinations in general accordance with ASTM D 7263
    - Three 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 consolidation test in accordance with ASTM D 2435
    - Two expansion index tests in general accordance with ASTM D 4829
    - One set of direct shear tests in general accordance with ASTM D 3080

- 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.
- 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

The Seaside Heights Elementary School area was initially graded by cutting from higher areas and filling to lower areas to create a relatively flat bench in 1972. The existing Seaside Heights Elementary School building has experienced distress from the shrink and swell of soils supporting the shallow foundations. We understand modifications were made to the drainage systems at the site and a French drain may have been installed between the school and upper slopes sometime after 1987 to help limit the shrink-swell potential of the soils.

A 1987 geotechnical report (Kelly/Strazer, 1987) indicates areas of apparent or possible slope movement for the fill slope southwest of the existing school and the cut slope north of the existing school where the cut wall is proposed. Slope failures have since been documented for the fill slope to the southwest of the existing school in our 2011 report (GeoDesign, 2011). An initial older failure was interpreted to have occurred 5 to 10 years prior with a head scarp of up to 5 feet high located near the middle of the slope. Heavy rain in 2011 reactivated the failure and created a new head scarp that extends further upslope to near the edge of the fence and pavement approximately 50 feet from the closest edge of the school building. The toe of the landslide appeared to be at or just above the slope break at the bottom edge of the fill prism.

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 is mapped near the contact of Quaternary alluvial deposits and Tertiary marine sedimentary bedrock consisting of the Cannon Beach Member of the Astoria Formation (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. Local basalt intrusions subsequently disrupted and transported downslope by the ancient landslide and colluvial action may account for the basalt boulders present on the site.

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 approximately 1 mile east 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 existing Seaside Heights Elementary School is located at the east end of Spruce Drive. The existing single-story school and surrounding pavements, playgrounds, and parking area are situated at an elevation of roughly 70 feet. A playground area and basketball court are located northwest of the school building, and a parking area is located southeast of the school building. A modular classroom building is located near the edge of the pavement area on the north side of the existing school. The site is situated on a large hillside generally sloping east to west. The relatively flat, developed area was graded with cut slopes on the east side and fill slopes to the southwest. The eastern cut slopes have grades ranging from roughly 2.5H:1V to 3H:1V and transition to flatter, native slopes above. The fill slope to the southwest of the developed area has an average grade of roughly 2.5H:1V, with some localized steeper areas near the top of the slope and landslide headscarps, which are discussed in the "Background" section. The toe of the fill slope is located in a flat stream drainage and wetland area. A much flatter fill slope of roughly 4H:1V to 5H:1V is located near the west corner and leads down to a lower, relatively flat, filled area, which was formerly used for a soccer field but is now overgrown. A small nob rises up approximately 15 feet to the northwest of the developed area.

4.3 SUBSURFACE CONDITIONS

4.3.1 General

We explored subsurface conditions by drilling three borings (B-6 through B-7) to depths between 16.5 and 46.5 feet BGS, excavating five test pits (TP-27 through TP-31) to depths between 5.0 and 11.0 feet BGS, and advancing one CPT probe (CPT-1ES) to refusal at a depth of 48.1 feet BGS. Other explorations completed nearby upslope by GeoDesign included three test pits (TP-1, TP10, and TP-11) and a CPT probe (CPT-7a), and past explorations completed at or near the site by others included test pits and a boring. 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 results are presented in Appendix C. Logs of GeoDesign's nearby test pits and CPT probe are presented in Appendix D. Logs of past explorations completed by others are presented in Appendix E.

In general, the soil conditions encountered at the site consist of undocumented fill where present overlying decomposed to weathered claystone of the Astoria Formation. A 7- to 11-foot-thick zone of clay with variable sand and gravel interpreted as colluvium was also encountered below the fill and above the claystone in borings B-6 and B-8. The following sections provide a more detailed description of the subsurface conditions encountered.

4.3.2 Undocumented Fill

The undocumented fill generally consists of soft to medium stiff clay with variable amounts of sand and gravel. The clay transitions to clayey gravel at the base of the fill in boring B-7. The clay exhibits high plasticity. Tested moisture contents for the clay range from 31 to 62 percent; however, a moisture content of 101 percent was tested for one sample of fill from test pit TP-29. Loose or medium dense gravel fill was also encountered from the surface to 3.0 feet BGS in boring B-6 and to 6.5 feet BGS in boring B-8. Fill soil was encountered to a maximum depth of 32.0 feet BGS in boring B-8 and was encountered to a maximum depth of 12.0 feet near the west end of the proposed new building in boring B-7. Direct shear testing on a sample from B-8 at 15.0 feet BGS indicates a friction angle of 31 degrees and cohesive strength of 300 psf for the clay fill. The tested dry densities of the fill were 72 and 78 pcf. Based on the consistency and densities, the clay fill is expected to have moderate to high compressibility. The fill is also expected to have moderate to high expansion potential considering it exhibits high plasticity and was derived from cuts from soil with swell potential at the site.

4.3.3 Colluvium

The colluvium at the site generally consists of medium stiff to stiff clay with variable proportions of sand and gravel. The tested moisture content of the clay colluvium ranges from 35 to 61 percent. The clay exhibits high plasticity.

4.3.4 Astoria Formation

Decomposed to weathered claystone of the Astoria Formation was generally encountered at the site below the fill and/or colluvium where present. Siltstone was also observed for portions of the Astoria Formation. CPT-1ES indicates the formation is mostly clay but transitions back and forth between more silty zones. The decomposed claystone and siltstone consists of medium stiff to very stiff silt and clay. The weathered claystone and siltstone is very stiff to hard clay and silt by soil classification methods as used in the most of the logs but would be termed extremely soft to soft by rock classification methods. Tested moisture contents of the decomposed to weathered claystone and siltstone varies from 23 to 68 percent. Atterberg limits testing indicates the clay and silt exhibits high plasticity. Expansion index testing of a sample from TP-31 at 3.0 feet BGS and composite sample from B-7 at 15.0 to 25.0 feet BGS indicates the clay has medium to high expansion potential. A consolidation test on the stiff to very stiff silt (decomposed siltstone) from TP-30 at 1.0 foot BGS did not experience any significant swelling under the seating load of 100 psf when the sample was saturated and also indicated the soil is over-consolidated. Based on the CPT probe and borings, the consistency of the Astoria Formation fluctuates somewhat but generally becomes stronger with depth.

4.3.5 Groundwater

Slow perched groundwater seepage was encountered in test pit TP-31 at 8.0 feet BGS. Groundwater was not observed in any of our other explorations at the site. Groundwater has been measured at 4.0 to 6.0 feet BGS with piezometers installed in boring SI-1 near the base of the cut slope at the east edge of the parking lot. Seepage has also been observed in past explorations at the site along the contact of fill and native soils. Based on our observations and research, we anticipate perched water can be encountered at shallow depths below the ground surface and at contacts between fill and native soils at the site, particularly 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.

5.0 CONCLUSIONS

We encountered up to 12 feet of undocumented fill in boring B-7 near the southwest corner of the proposed new building. The upper 9 feet of the fill encountered consists of soft to medium stiff clay. The soft fill has insufficient strength and high compressibility; therefore, we recommend removing any soft fill from the planned building area or where fill needs to be placed to achieve grades for the planned building area. On-site soil compacted as structural fill without lime amendment will have high swell potential; therefore, new fill required for the south end of the building should be constructed out of imported granular material. Alternately, on-site soil can be lime amended for placement as structural fill below the building, provided a minimum 12-inch-thick layer of imported granular material overlain by a drainage geotextile is placed for drainage at the base of the fill. Lime amendment of the on-site material as recommended in this report will significantly decrease the swell potential of the fill. Fills on slopes steeper than 5H:1V will also require benching for placement.

Shrink-swell damage has been documented as an ongoing issue for the existing school building at the site. The soils at the site exhibit high plasticity, and two expansion index tests conducted on the very stiff to hard clay from near the building area indicate the soil has medium to high expansion/swell potential; however, significant swelling was not observed under the seating load when a relatively undisturbed sample was inundated with water during our consolidation testing. Testing completed in 1987 on four undisturbed samples from the existing building area also indicate soils at the site have medium to high swell potential. Accordingly, we recommend supporting the new school building on a mat foundation to resist potential differential movements from shrinking and swelling of the soils. Design of the mat reinforcement should consider that it may be subjected to swell pressures at the base of the mat. We also recommend installing a minimum 6-inch-thick layer of crushed rock base for the mat foundation and installing a foundation drain around the perimeter of the building, which extends deeper than the crushed rock base layer, to provide drainage and help reduce fluctuations in the moisture content of the soils below the building.

As an alternate to a mat foundation, the building could be supported on spread footings, provided a minimum of 2 feet of soil below the lowest footing subgrade elevation is excavated and replaced with imported granular material or lime amended to reduce the swell potential. For this approach additional laboratory testing should be conducted on undisturbed soil samples prior to construction to evaluate if 2 feet of granular fill or soil treatment provides sufficient overburden to resist swelling from underlying soils or if greater depths are necessary to resist swelling potential at the site.

Preliminary grading plans provided by Cameron McCarthy indicate a wall ranging up to 15 feet high is planned to create a relatively flat cut for a new playground area and structures north of the existing school and where the modular classroom building is currently located. Our test pits and CPT-7a on the slope at or near this area indicate the subsurface silt and clay is stiff to hard and exhibits high plasticity. Basalt cobbles and boulders were observed in the silt and clay in test pits TP-10 and TP-11 but were not encountered in test pits TP-27 and TP-28 at or near the planned cut area. Cobbles and boulders that may have originated from the prior slope cuts are also piled near the toe of the existing slope cut. Groundwater was measured upslope at a depth of 43.9 feet BGS in CPT-7a and groundwater was not observed in our test pit explorations on the slope. The area is near the toe of the ancient landslide terrain, which extends roughly 1 mile upslope to the east of the site. The proposed cuts are very small relative to the large scale of the ancient landslide terrain and will be in competent soils based on our explorations. However, since the area is near the toe of the ancient landslide terrain, there is a potential for localized instability associated with cuts. We recommend planning the layout of the improvements and terracing the grades to limit cuts to less than 15 feet. The upslope conditions and ancient landslide will be discussed further in our forthcoming report for the planned middle school and high school campus. We also recommend permanent cut slopes for the cut playground area not exceed the current approximate maximum cut slope grade of 2.5H:1V. Drainage will be critical for the area and should be provided behind all walls as recommended in this report. GeoDesign should be contacted to review and comment prior to finalizing the grading and wall plans for the area.

Slope failures have previously occurred on the fill slope southwest of the existing school as discussed in the "Background" section. Increased hydrostatic pressures from heavy precipitation or drainage changes and seismic loading could reactive the fill slope failure and force it further upslope. We understand that above-grade structures, additional fill, or sustained loads are not planned near the fill slope. If plans change to include any new structures, fill, or sustained loads, GeoDesign should be contacted to evaluate the plans and construction of a wall or a combination of slope stabilization and drainage will likely be required for the area.

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 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 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 it is lime amended due to the medium to high shrink-swell potential of the non-treated soil. 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.

The existing pavement sections at the site are not designed for the support of construction traffic. Accordingly, 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. The on-site fine-grained subgrades can also be protected from disturbance using granular haul roads and staging areas as described in this report.

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

6.0 DESIGN

6.1 PERMANENT SLOPES

Permanent cut slopes on the site should not exceed a gradient of 2.5H:1V and fill slopes on the site should not exceed a gradient of 2H:1V, unless specifically evaluated for stability. Slopes that will be maintained by mowing should not be constructed steeper than 3H:1V. 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.

6.2 DRAINAGE

6.2.1 General

Slow perched groundwater seepage was encountered in test pit TP-31 at 8.0 feet BGS. Groundwater was not observed in any of our other explorations at the site. Groundwater has been measured at 4.0 to 6.0 feet BGS with piezometers installed in boring SI-1 near the base of the cut slope at the east edge of the parking lot. Seepage has also been observed in past explorations at the site along the contact of fill and native soils. Drainage is very important for the site due to the tendency for water to perch on the high plasticity soils and along the interface of native and fill materials, the shrink-swell potential of the soils at the site, and the past slope failures for the fill slope at the south edge of 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.

6.2.2 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.

6.2.3 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 building be sloped away from the building to facilitate drainage away from the building.

6.2.4 Foundation Drains

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 below the ground surface 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.

6.2.5 French Drains

We recommend French drains be installed to intercept groundwater at the toe of any new cut slopes. 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. The drain rock and drainage geotextile should meet the requirements specified in the "Materials" section.

6.3 SEISMIC DESIGN CRITERIA

6.3.1 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 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 report.

6.3.2 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 soils considered to be susceptible to liquefaction or cyclic failure at the site.

6.4 FOUNDATION SUPPORT RECOMMENDATIONS

6.4.1 General

Based on the results of our explorations, analysis, and site research, we recommend supporting the proposed new school building on a mat foundation due to the shrink-swell potential of the soils at the site. Alternately, the building could be supported on spread footings, provided a minimum of 2 feet of soil below the lowest footing subgrade elevation is excavated and replaced with imported granular material or lime amended to reduce the swell potential. To support the building on shallow foundations, additional laboratory testing should be conducted on undisturbed soil samples prior to construction to evaluate if 2 feet of granular fill or soil treatment is sufficient or if greater depths are necessary to resist swelling potential at the site. Foundations should not be established on soft soil or soil containing deleterious material. If present, this material should be removed and replaced with structural fill. Foundations should not be established on the on-site high plasticity soil placed as structural fill unless it is lime amended to reduce the swell potential. In addition, native subgrades should be covered or backfilled to avoid excessive drying from exposure, which can increase the post-construction shrink-swell potential.

6.4.2 Mat Foundation

We recommend supporting the proposed new school building on a mat foundation bearing on firm, undisturbed native soil or structural fill consisting of imported granular material 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 soils. 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.

6.4.3 Spread Footings

Spread footings can be used to support the building, provided a minimum of 2 feet of soil below the lowest footing subgrade elevation in the building area is excavated and replaced with imported granular material or lime amended to reduce the swell potential. For this approach, additional laboratory testing should be conducted to evaluate the swell potential on additional relatively undisturbed soil samples prior to construction in order to determine if more than 2 feet of granular fill or soil treatment is recommended to resist swelling potential. Ancillary structures that can tolerate shrinkage and swell movements, such as some walls, can be supported on a minimum 6-inch-thick crushed rock leveling course bearing on firm, undisturbed native subgrade.

Footings established on firm, undisturbed native soil or structural fill over firm, undisturbed native soil 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.

6.4.4 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 the on-site soil or lime-amended on-site soil, a coefficient of friction equal to 0.30 may be used when calculating resistance to sliding. This value should be increased to 0.40 for footings bearing on imported granular material.

6.4.5 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.

6.5 FLOOR SLABS

Satisfactory subgrade support for building floor slabs supporting floor loads of up to 100 psf areal loading can be obtained provided the subgrade consists of imported granular fill or lime-amended on-site soil placed as structural fill down to the lowest foundation subgrade elevation. A minimum 6-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade to assist as a capillary break. The floor slab base rock should meet the requirements outlined in the "Materials" section. The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557. Floor slab base rock contaminated with excessive fines (greater than 5 percent by dry weight passing the U.S. Standard No. 200 sieve) should be replaced. A modulus of reaction of 200 pci can be used for slabs on grade constructed with the base rock section over a minimum of 2 feet of imported granular fill or lime-amended on-site soil

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.

6.6 RETAINING STRUCTURES

6.6.1 General

Retaining walls will be required as part of construction of the school facility. Based on the site grades and preliminary site plan, we anticipate walls will be less than 15 feet in height. Most of the walls are cut walls; however, some small fill walls less than 4 feet high may also be constructed.

6.6.2 Assumptions

Our retaining wall design recommendations are based on the following assumptions: (1) the walls consists 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.

6.6.3 Wall Design Parameters

Cantilever, gravity, or conventional retaining walls can be designed using the pressures in this section. For unrestrained retaining walls, we recommend an active pressure of 35 pcf equivalent fluid pressure should be used for design. Where retaining walls (such as basement stem walls) are restrained from rotation prior to being backfilled, a pressure of 55 pcf equivalent fluid pressure should be used for design. 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.

6.6.4 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.

6.6.5 Soldier Piles

Cantilever soldier piles may be used for planned cut walls. Cobbles and boulders were observed in test pits near the planned cut area and will result in difficult installation of the soldier piles if encountered. 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.

6.6.6 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.

6.6.7 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 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.

6.6.8 Wall Foundations

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

6.6.9 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, then 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.

6.6.10 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.

6.7 PAVEMENT RECOMMENDATIONS

6.7.1 General

Traffic at the proposed school facility 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 on-site soil will be used as structural fill, the upper 1 foot below pavements 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.

6.7.2 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

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 of this report.

7.0 CONSTRUCTION

7.1 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.

7.2 SITE PREPARATION

7.2.1 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.

7.2.2 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.

7.2.3 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. Areas that appear soft or loose should be improved in accordance with subsequent sections of this report.

7.2.4 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.

7.3 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.

7.4 EXCAVATION

7.4.1 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.

7.4.2 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.

7.4.3 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.

7.5 MATERIALS

7.5.1 Structural Fill

7.5.1.1 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.

7.5.1.2 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 soils exhibit 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.

7.5.1.3 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.

7.5.1.4 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.

7.5.1.5 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.

7.5.1.6 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.

7.5.1.7 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.

7.5.1.8 Geotextile Fabric

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.

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.

7.5.1.9 Soil Amendment

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 soils at the site, amendment with quicklime or hydrated lime will be most suitable for the site. After treatment with lime, soils 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.

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 80 psi. The amount of lime and 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 is required to stabilize soil. For preliminary design purposes, we recommend assuming 5 percent dry lime 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 should 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.

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.

8.0 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 asphalt placement and compaction.

9.0 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

REFERENCES

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FIGURES

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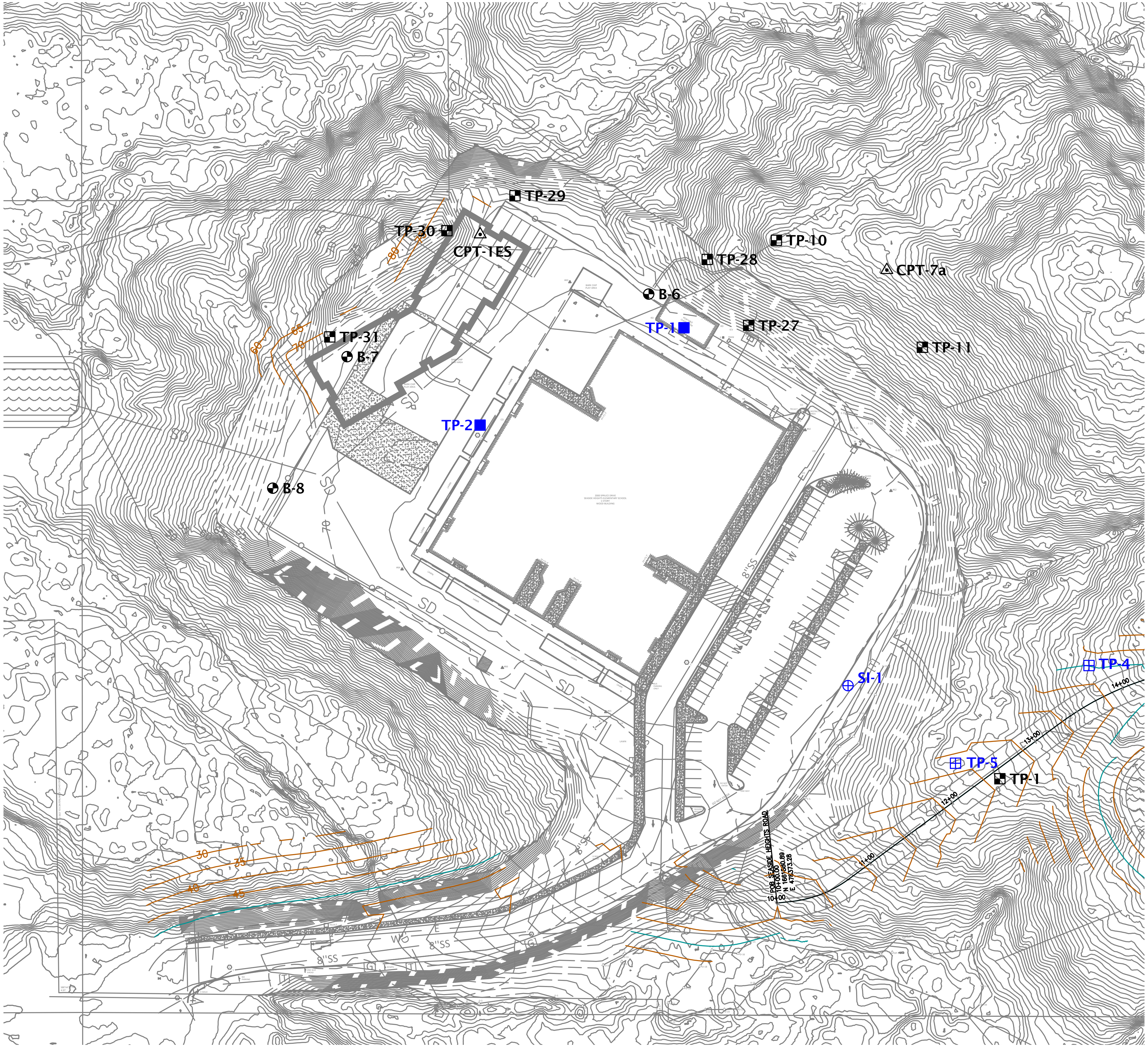
SEASIDESD-1-03-02	VICINITY MAP	
NOVEMBER 2017	SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION SEASIDE, OR	FIGURE 1

Printed By: mmiller | Print Date: 11/6/2017 8:26:31 AM
File Name: J:\S-Z\SeasideSD\SeasideSD-1\SeasideSD-1-03\SeasideSD-1-03-02\Figures\CAD\SeasideSD-1-03-02-SP-vm.dwg | Layout: FIGURE 1



LEGEND:

TP-1 ■	TEST PIT (KELLY/STRAZER, MARCH 1987)
B-1 ⊕	BORING (GEOCON, NOVEMBER 2012)
SI-1 ⊕	BORING (GEOCON, NOVEMBER 2012)
TP-4 ⊞	TEST PIT (GRI, MAY 2013)
B-1	BORING (GEODESIGN, SEPTEMBER 2017)
TP-1	TEST PIT (GEODESIGN, SEPTEMBER 2017)
CPT-1ES △	CONE PENTROMETER PROBE (SEPTEMBER 2017)

N

0 100 200

(SCALE IN FEET)

SITE PLAN BASED ON DRAWING PROVIDED BY
KPFF CONSULTING ENGINEERS

FIGURE 2

SITE PLAN

SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION
SEASIDE, OR

SEASIDESD-1-03-02

NOVEMBER 2017

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APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

Our subsurface exploration program included drilling three borings (B-6 through B-8) to depths between 16.5 and 46.5 feet BGS and excavating five test pits (TP-27 through TP-31) to depths between 5.0 and 11.0 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 13 and 14, 2017. The test pits were excavated using a John Deere 35C rubber-tracked excavator on September 20, 2017 by Dan J. Fischer Excavating, Inc. of Forest Grove, Oregon. The exploration logs are presented in this appendix. The explorations were observed by a member of our 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.

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 tests 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 TESTING

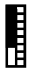
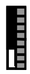
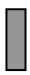
We performed one-dimensional consolidation testing in general accordance with ASTM D 2435 on a relatively undisturbed soil sample. The test measures the volume change of a soil sample under predetermined loads. a track-mounted drill rig and mud rotary drilling methods.

GRAIN-SIZE TESTING

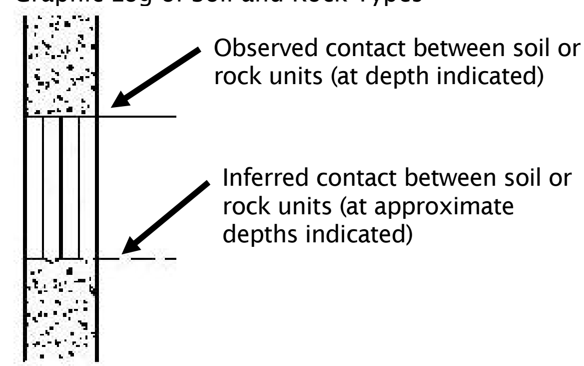
Grain-size testing was performed on select soil samples to determine the distribution of soil particle sizes. The testing consisted of a sieve analyses and percent fines determination (percent passing the U.S. Standard No. 200 sieve) completed in general accordance with ASTM C 136 and ASTM D 1140 (P200). The test results are presented in this appendix.

DIRECT SHEAR TESTING

Direct shear testing was performed on a select soil sample 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



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: Fine-Grained Soils	Silt and Clay In: Coarse-Grained Soils	Percent	Sand and Gravel In: Fine-Grained Soils	Sand and Gravel In: 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	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
0.0		Medium dense, gray GRAVEL (GP), trace silt; moist, coarse (topsoil to 7 inches, 2-inch-thick root zone) - **FILL**.					Heavy chatter from 1.0 to 2.5 feet.
2.5		Medium stiff, gray-brown CLAY (CH), minor gravel; moist to wet - **FILL**.	3.0			4	
5.0						5	
7.5		Medium stiff, brown with orange mottled CLAY (CH), minor gravel and sand; moist (colluvium). brown at 7.5 feet	7.0			5	
10.0						4	
12.5							
15.0		Stiff, dark gray CLAY (CH); moist, weakly indurated (decomposed claystone).	14.0			9	Chatter at 14.0 feet.
17.5		Exploration completed at a depth of 16.5 feet. Hammer efficiency factor is 88.3 percent.	16.5				Surface elevation was not measured at the time of exploration.
20.0							
22.5							
25.0							
27.5							
30.0							

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

GEODESIGN INC 9450 SW Commerce Circle - Suite 300 Wilsonville OR 97070 503.968.8787 www.geodesigninc.com	SEASIDESD-1-03-02	**BORING B-6**	
	NOVEMBER 2017	SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION SEASIDE, OR	**FIGURE A-1**

BORING LOG SEASIDESD-1-03-02-B6_8-TP27_31.GPJ GEODESIGN.GDT PRINT DATE: 11/4/17:RC:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	Soft to medium stiff, dark gray-brown CLAY (CH), minor silt and sand, trace gravel; moist, high plasticity, disturbed structure - **FILL**.				
2.5				4	
5.0			ATT	3	LL = 68% PL = 25%
7.5			DD		DD = 78 pcf
9.0	Medium dense, dark brown to black, clayey GRAVEL (GC); moist to wet, fine to coarse and subangular - **FILL**.	9.0		19	
12.0	Hard, dark gray CLAY (CH); moist, medium to high plasticity (weathered claystone).	12.0			Smoother, firm at 12.0 feet.
15.0				33	Expansion Index = 113 on composite sample from 15.0, 20.0, and 25.0 feet.
20.0				32	LL = 71% PL = 26%
25.0	very stiff at 25.0 feet			24	
30.0					

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

0 50 100

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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NOVEMBER 2017

BORING B-7

SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION
SEASIDE, OR

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
30.0		(continued from previous page)				24	
32.5							
35.0							
37.5							
40.0		hard at 40.0 feet				39	
		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.
42.5							
45.0							
47.5							
50.0							
52.5							
55.0							
57.5							
60.0							

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-1-03-02

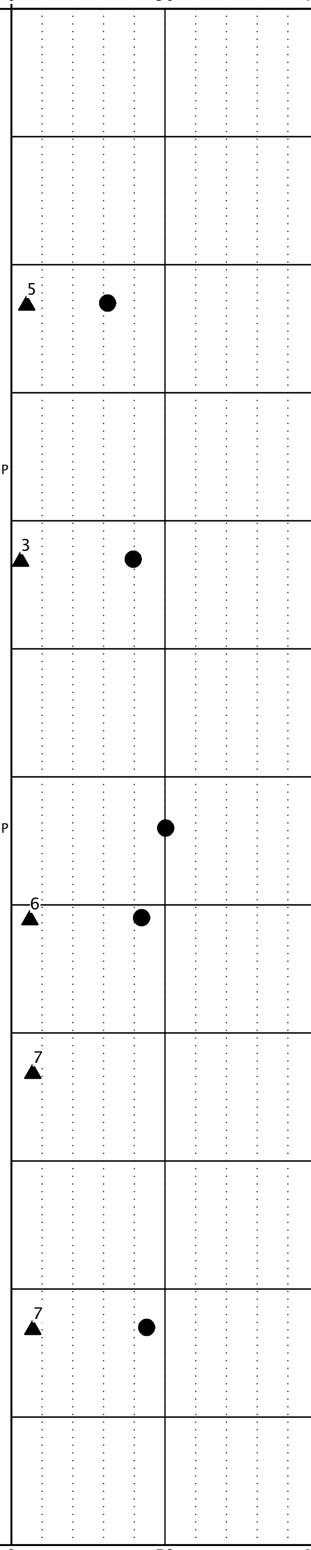
NOVEMBER 2017

BORING B-7
(continued)

SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION
SEASIDE, OR

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
0.0		Loose, dark gray GRAVEL with sand and silt (GP-GM); wet, fine to coarse and subangular (topsoil to 6 inches, 2-inch-thick root zone) - **FILL**.					Loose gravel caving in hole.
2.5							No sample at 2.5 feet.
5.0						5	Drove casing to 5.0 feet.
		Soft, dark gray-brown and yellow-brown CLAY with sand (CH), minor gravel; moist, disturbed structure - **FILL**.	6.5				Smoother at 6.5 feet.
7.5					P		No recovery.
10.0						3	
12.5							
15.0				DD DS	P		DD = 72 pcf
17.5		medium stiff, dark gray-brown to black at 17.0 feet				6	
20.0						7	
22.5							
25.0						7	
27.5							
30.0							



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-8

SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION
SEASIDE, OR

FIGURE A-3

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
30.0		stiff, gray-brown to dark brown, with wood and organics at 30.0 feet				9	
32.5		Medium stiff, light yellow-brown with orange mottled, sandy CLAY (CH); moist, medium to high plasticity, sand is fine to coarse and subangular (colluvium).	32.0				
35.0						6	
37.5							
40.0		stiff at 40.0 feet				13	
42.5		Very stiff, dark gray CLAY (CH); moist, medium to high plasticity, weakly indurated (weathered claystone).	43.0				
45.0						19	
47.5		Exploration completed at a depth of 46.5 feet. Hammer efficiency factor is 88.3 percent.	46.5				Surface elevation was not measured at the time of exploration.
50.0							
52.5							
55.0							
57.5							
60.0							

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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NOVEMBER 2017

BORING B-8
(continued)

SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION
SEASIDE, OR

FIGURE A-3

BORING LOG SEASIDESD-1-03-02-B6_8-TP27_31.GPJ GEODESIGN.GDT PRINT DATE: 11/4/17:RC:KT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Stiff, light brown SILT (MH), some clay, trace gravel; moist (decomposed siltstone) (~6-inch-thick root zone).					
2.5							
		Very stiff to hard, gray CLAY (CH), trace sand; moist, sand is fine, cemented (weathered claystone).	4.0	PP	⊠	●	PP = >3.0 tsf
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							

EXCAVATED BY: Dan J. Fischer Excavating, Inc.   LOGGED BY: CR   COMPLETED: 09/20/17

EXCAVATION METHOD: trackhoe (see document text)

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	NOVEMBER 2017	SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION SEASIDE, OR	**FIGURE A-4**

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Stiff, dark brown SILT (ML), some organics, trace gravel and sand; moist (topsoil, 18-inch-thick root zone).		PP			PP = 1.0 tsf
2.5		Very stiff, brown with orange mottled SILT (MH), some clay, trace sand; moist, sand is fine, cemented (weathered siltstone).	2.0	PP	☒	●	PP = >3.0 tsf
5.0							
7.5		Very stiff to hard, gray CLAY (CH), trace sand; moist, sand is fine (weathered claystone).	9.0	PP	☒	●	PP = >3.0 tsf
10.0		Exploration completed at a depth of 10.0 feet.	10.0				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: CR COMPLETED: 09/20/17

EXCAVATION METHOD: trackhoe (see document text)

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TEST PIT TP-28

SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION
SEASIDE, OR

FIGURE A-5

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	MOISTURE CONTENT % (0 / 50 / 100)	COMMENTS
0.0		Medium stiff, brown SILT (MH), some clay, minor gravel, trace sand and organics; moist, gravel is subrounded (~4- to 5-inch-thick root zone) - **FILL**.					
2.5				PP	X	●	PP = 0.75 tsf
		trace debris (plastic) at 4.0 feet					
		without debris at 4.5 feet					
5.0				PP	X	●	PP = 0.5 tsf
		without gravel at 6.0 feet					
7.5							
		Medium stiff, brown with orange mottled CLAY (CH), some silt; moist (decomposed claystone).	9.0				
10.0				PP	X	●	PP = 0.75 tsf
		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							

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: CR COMPLETED: 09/20/17

EXCAVATION METHOD: trackhoe (see document text)

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NOVEMBER 2017

TEST PIT TP-29

SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION
SEASIDE, OR

FIGURE A-6

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	MOISTURE CONTENT % (0–50–100)	COMMENTS
0.0	Medium stiff, brown SILT (ML), minor gravel; moist (topsoil, 8-inch-thick root zone).				
	Stiff to very stiff, brown with orange mottled SILT (MH), some clay, minor gravel; moist (decomposed siltstone).	0.9	DD CON	●	DD = 58 pcf
2.5			ATT PP	●	PP = 1.5 tsf LL = 96% PL = 46%
	Very dense, dark brown-orange GRAVEL (GP); moist (claystone).	4.5		●	
5.0	Exploration terminated at a depth of 5.0 feet due to refusal on bedrock.	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.
7.5					
10.0					
12.5					
15.0					

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: CR COMPLETED: 09/20/17

EXCAVATION METHOD: trackhoe (see document text)

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SEASIDESD-1-03-02

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TEST PIT TP-30

SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION
SEASIDE, OR

FIGURE A-7

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Loose, gray-brown, silty GRAVEL (GM); moist, subangular - **FILL**.					
		Stiff to very stiff, brown with orange and gray mottled SILT with gravel (MH), some clay, trace sand; moist (decomposed siltstone).	0.5				
				PP			PP = 2.0 tsf
2.5							
				PP			Shelby tube pushed to practical refusal. Expansion Index = 83 at 3.0 feet. PP = >3.0 tsf
		sandy, trace gravel at 4.0 feet					Hard excavating at ~4.0 feet.
5.0				PP SIEV		●	PP = >3.0 tsf
7.5							Slow groundwater seepage observed at 8.0 feet.
		Stiff, dark gray CLAY (CH), minor gravel; moist (decomposed claystone).	9.0	PP		●	PP = 1.0 tsf
10.0		Exploration completed at a depth of 10.0 feet.	10.0				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: CR COMPLETED: 09/20/17

EXCAVATION METHOD: trackhoe (see document text)

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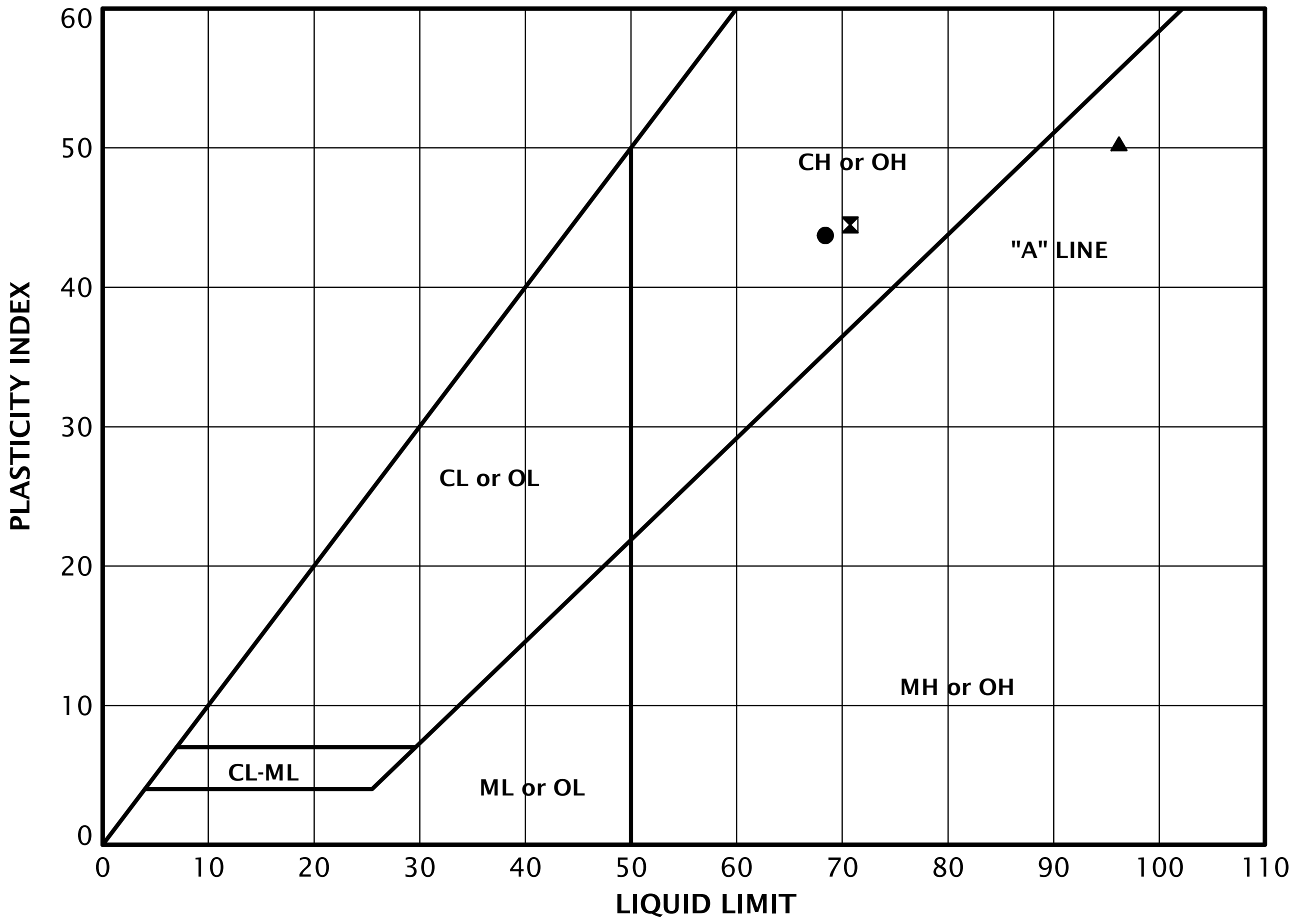
SEASIDESD-1-03-02

NOVEMBER 2017

TEST PIT TP-31

SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION
SEASIDE, OR

FIGURE A-8



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-7	5.0	44	68	25	43
⧗	B-7	20.0		71	26	45
▲	TP-30	3.0	49	96	46	50

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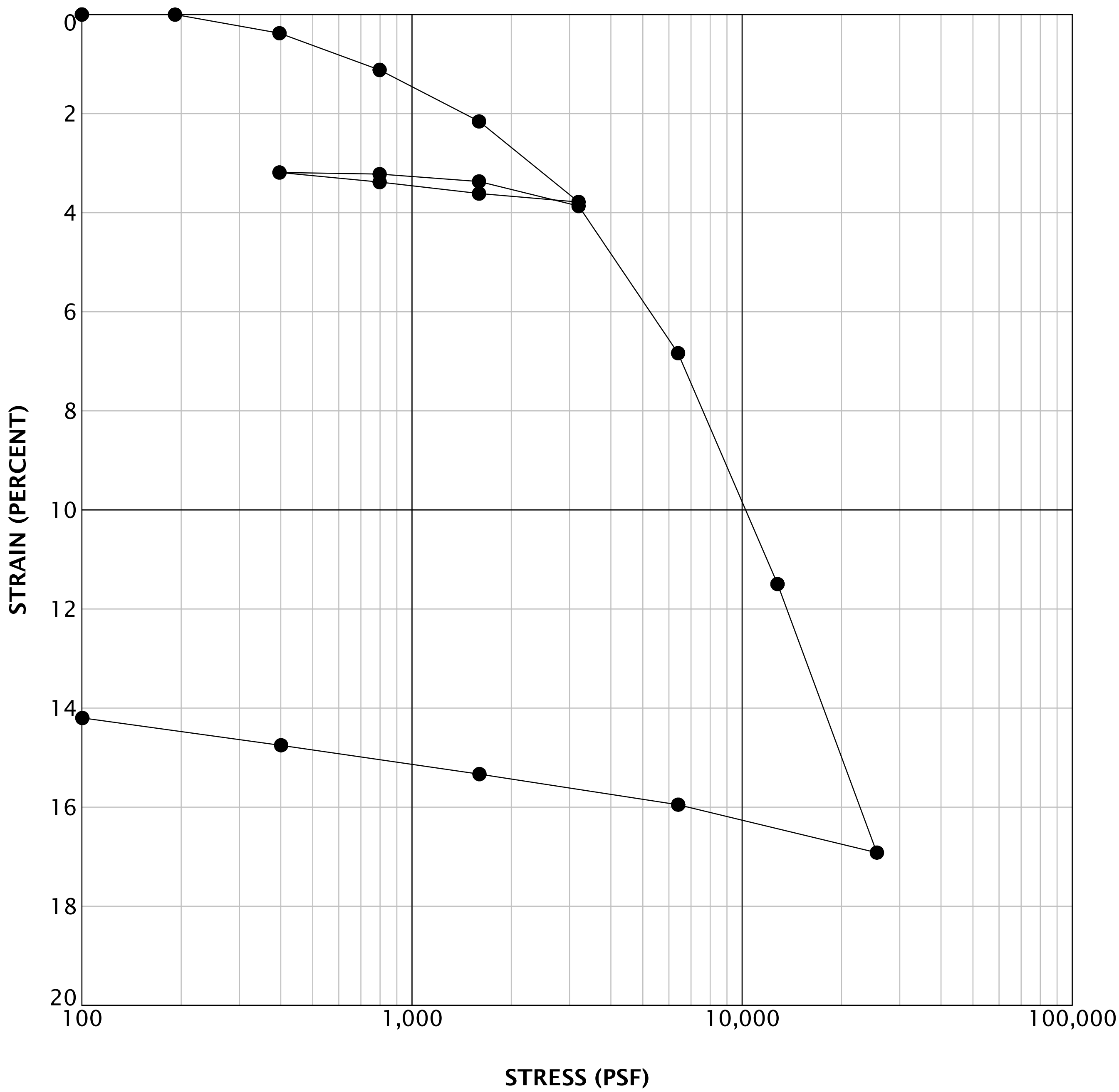
SEASIDESD-1-03-02

NOVEMBER 2017

ATTERBERG LIMITS TEST RESULTS

SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION
SEASIDE, OR

FIGURE A-9



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)
●	TP-30	1.0	55	58

CONSOL_STRAIN_100K SEASIDESD-1-03-02-B6_8-TP27_31.GPJ GEODESIGN.GDT PRINT DATE: 11/4/17:KT

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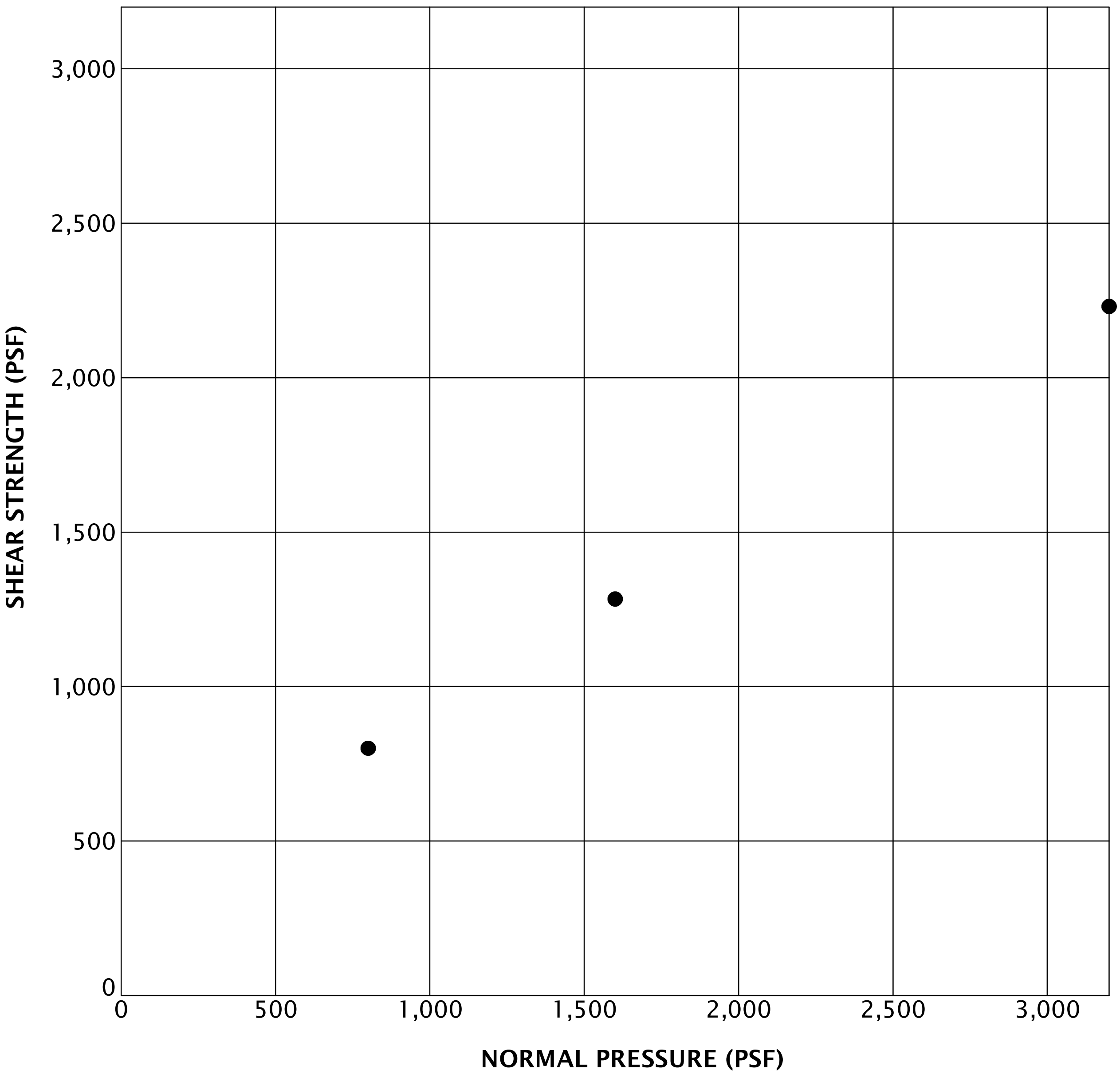
CONSOLIDATION TEST RESULTS

SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION
SEASIDE, OR

FIGURE A-10



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SOAKED
●	B-8	15.0	42	79	YES

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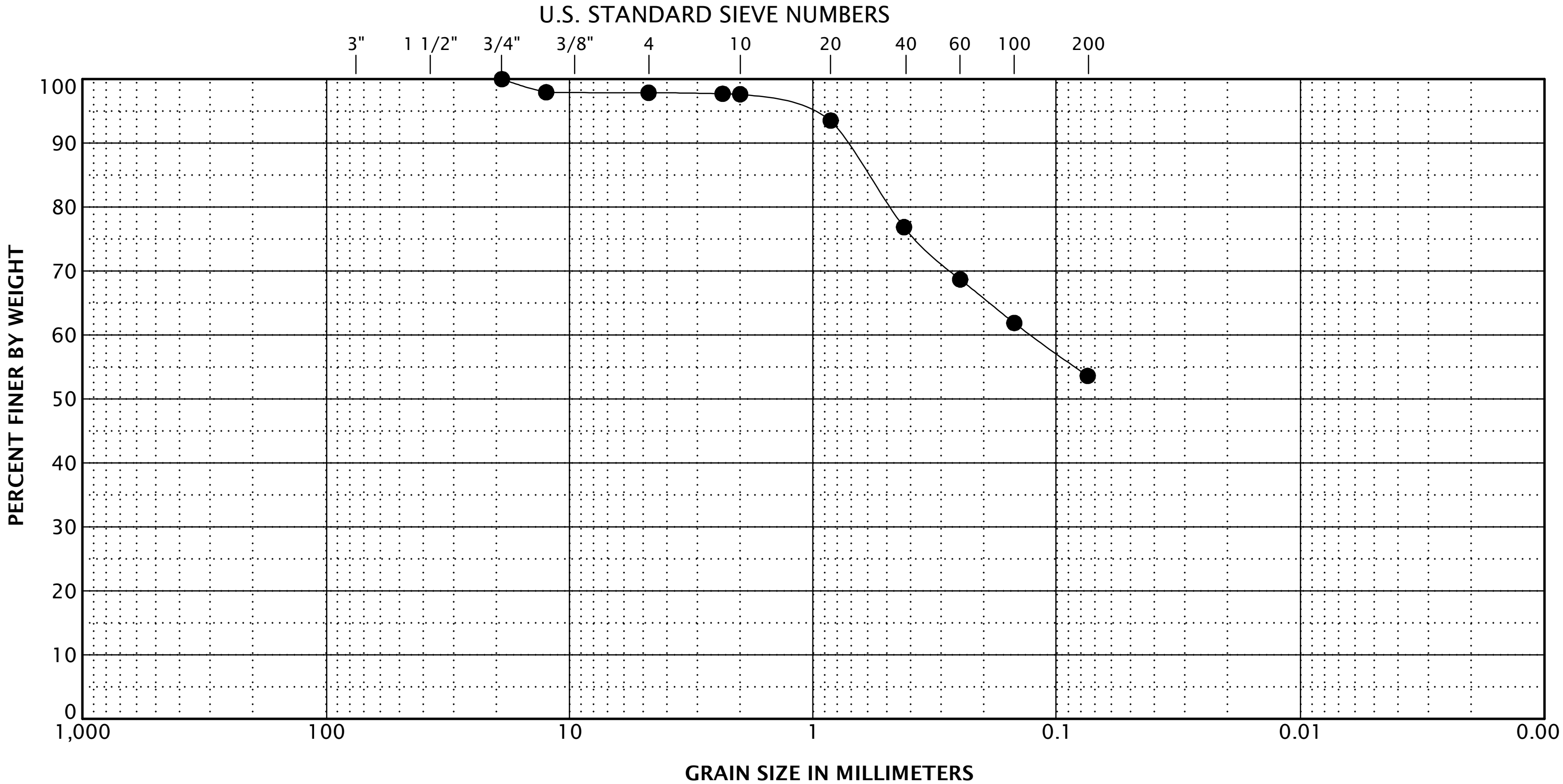
SEASIDESD-1-03-02

NOVEMBER 2017

DIRECT SHEAR TEST RESULTS

SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION
SEASIDE, OR

FIGURE A-11



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-31	5.0	39	0.13					2	44	54	

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GRAIN-SIZE TEST RESULTS

SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION
SEASIDE, OR

FIGURE A-12

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-6	5.0		53							
B-6	10.0		49							
B-6	15.0		34							
B-7	2.5		45							
B-7	5.0		44					68	25	43
B-7	8.0		35	78						
B-7	9.0		42							
B-7	15.0		28							
B-7	20.0							71	26	45
B-7	25.0		31							
B-7	40.0		29							
B-8	5.0		31							
B-8	10.0		40							
B-8	15.0		50	72						
B-8	17.0		42							
B-8	25.0		44							
B-8	30.0		51							
B-8	35.0		61							
B-8	40.0		35							
B-8	45.0		29							
TP-27	4.0		33							
TP-27	8.0		28							
TP-28	5.0		35							
TP-28	9.0		32							
TP-29	2.0		62							
TP-29	5.0		101							
TP-29	9.5		68							

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-30	1.0		55	58						
TP-30	3.0		49					96	46	50
TP-30	4.8		23							
TP-31	5.0		39		2	44	54			
TP-31	9.0		40							

LAB SUMMARY SEASIDESD-1-03-02-B6_8-TP27_31.GPJ GEODESIGN.GDT PRINT DATE: 11/4/17:KT

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SEASIDESD-1-03-02

NOVEMBER 2017

SUMMARY OF LABORATORY DATA
(continued)

SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION
SEASIDE, OR

FIGURE A-13

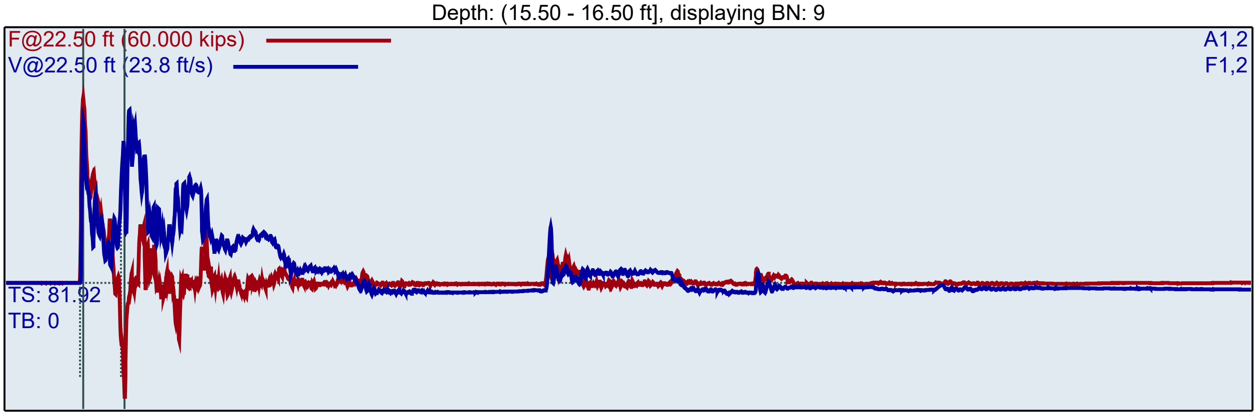
WSSC-8-01
JDT

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

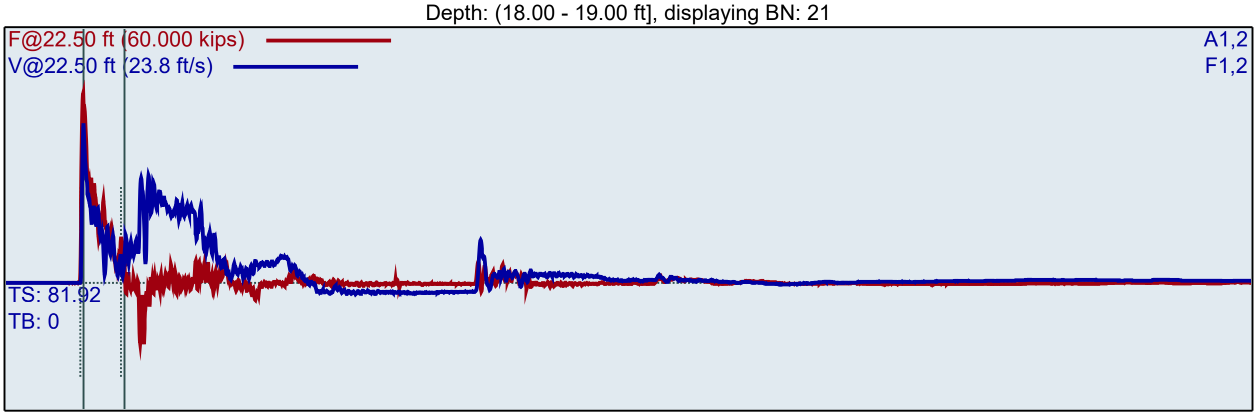
BPM: Blows/Minute
EMX: Maximum Energy
ETR: Energy Transfer Ratio - Rated

BL#	BC	LP	FMX	VMX	BPM	EMX	ETR
	/6"	ft	kips	ft/s	bpm	ft-lb	(%)
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

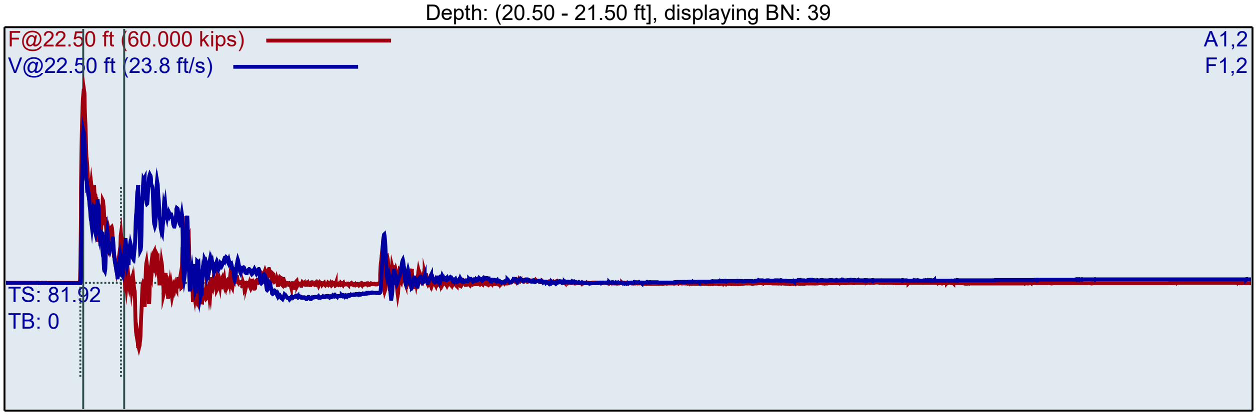


BL#	BC /6"	LP ft	FMX kips	VMX ft/s	BPM bpm	EMX ft-lb	ETR (%)
12	4	18.13	43.583	14.3	54.5	310.6	88.8
13	4	18.25	43.570	14.1	54.5	312.4	89.3
14	4	18.38	43.975	14.3	54.2	310.8	88.8
15	4	18.50	42.792	15.5	54.6	312.5	89.3
16	8	18.56	43.422	15.0	54.5	317.1	90.6
17	8	18.63	43.768	14.6	54.1	316.0	90.3
18	8	18.69	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

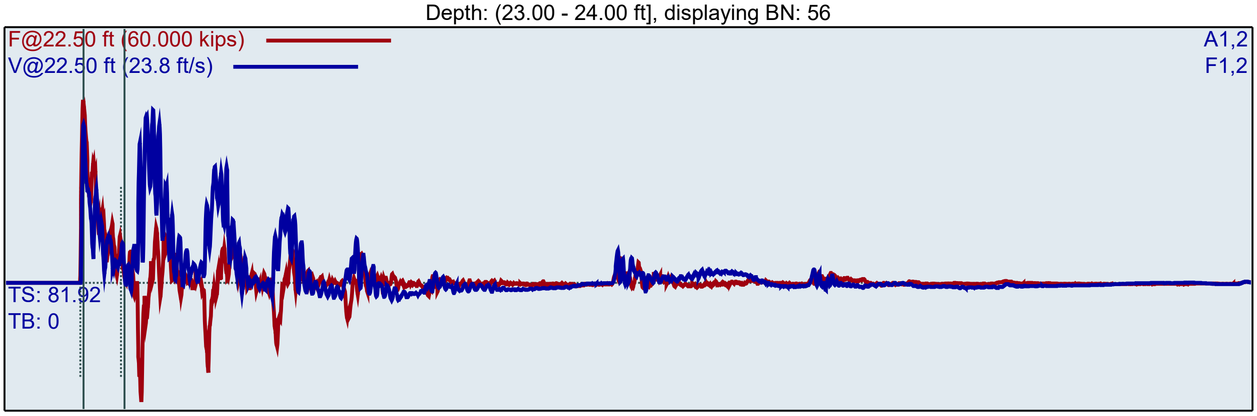


BL#	BC /6"	LP ft	FMX kips	VMX ft/s	BPM bpm	EMX ft-lb	ETR (%)
24	8	20.56	44.057	14.8	54.5	317.7	90.8
25	8	20.63	43.219	16.8	54.2	306.1	87.5
26	8	20.69	44.090	14.1	54.2	313.8	89.7
27	8	20.75	44.898	14.9	54.4	316.2	90.3
28	8	20.81	44.836	16.6	54.2	311.3	88.9
29	8	20.88	44.302	16.5	54.4	314.1	89.8
30	8	20.94	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	LP	FMX	VMX	BPM	EMX	ETR
	/6"	ft	kips	ft/s	bpm	ft-lb	(%)
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

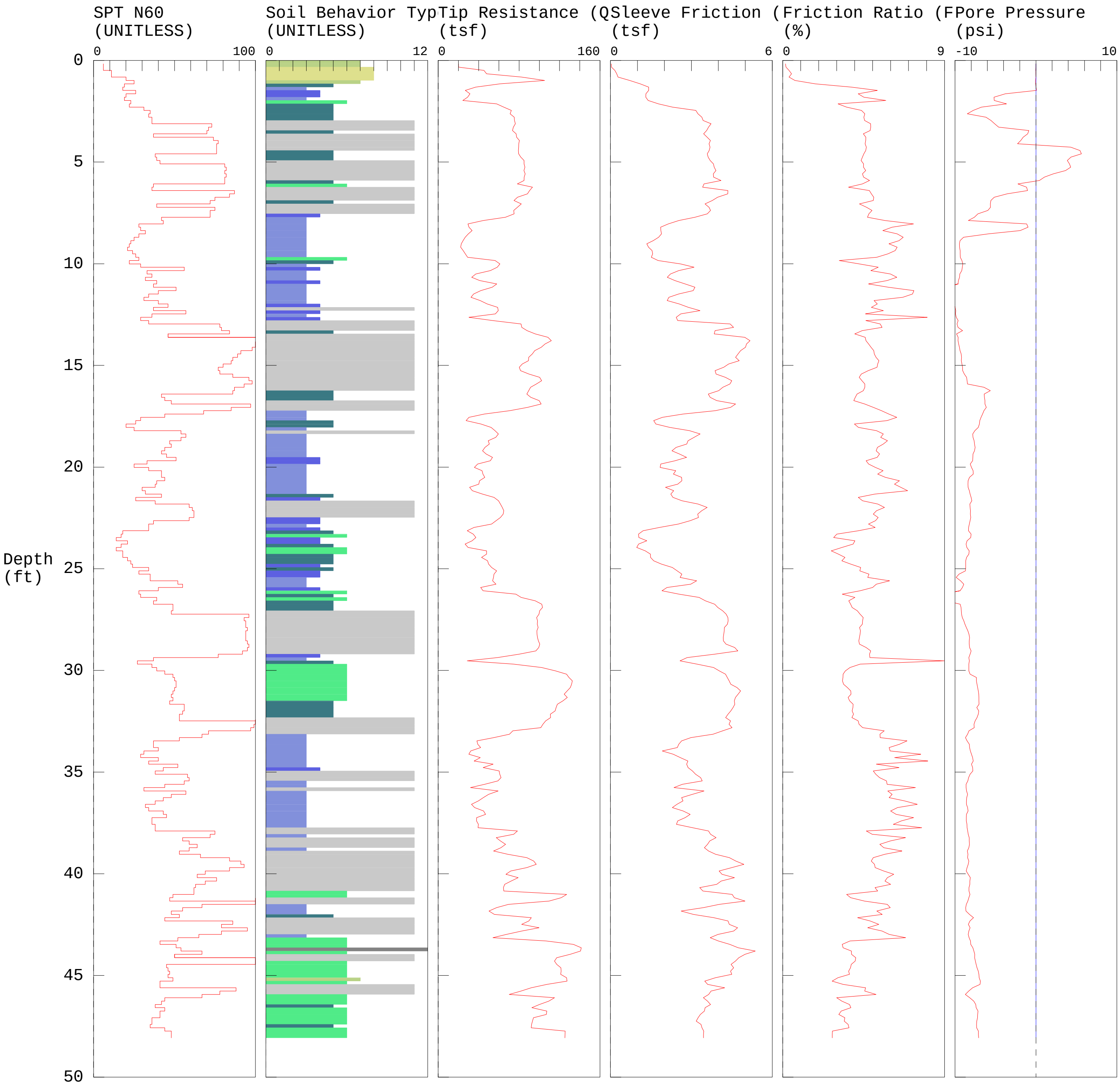
CONE PENETROMETER TESTING

One CPT probe (CPT-1ES) was advanced to a depth of depth of 48.1 feet BGS. Figure 2 shows the location of the CPT probe relative to existing site features. The CPT was performed in general accordance with ASTM D 5778 by Oregon Geotechnical Explorations, Inc. of Keizer, Oregon, on September 7, 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 the CPT. This appendix presents the results of the CPT completed for this project.

GeoDesign / CPT-1ES / 2000 Spruce Loop Seaside

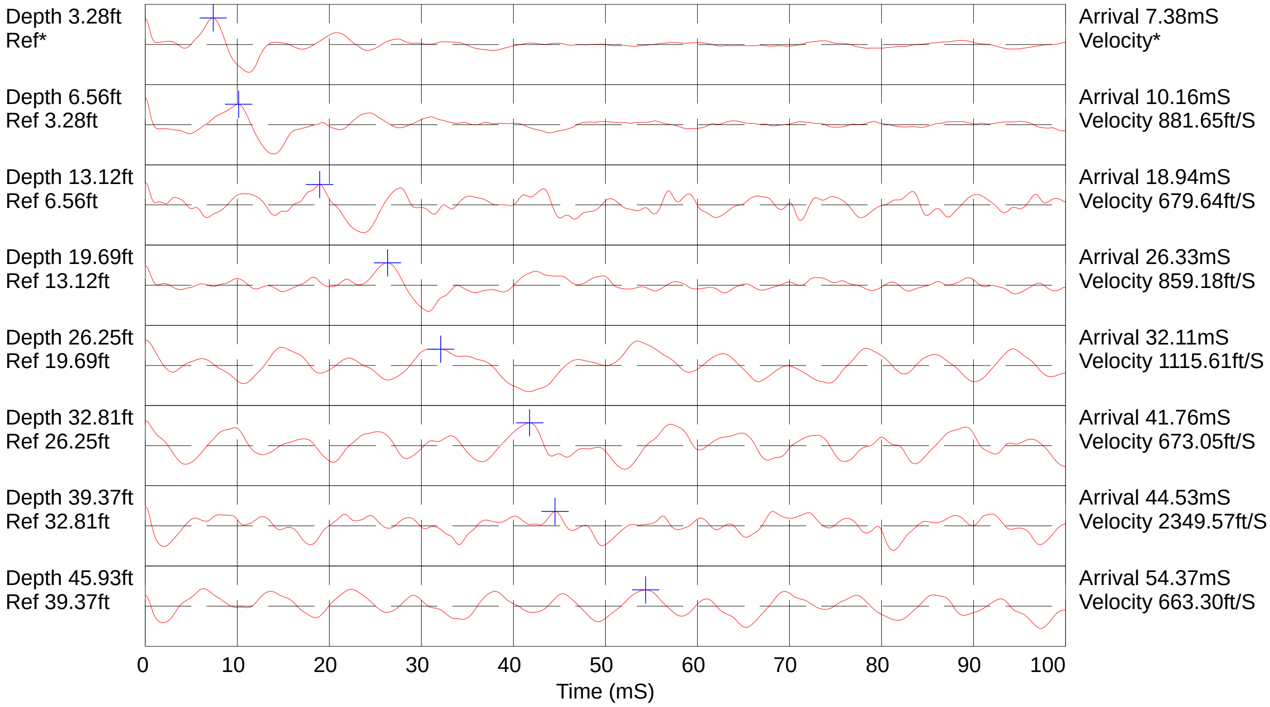
OPERATOR: OGE BB
CONE ID: DPG1211
HOLE NUMBER: CPT-1
TEST DATE: 9/7/2017 4:05:03 PM
TOTAL DEPTH: 48.064 ft



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

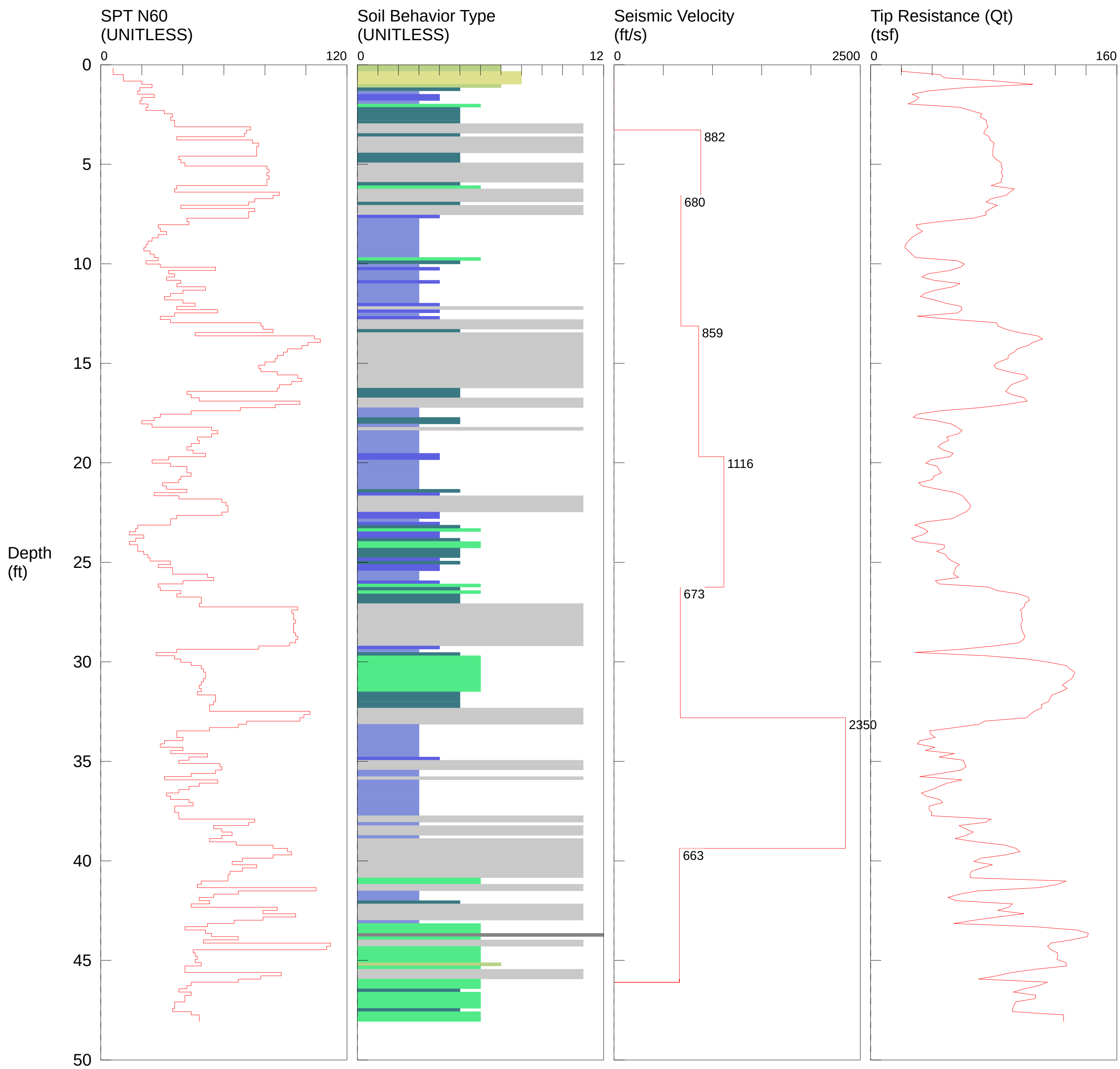
COMMENT: 17142 / GeoDesign / CPY-1 / 2000 Spruce Loop Seaside



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

GeoDesign / CPT-1 / 2000 Spruce Loop Seaside

OPERATOR: OGE BB
CONE ID: DPG1211
HOLE NUMBER: CPT-1
TEST DATE: 9/7/2017 4:05:03 PM
TOTAL DEPTH: 48.064 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

APPENDIX C

APPENDIX C

EXPANSION INDEX TESTING

Expansion index testing was conducted on two soil samples from the Seaside Elementary School site and one from the middle school/high 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.

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

APPENDIX D

APPENDIX D

GEODESIGN NEARBY EXPLORATIONS

GeoDesign completed nearby explorations to the elementary school that consisted of excavating three test pits (TP-1, TP-10, and TP-11) to depths between 11.0 and 12.0 feet BGS and advancing one CPT probe (CPT-7a) to 57.1 feet BGS. The test pits were excavated on September 6 and 7, 2017 using a John Deere 35C rubber-tracked excavator by Dan J. Fischer Excavating, Inc. of Forest Grove, Oregon. The CPT was performed in general accordance with ASTM D 5778 by Oregon Geotechnical Explorations, Inc. of Keizer, Oregon, on September 15, 2017. The associated exploration logs are presented in this appendix.

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. This information should be considered accurate only to the degree implied by the methods used.

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	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 (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					
7.5		Very stiff, light brown with orange streaked CLAY (CL/CH), trace sand; moist (decomposed to weathered claystone).	7.0		⊠	●	
10.0		dark gray with orange streaks at 10.0 feet					
		very stiff to hard at 11.0 feet					
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							

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

EXCAVATION METHOD: trackhoe (see document text)

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Wilsonville OR 97070
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SEASIDESD-1-03-01

NOVEMBER 2017

TEST PIT TP-1

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-6

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Stiff, gray with orange mottled SILT (MH), some clay, trace sand and gravel; moist (topsoil to 4 inches, 4-inch-thick root zone).					
2.5					☒	●	
5.0		with cobbles at 5.0 feet					
		dark gray at 6.0 feet			☒		
7.5							
		Stiff to very stiff, gray CLAY (CH), trace sand; moist.	9.0				
10.0					☒	●	
		with cobbles 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							

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: RSK COMPLETED: 09/07/17

EXCAVATION METHOD: trackhoe (see document text)

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SEASIDESD-1-03-01

NOVEMBER 2017

TEST PIT TP-10

SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-15

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		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					
2.5							
		dark gray, boulder at 3.0 feet			☒	●	
		boulder at 4.0 feet					
5.0		very stiff at 5.0 feet					
		boulder at 7.0 feet					
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							

EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: RSK

COMPLETED: 09/07/17

EXCAVATION METHOD: trackhoe (see document text)

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SEASIDESD-1-03-01

NOVEMBER 2017

TEST PIT TP-11

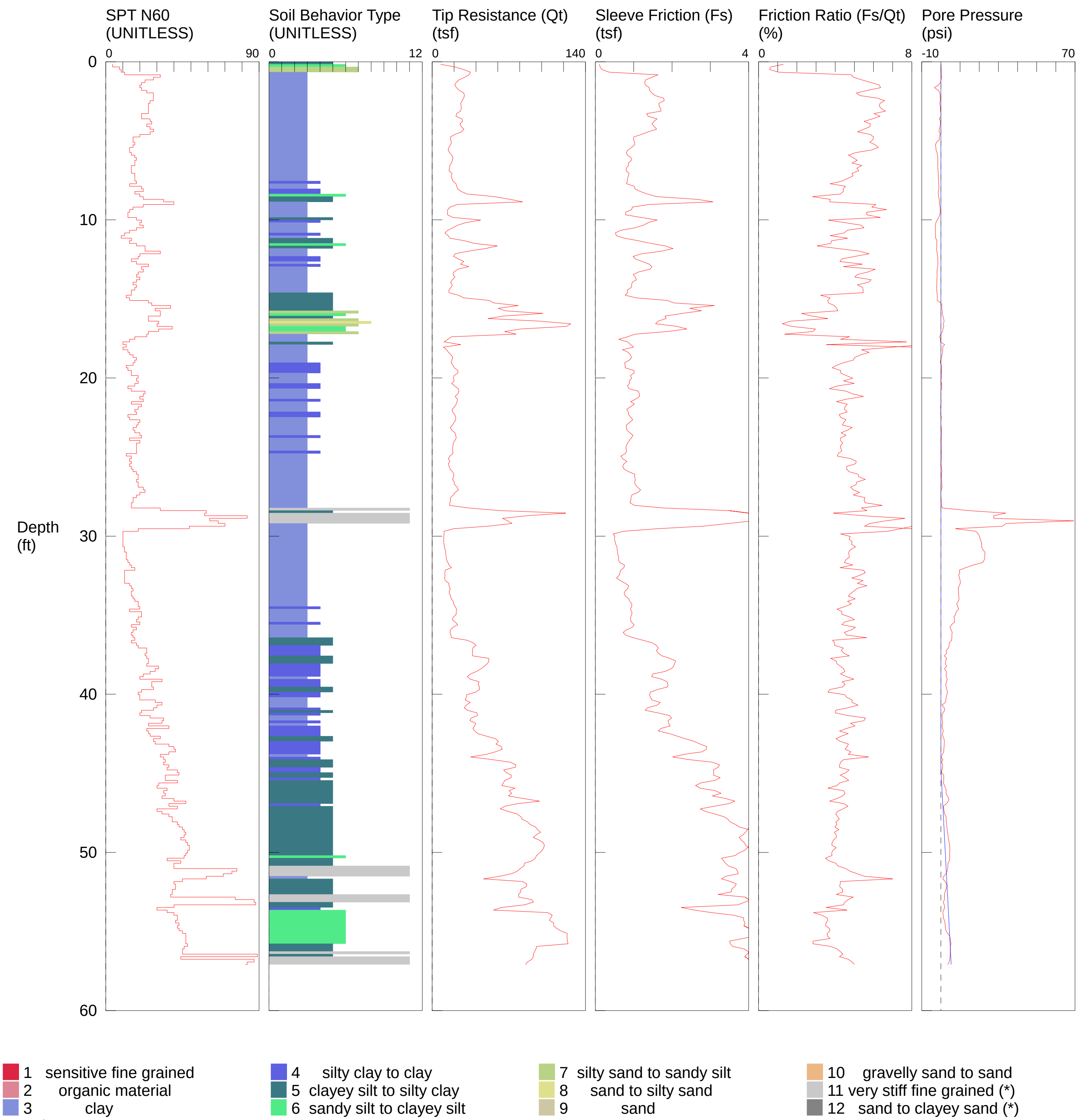
SEASIDE SCHOOL DISTRICT CAMPUS
SEASIDE, OR

FIGURE A-16

TEST PIT LOG - 1 PER PAGE SEASIDESD-1-03-01-B1_5-TP1_22,26.GPJ GEODESIGN.GDT PRINT DATE: 11/4/17:RC:KT

GeoDesign / CPT-7a / Seaside Heights School

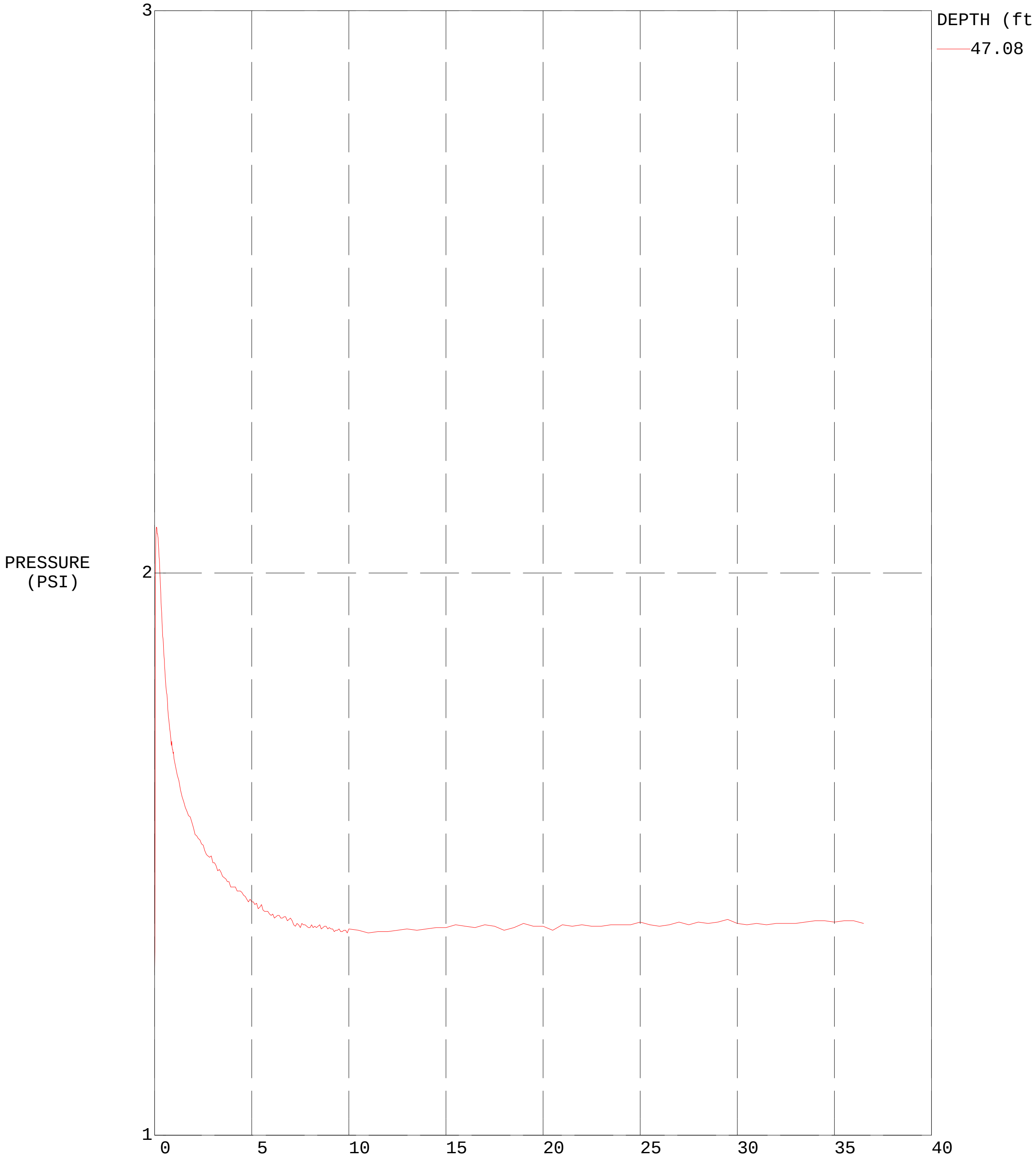
OPERATOR: OGE BB
CONE ID: DDG1415
HOLE NUMBER: CPT-7a
TEST DATE: 9/15/2017 9:16:05 AM
TOTAL DEPTH: 57.087 ft



*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 E

APPENDIX E

PAST EXPLORATIONS BY OTHERS

Past explorations completed near the elementary school include a boring (SI-1) completed by GeoCon in November 2012 and two test pits (TP-4 and TP-5) completed by GRI in May 2013. The approximate locations of the explorations are shown on Figure 2 and the associated exploration logs are presented in this appendix.

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				-Becomes soft with 2'x1" layers of wood/roots	1		125.3
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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING SI 1 ELEV. (MSL.)_______ DATE COMPLETED 11-06-2012 EQUIPMENT CME 75 TRACK w/MUD BY: S. DIXON 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	... 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

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

P1910-05-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

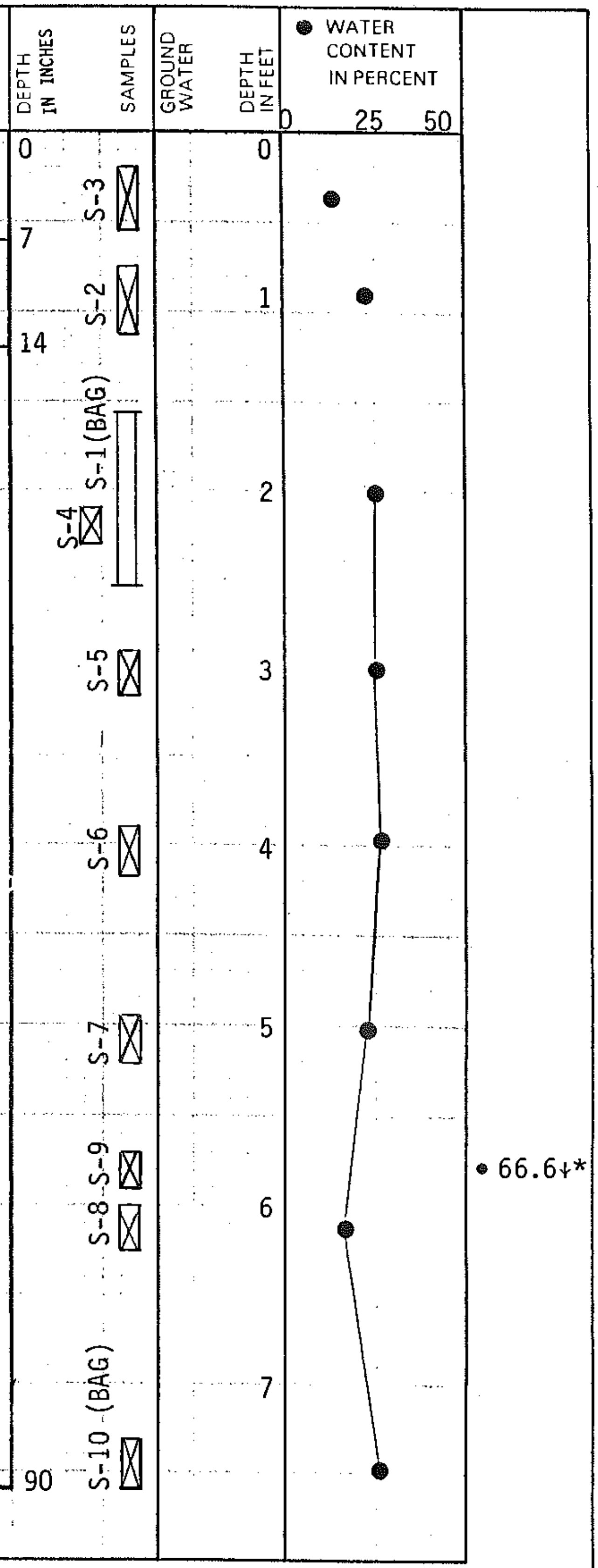
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

Approx. Surface Elev. = +67.0 ft. ¶*

DEPTH IN INCHES

Depth (in.)	Description
0	2" ASPHALT CONCRETE SLAB
	SUBGRADE: PIT ROCK (MAN-MADE FILL)
12	Medium dense, fine to medium SAND (MAN-MADE FILL)
24	Soft**, gray, fractured, CLAYSTONE; two primary fracture planes oriented A) 35° to 40° (from horizontal) facing south and B) near vertical (i.e. 80°) relatively closed fractures, rock separates into fragments less than 1 foot size
36	NOTES: Refusal for 4" tube driven by backhoe bucket at 20 blows/6"
	¶* Original Ground Surface approximately 25 feet higher.
48	** see report text for discussion of soft rock versus hard soil.
60	
72	Slight seepage and free water observed on bedding planes and possible fracture zone of ½" crumbled disturbed hard clay
	↓* Nodule of white clay (fossil)
84	
	Bottom of Test Pit, Completed 3/24/87



GRAB SAMPLE LOCATION AND NUMBER
SHELBY TUBE SAMPLE LOCATION AND NUMBER
1" OPEN BARREL SAMPLE & NUMBER

IMPERVIOUS SEAL
WATER LEVEL FOR DATE SHOWN
OBSERVATION WELL TIP
2 WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR

Kelly/Strazer Associates Geotechnical Consultants	SEASIDE ELEMENTARY SCHOOL Seaside, Oregon	LOG OF TEST PIT TP—1 April, 1987 0-298.02	FIGURE 12

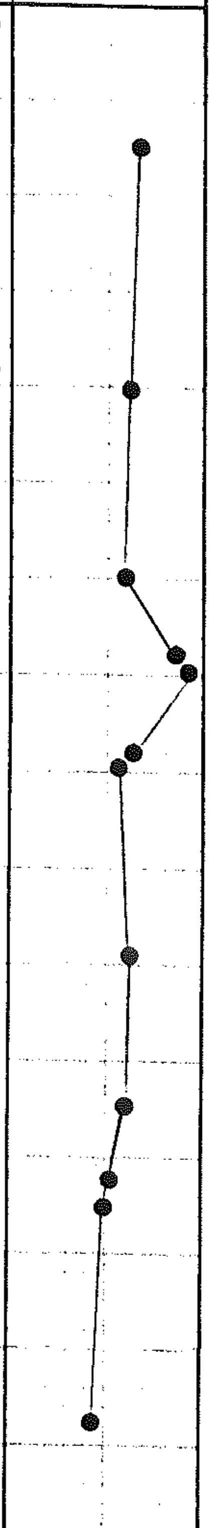
Approx. Surface Elev. = +67.0 ft. ¶*

DEPTH IN INCHES	DESCRIPTION	DEPTH IN INCHES	SAMPLES	GROUND WATER	DEPTH IN FEET
0	8" ORGANIC PEAT (MAN-MADE FILL)	0			0
	Slight seepage @ FILL-CLAY Boundary	8			
12	Hard, moist, brown CLAY		S-2		1
	½" seam of firm, light gray/white highly plastic CLAY		S-4		
24	Very hard, brown damp to moist CLAY with fracture planes	24	S-3		2
36	Very stiff to hard, brown, moist CLAY without bedding planes; CLAY crumbly and has disturbed appearance as possible fracture zone	36	S-4, S-12 (BAG)		3
48	NOTE: Samples S-8, S-11 & S-12 taken from MAN-MADE FILL east wall of Test Pit		S-8 (BAG), S-5		4
	Decomposed organic tree fragment		S-6(BAG)		
60	(Note: Fill above bedding is COMPACTED CLAY)	60			5
	Concrete Drain Pipe		S-11 (30%)		
	[Seepage from Sand Bedding below drain pipe]				
72	Soft**, gray fractured, CLAYSTONE; two primary fracture planes oriented A) 35° to 40° (from horizontal) facing south and B) near vertical (i.e. 80°) relatively closed fractures, rock separates into fragments less than 1 foot size, sidewalls of test pit sloughing, open thin fractures are wet		S-7, S-10		6
84	** see report text for discussion of soft rock versus hard soil.				7
	¶* Original Ground Surface approximately 20 feet higher.	90	S-9		
	Bottom of Test Pit, Completed 3/24/87				8

DEPTH IN INCHES

● WATER CONTENT IN PERCENT

0 25 50



☒ S- GRAB SAMPLE LOCATION AND NUMBER

▯ S- SHELBY TUBE SAMPLE LOCATION AND NUMBER

I S- 1" OPEN BARREL SAMPLE & NUMBER

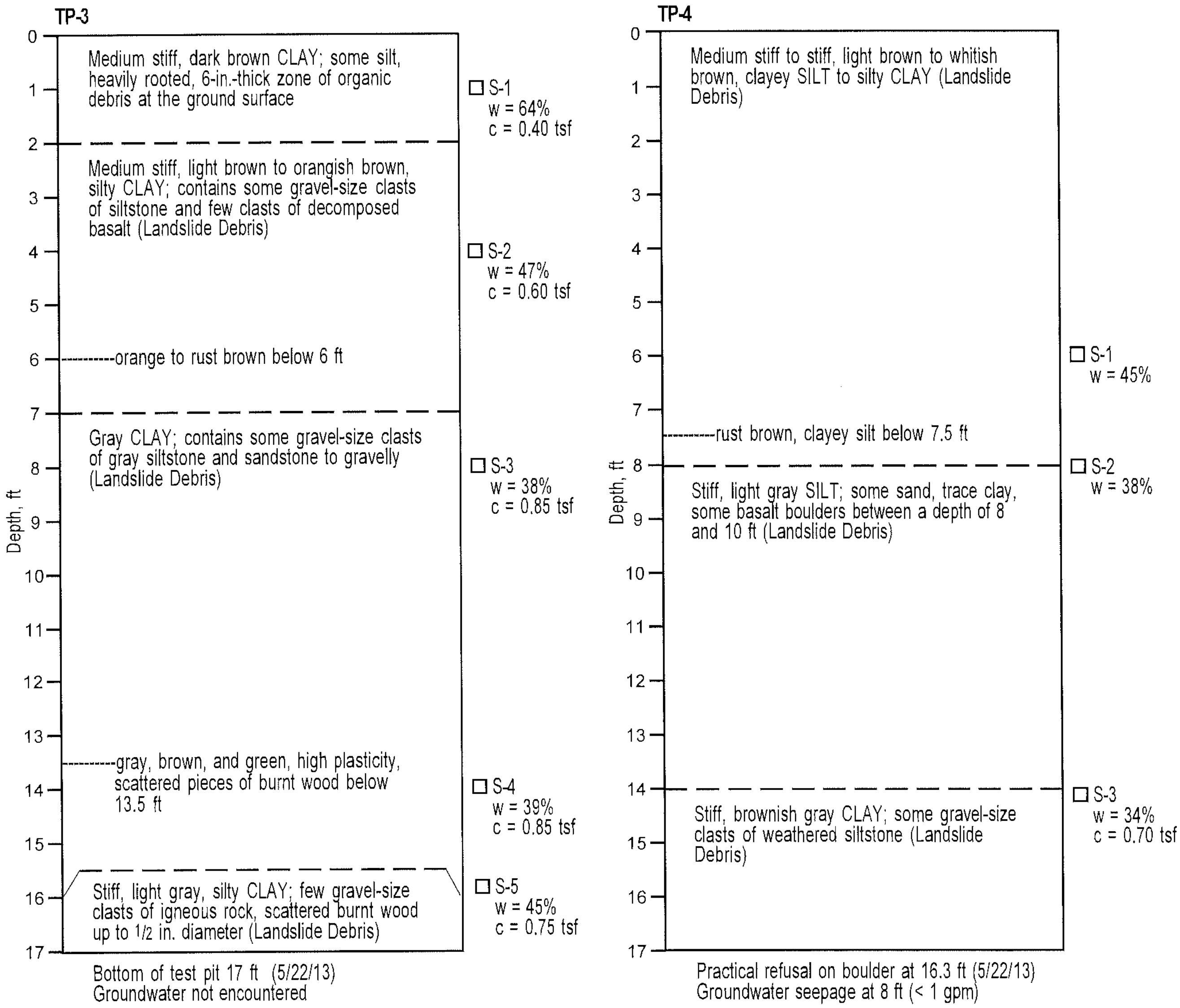
IMPERVIOUS SEAL

▽ WATER LEVEL FOR DATE SHOWN

OBSERVATION WELL TIP

2 WATER LEVEL IS FOR DATE SHOWN AND MAY VARY WITH TIME OF YEAR.

Kelly/Strazer Associates, Geotechnical Consultants	SEASIDE ELEMENTARY SCHOOL Seaside, Oregon	LOG OF TEST PIT TP—2 April, 1987 0-298.02	FIGURE 13



LEGEND

☐ = GRAB SAMPLE

w = NATURAL MOISTURE CONTENT

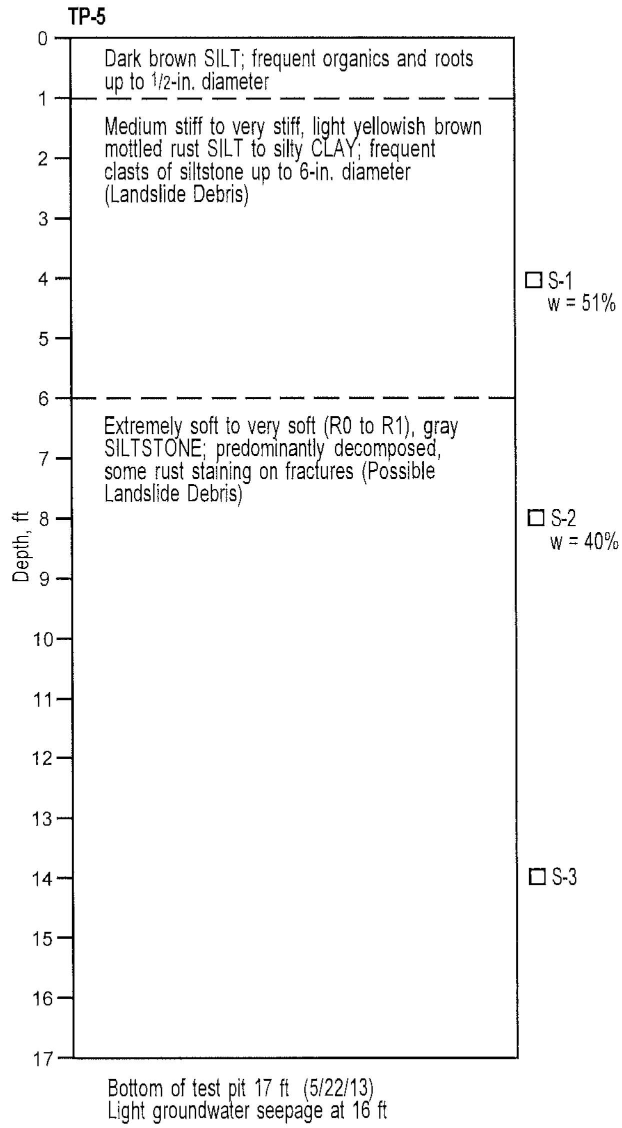
c = TORVANE SHEAR STRENGTH

GROUND SURFACE ELEVATIONS NOT AVAILABLE

GRI

TEST PIT LOGS

JUNE 2013 JOB NO. 5422 FIG. 3A



LEGEND

☐ = GRAB SAMPLE

w = NATURAL MOISTURE CONTENT

c = TORVANE SHEAR STRENGTH

GROUND SURFACE ELEVATIONS NOT AVAILABLE

GRI

TEST PIT LOGS

APPENDIX F

APPENDIX F

SITE-SPECIFIC SEISMIC HAZARD EVALUATION

INTRODUCTION

The information in this appendix summarizes the results of a site-specific seismic hazard evaluation for the proposed Seaside Elementary 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 F-1 shows the locations of faults with potential Quaternary movement within a 20-mile radius of the site. Figure F-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 F-1 presents the closest mapped distance and mapped length of these faults.

Table F-1. Significant Crustal Faults

Source	Closest Mapped Distance[1] (km)	Mapped Length[1] (km)
Gales Creek	17.5	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 F-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 F-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 F-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 1,175 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 1,175 ft/s. Profile 2 reduced the assumed Vs_{30} by 20 percent (940 ft/s). Profile 3 increased the assumed Vs_{30} by 25 percent (1,470 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 only used in Profile 3 analysis 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 are presented in Table F-3. In our opinion, the use of multiple attenuation relationships addresses epistemic uncertainty.

Table C-3. Attenuation Relationships Weights for Seismic Sources

Faulting Type	Ground Motion Prediction Equation	Profile 3	Profiles 1 and 2
Shallow Faults and Shallow Crustal Background Seismicity	Abrahamson et al. (2014)	0.22	0.25
	Boore et al. (2014)	0.22	0.25
	Campbell and Bozorgnia (2014)	0.22	0.25
	Chiou and Youngs (2014)	0.22	0.25
	Idriss (2014)	0.12	0.0
Subduction (CSZ)	Zhao et al. (2006)	0.3	0.3
	BC Hydro (Abrahamson et al., 2016)	0.3	0.3
	Atkinson-Macias (2009)	0.1	0.1
	Atkinson and Boore (2003) Global Model	0.3	0.3
Deep Intraslab	Atkinson and Boore (2003) Cascadia Model	0.1667	0.1667
	Zhao et al. (2006)	0.33	0.33
	BC Hydro (Abrahamson et al., 2016)	0.33	0.33
	Atkinson and Boore (2003) Global Model	0.1667	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 F-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 F-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 F-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 F-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 two times the acceleration at 2 seconds. Figure F-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} = 1.045 g
- S_{D1} = 0.740 g

- S_{MS} = 1.568 g
- S_{M1} = 1.110 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 proposed new building location has a low landslide risk since it is below the ancient landslide terrain and is not near any steep slopes. Past landslides have been observed for the fill slope southwest of the existing school and there is a risk of additional failures for the fill slope as discussed in the main report. The proposed cuts north of the existing school are near the toe of the ancient landslide terrain. A discussion and recommendations for the proposed cuts north of the school are also provided in the main report.

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

According to the Tsunami Inundation Map Clat-08 Plate 1 from DOGAMI, the site is located within an inundation zone for only in the largest considered CSZ rupture.

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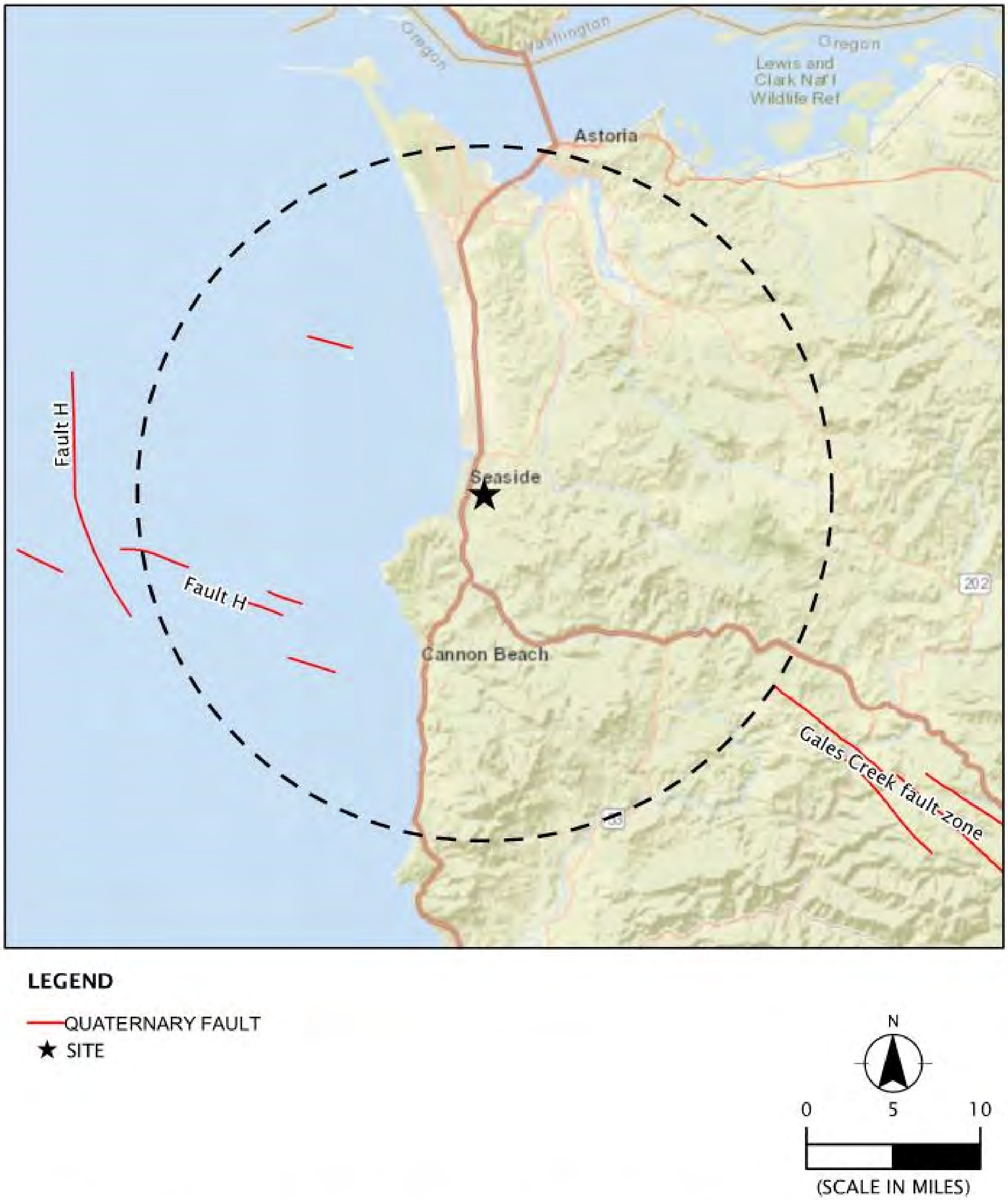

FAULTS PROVIDED BY THE USGS (2006) QUATERNARY FAULT AND FOLD DATABASE, ACCESSED 9/21/17 FROM: http//earthquakes.usgs.gov/hazards/qfaults/

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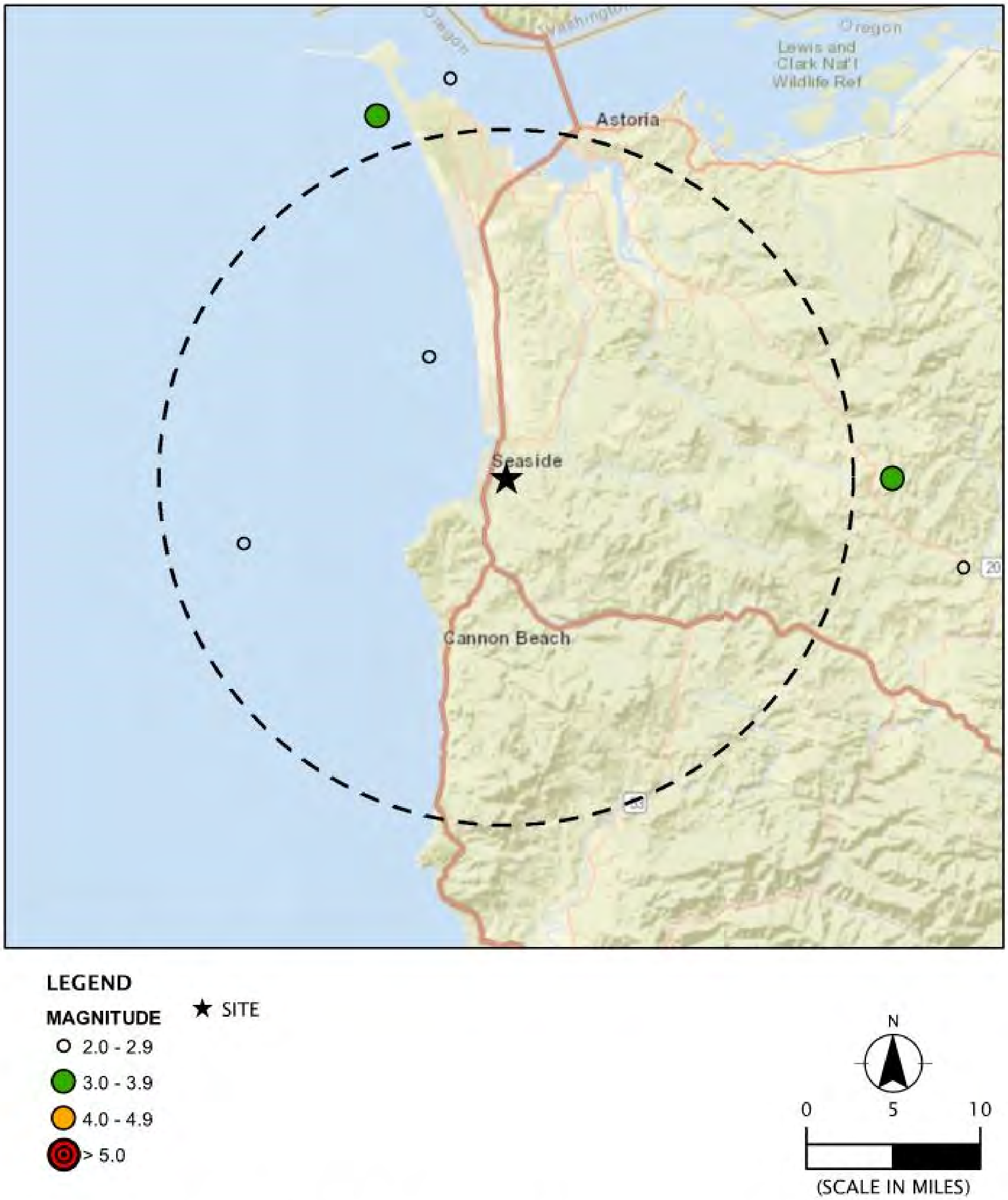
SEASIDESD-1-03-02	**QUATERNARY FAULT MAP**	
NOVEMBER 2017	SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION SEASIDE, OR	**FIGURE F-1**



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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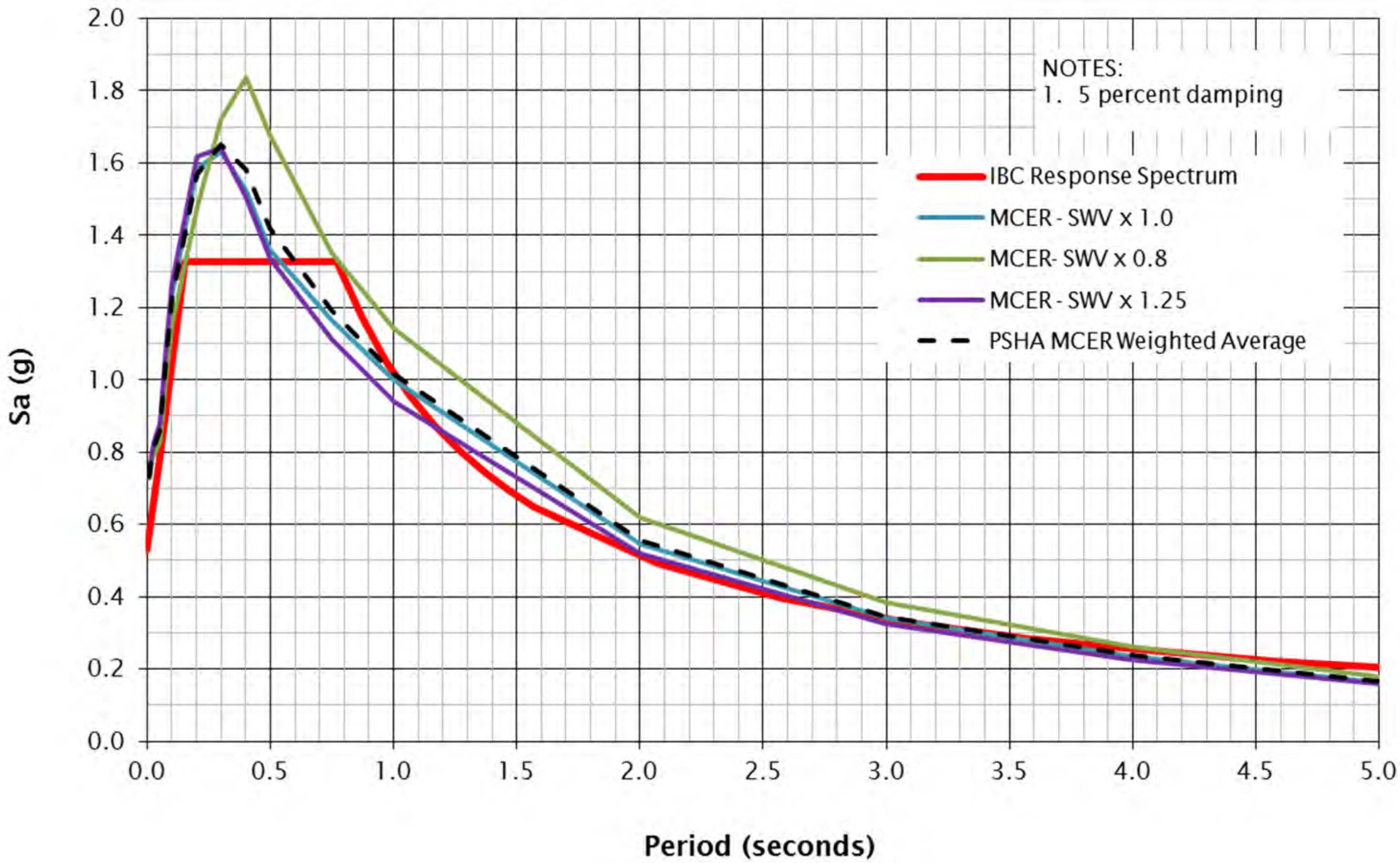
SEASIDESD-1-03-02	**HISTORICAL SEISMICITY MAP**		
NOVEMBER 2017	SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION SEASIDE, OR	**FIGURE F-2**	



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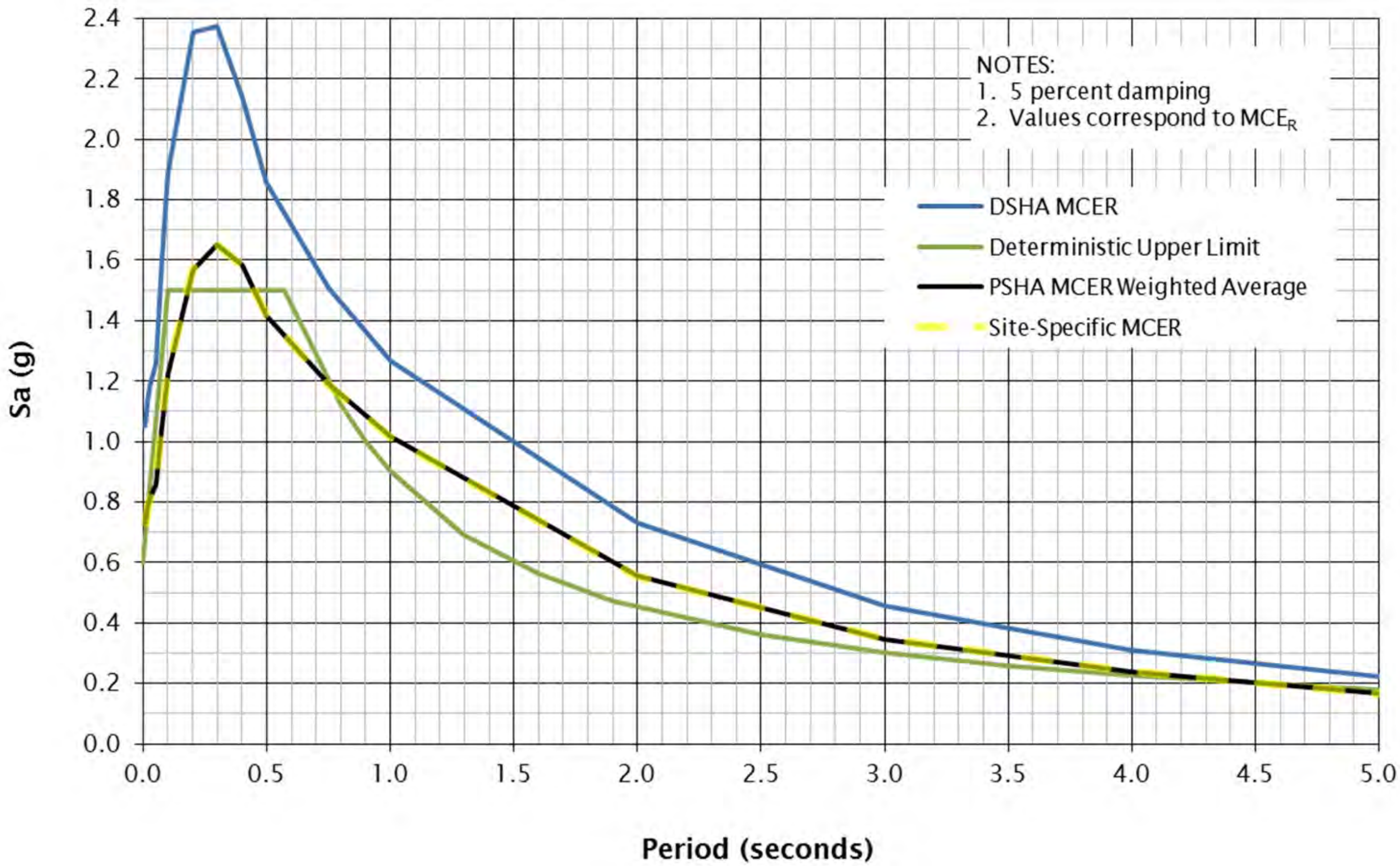
	NOVEMBER 2017	SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION SEASIDE, OR — **FIGURE F-3**



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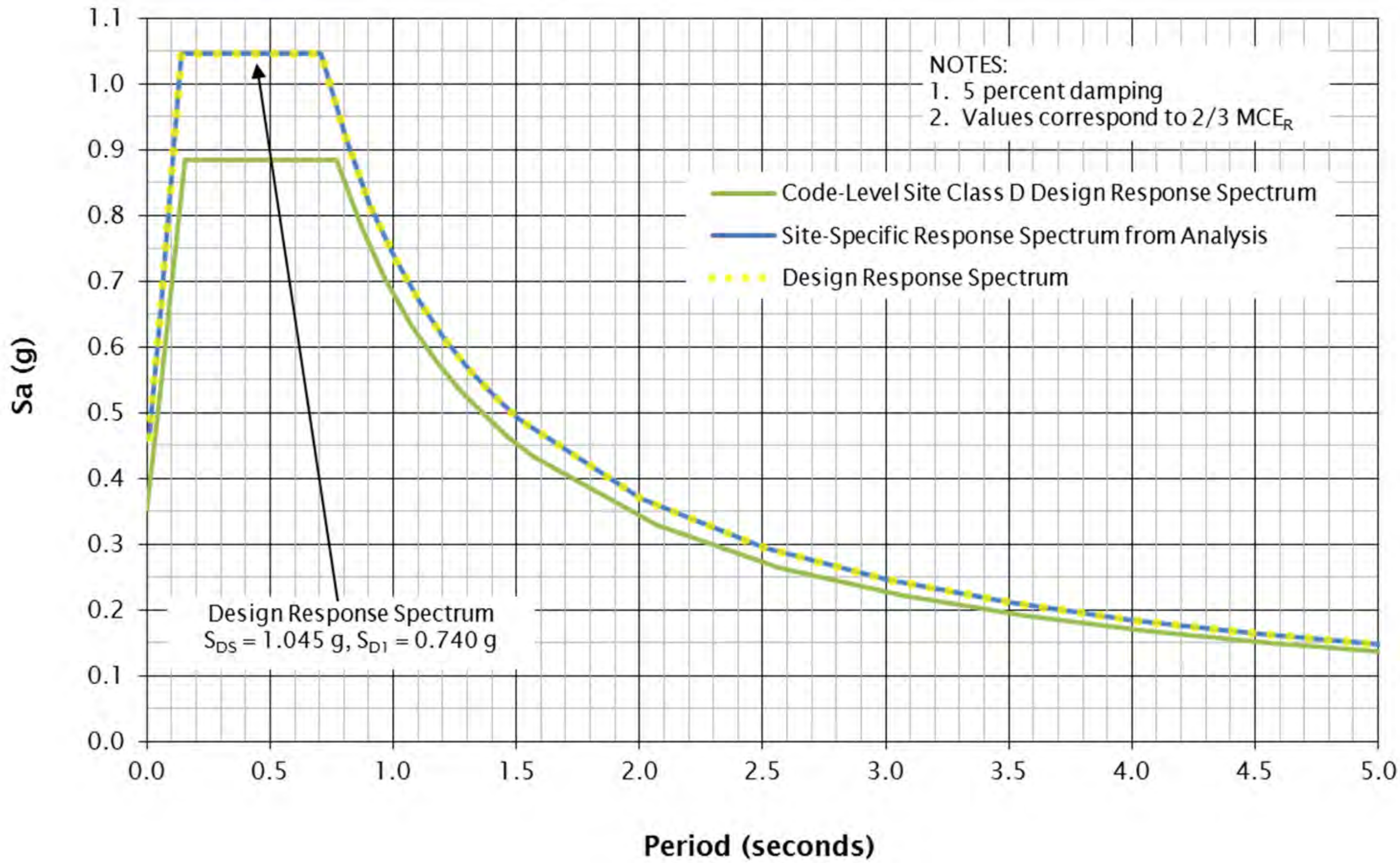
SEASIDESD-1-03-02	**SITE-SPECIFIC RESPONSE SPECTRA**	
NOVEMBER 2017	SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION SEASIDE, OR	**FIGURE F-4**



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SEASIDESD-1-03-02	**DESIGN RESPONSE SPECTRUM**		
NOVEMBER 2017	SEASIDE HEIGHTS ELEMENTARY SCHOOL EXPANSION SEASIDE, OR	**FIGURE F-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)
H:V	horizontal to vertical
IBC	International Building Code
km	kilometers
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 (2015)
PCC	Portland cement concrete
pcf	pounds per cubic foot
pci	pounds per cubic inch
PG	performance grade
psf	pounds per square foot
psi	pounds per square inch
SOSSC	State of Oregon Structural Specialty Code
SPT	standard penetration test
USGS	U.S. Geological Survey