REPORT OF GEOTECHNICAL ENGINEERING SERVICES

New Beaverton Middle School at Timberland
NW 118th Avenue and NW Stone Mountain Lane
Beaverton, Oregon

For
Beaverton School District
December 22, 2014

GeoDesign Project: BeavSchool-45-01

GeoDesign Inc

December 22, 2014

Beaverton School District
District Administration Center
16550 SW Merlo Road
Beaverton, OR 97006

Attention: Mr. Scott C. Johnson

Report of Geotechnical Engineering Services
New Beaverton Middle School at Timberland
NW 118th Avenue and NW Stone Mountain Lane
Beaverton, Oregon
GeoDesign Project: BeavSchool-45-01

GeoDesign, Inc. is pleased to submit our report of geotechnical engineering services for the proposed new Beaverton Middle School located in the Timberland development in Beaverton, Oregon. The approximately 16-acre site is located northeast of the intersection of NW 118th Avenue and NW Stone Mountain Lane. Our services for this project were conducted in accordance with our revised proposal dated October 23, 2014.

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.
Associate Engineer

George Saunders, P.E., G.E.
Principal Engineer

cc: Mr. Kurtis Zenner, Mahlum Architects (via email only)
Mr. Nick Saari, KPFF Consulting Engineers (via email only)
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1.0 INTRODUCTION

GeoDesign, Inc. is pleased to present this geotechnical engineering report for the proposed new Beaverton Middle School located in the Timberland development in Beaverton, Oregon. The approximately 16-acre site is located northeast of the intersection of NW 118th Avenue and NW Stone Mountain Lane. Figure 1 shows the site location relative to existing topographic and physical features.

We understand the project will consist of a new one- to two-story school building; parking area; and associated athletic fields, tennis courts, and play areas. Current plans consist of the building in the southwest portion of the site, athletic fields at the north end and east-central portion of the site, a bus drop off and parking at the south end, and a car drop off area and parking area on the north side of the building as shown Figure 2. The southern end of the building, including the basketball courts, will be partially embedded below grade.

KPFF Consulting Engineers indicated maximum column loads will be less than 200 kips and maximum wall loads of less than 12 kips per lineal foot. We have assumed maximum floor slab loads of 100 psf. Based on the preliminary grading plan provided by Cameron McCarthy, the majority of the site will require cuts ranging from minor depths up to 3 to 5 feet deep. However, deeper cuts will be required at the southwest corner and southern end of the site. The finish floor grades for the basketball courts at the southwest corner of the site will be as low as elevation 294 feet, resulting in up to a 19-foot cut. In addition, the finished grade of the southern bus pavement area will generally vary between elevation 292 and 293, resulting in up to 9 feet of cut. The main entrance to the school at the southwest corner of the site will also include cuts deeper than 10 feet. A small area with fills of up to 10 feet is planned for the east end of the tennis courts in the southeast corner of the site.

For your reference, acronyms 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 work included the following:

- Coordinated and managed the field evaluation, including utility checks, site access, and scheduling of subcontractor and GeoDesign field staff.
- Reviewed our prior work and geology maps for the area.
- Completed the following subsurface explorations at the site:
  - Fourteen borings to depths of 16.5 to 36.5 feet BGS
  - One CPT probe advanced to practical refusal at a depth of 59.4 feet BGS
  - Shear wave velocity testing at 2-meter intervals in the CPT probe
  - Seven test pits to depths of 5.0 to 8.5 feet BGS
  - Two infiltration tests at locations selected based on correspondence with Mahlum Architects and Cardno
- Obtained soil samples at select depths in the explorations.

- Classified the materials encountered in, and maintained a detailed log of, each exploration.
- Complete the following laboratory tests on selected samples:
  - Fifty-eight moisture content determinations in general accordance with ASTM D 2216
  - Three moisture density determinations in general accordance with ASTM D 7263
  - Four Atterberg limits determinations in general accordance with ASTM D 4318
  - Two percent fines determinations in general accordance with ASTM C117
  - Two consolidation tests in accordance with ASTM D2435
- 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 shallow foundations for the project. Our recommendations include allowable bearing capacity, lateral resistance parameters, and settlement estimates.
- Provide recommendations for preparation of floor slab subgrade.
- Recommend design criteria 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 asphalt pavements for on-site access roads and parking areas, including subbase, base course, and asphalt paving thickness.
- Provided recommendations for subsurface drainage of foundations and roadways, as necessary.
- Provided recommendations for IBC seismic coefficients.
- Prepared this written report summarizing the results of our geotechnical evaluation.
- Prepared a site-specific seismic hazard study for the site, which is presented in an appendix of this report.

3.0 BACKGROUND

GeoDesign has extensive experience on the school site and the surrounding areas of the Timberland development dating back to 2004. Our initial geotechnical explorations and reports for the Timberland Development (former Teufel Nursery site), including the school site, were completed in 2004 and 2005 (GeoDesign, 2004; GeoDesign 2005). Our early explorations for the Timberland development included test pits and borings. The native soil at the Timberland site generally consists of medium stiff to very stiff silt and clay. Basalt bedrock was encountered at depths between 31 and 40 feet BGS in three borings at the Timberland site and is exposed at the base of the nearby Cedar Mills Falls, but was not encountered in five other deeper borings advanced at the Timberland site to depths between 41.5 and 61.5 feet BGS.

GeoDesign was previously retained by Polygon Northwest Company to provide construction observation services for the development of the Timberland site. Development of the site included placing structural fill over the majority of the school site with fill depths ranging up to nearly 30 feet at the southern end of the site. Cuts of roughly 10 feet or less were also completed at the northern end of the site. We provided construction observation and testing for the fills placed on the school site during 42 site visits in 2005, 9 site visits in 2007, and 6 site visits in 2008. Bones Construction conducted the mass grading operations, and we conducted proof rolling and density testing to evaluate the structural fills placed on the school site. The fill was placed and compacted as structural fill to the extent of our observations and testing.

4.0 SITE CONDITIONS

4.1 GEOLOGIC CONDITIONS

The site is located in the eastern portion of the Tualatin Basin physiographic province. The Coast Range and Chehalem Mountains bound the basin to the west and the Tualatin Mountains and Portland Hills bound the basin to the east. The geologic profile in the vicinity of the site consists of a mix of catastrophic flood deposits and Portland Hills silt underlain by the Boring volcanics. The CRBG is considered the basement bedrock at this site and is present below depths ranging between 450 and 600 feet BGS (Wilson, 1998; Madin, 1990; Schlicker and Deacon, 1967).

The near-surface geologic unit is mapped as Pleistocene Age (15,500 to 13,000 years before present) fine-grained catastrophic flood deposits. The unit is mapped on the lower elevations (less than 300 feet) of the site and consists of poorly consolidated, fine- to coarse-grained sand, silt, and clay. The catastrophic flood deposits originated from multiple outburst floods from glacial Lake Missoula during the last episode of glaciations (Orr and Orr, 1999). The thickness of this unit ranges from 30 to 60 feet in the site vicinity based on our review of published geologic data and a review of water well logs in the site vicinity (Wilson, 1998; Madin, 1990; Schlicker and Deacon, 1967). Pleistocene loess deposits are mapped at elevations of 300 feet on the site. The loess deposits (commonly called Portland Hills silt) consist of poorly consolidated, wind-blown, micaceous silt that mantles the highlands surrounding the Tualatin Basin.

4.2 SURFACE CONDITIONS

The site is vacant and bound by NW Stone Mountain Lane to the south, NW 118th Avenue to the west, residential properties in the Timberland development to the north, and residential properties to the east. The site generally slopes gently to the south as shown by the contours on Figure 2. A roughly 40- to 80-foot-wide ramp is benched into steeper fill slopes at the south end of the site, which range up to grades of approximately 2H:1V. A zone of drain rock presumably for a french drain lines the toe of the slope at the edge of a landscaping strip for NW Stone Mountain Lane. Another ramp slopes up the east edge of the site. Elevations at the site range from approximately 342 feet at the northeast corner down to 289 feet at the southeast corner. The site is vegetated with short grass and weeds. At the time of our explorations there was a small stockpile of crushed rock in the west-central portion of the site.

4.3 SUBSURFACE CONDITIONS

4.3.1 General

We explored subsurface conditions by drilling 14 borings (B-1 through B-14) to depths ranging between 16.5 and 36.5 feet BGS, excavating 7 test pits (TP-1 through TP-7) to depths ranging between 5.0 and 8.5 feet BGS, and pushing one CPT probe (CPT-1) to refusal at a depth of 59.4 feet BGS. The approximate exploration locations are shown on Figure 2. The boring and test pit logs and results of the laboratory testing are provided in Appendix A. The CPT logs are provided in Appendix B.

In general, the soil conditions encountered consist of variable amounts of previously documented structural fill overlying alluvial silt and clay. The alluvial soil is underlain by basalt, which was presumably encountered at the depth of refusal of 59.4 feet BGS in CPT-1. Figure 2 also summarizes the depth of fill encountered in the explorations. The following sections provide a more detailed description of the geologic units encountered.

4.3.2 Structural Fill

We observed previously placed structural fill soil consistent with our prior construction observation to depths ranging up to 30.0 feet BGS as indicated on Figure 2. The fill depth generally increases to the south up to the top of the crest in the fill slope. The structural fill consists of medium stiff to hard silt and clay. Most of the SPT blow counts indicate the fill is stiff to very stiff. Variable fractions of sand and gravel and trace organics and debris were generally encountered in the fill material. The trace organics included pieces/fragments of charcoal, roots, wood, and bark, and the trace debris included plastic pieces, concrete pieces, and geotextile fabric. Cobbles were also encountered in the fill in test pits TP-2, TP-3, TP-5, TP-6, and TP-7. One Atterberg limits test indicates the fill soil exhibits low to moderate plasticity. Laboratory testing of a sample of the fill indicates that the moisture content was approximately 17 to 27 percent at the time of our explorations.

4.3.3 Alluvial Silt and Clay

Alluvial silt and clay was encountered beneath the fill or at the surface in the explorations and extends to the maximum depth of the boring explorations at 36.5 feet BGS and to the depth of refusal interpreted (and based on our prior work at the Timberland site) as basalt at 59.4 feet BGS in CPT-1. The silt and clay is generally medium stiff to stiff but ranges from soft to very stiff. The alluvial silt and clay contains variable fractions of fine sand. Atterberg limits testing indicates the soil generally exhibits medium plasticity but the plasticity varies from low to high. Consolidation testing indicates the alluvial silt and clay is over-consolidated and moderately compressible. Laboratory testing indicates the silt and clay had moisture contents of approximately 22 to 41 percent at the time of our explorations.

4.3.4 Groundwater

Groundwater was encountered at 26 feet BGS (elevation 290 feet) in boring B-5 and at 25 feet BGS (elevation of 288 feet) in boring B-8 but was not encountered in the other borings. Perched groundwater seepage was also encountered in test pit TP-1 at a depth of 7.5 feet BGS, and an area of standing water was observed at the ground surface near the base of the fill slope in the southeast corner of the site. We anticipate perched water can be encountered near or at the ground surface, 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 INFILTRATION TESTING

Infiltration tests were conducted in test pits TP-1 and TP-2 in native silt at depths of 4.0 and 5.0 feet BGS, respectively. Infiltration rates were determined using an encased falling head test method under low head conditions of approximately 1 foot after allowing the soil to saturate.

Representative grab samples were collected below the infiltration test depths for grain-size analysis. Table 1 presents a summary of infiltration test results and fines content determinations. The exploration logs and grain-size analyses are presented in Appendix A.

Table 1. Field Infiltration Test Results

Exploration	Depth (feet BGS)	Observed Infiltration Rate (inches/hour)	Fines Content[1] (percent)
TP-1	4	0.6	89
TP-2	5	0.4	82

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

The infiltration rates provided above are measured rates on native soil with no factor of safety. Additional correction factors should be applied to the measured infiltration rates by the civil engineer during design to account for the degree of long-term maintenance and influent/pre-treatment control, as well as the potential for long-term clogging due to siltation and bio-buildup, depending on the proposed length, location, and type of infiltration facility. If built, we recommend that the infiltration facilities be equipped with a redundant overflow system.

6.0 CONCLUSIONS

Based on our review of the available information, the results of our explorations, and our laboratory testing and analyses, it is our opinion that the proposed buildings with the assumed loads previously stated can be supported by shallow foundations bearing on structural fill or native soil at the site and that the site can be developed as proposed. Design criteria are provided in the "Foundation Support Recommendations" section of this report. The following items will have an impact on design and construction of the proposed project.

The on-site soil can be used for structural fill. However, 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, as was required for the Timberland earthwork, 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.

Trafficability of the surficial fine-grained soil will be difficult during periods of wet weather or when the moisture content of the surface several feet of material is more than a few percentage points above optimum. This will likely be throughout the year, except mid-summer through early fall. When wet, the on-site silty soil is susceptible to disturbance and will provide inadequate support for rubber-tired construction equipment. Granular haul roads and working pads and/or cement-treated subgrades can be utilized to support construction traffic on the site during wet conditions.

Trace organics and debris was encountered in some of the fill material at the site. If debris or organics are encountered in the subgrade of structural elements such as foundations, slabs, and pavements, we may recommend removal and replacement with granular structural fill. We recommend the project include a contingency for the over-excavation and replacement of organics and/or debris in the fill at the site.

A wedge of fill is planned for the eastern tennis court in the southeast corner of the site. The fill area will range in depth up to 10 feet and be contained by a new retaining wall. The new structural fill should be benched into the existing slope as recommended in the "Structural Fill" section of this report. We recommend the flatwork for the tennis court fill area be postponed for at least four weeks after filling to final grade, unless survey data indicates that settlement is complete prior to that time.

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

7.0 DESIGN

7.1 PERMANENT SLOPES

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

7.2 DRAINAGE

7.2.1 General

Groundwater was encountered at 26 feet BGS (elevation 290 feet) in boring B-5, at 25 feet BGS (elevation of 288 feet) in boring B-8, shallow perched water was observed in test pit TP-1 at a depth of 7.5 feet BGS, and an area of standing water was observed at the ground surface near the base of the fill slope in the southeast corner of the site. Moreover a zone of drain rock and presumably a french drain exists along the toe of the slope at the edge of a landscaping strip for NW Stone Mountain Lane.

7.2.2 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. During rough and finished grading of the building site, the contractor should keep all footing excavations and building pads free of water.

7.2.3 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 buildings be sloped away from the buildings to facilitate drainage away from the buildings.

7.2.4 Foundation Drains

We recommend that perimeter foundation drains be installed in all areas where the finished floor grade will be below existing grades. The drainage for embedded walls such as required for the embedded building areas at the southeastern corner of the site should be completed as recommended in the "Retaining Structures" section of this report. The embedded wall should be water proofed.

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 to the ground surface and is wrapped in a drainage geotextile. The invert elevation of the drainpipe should be installed at least 18 inches below the elevation of the floor slab.

The drain rock and drainage geotextile should meet the requirements specified in the "Structural Fill" section of this report. The drain rock and geotextile should extend up the side of embedded walls to within a foot of the ground surface and geotextile wrapped over the top of the drain rock as recommended in the "Retaining Structures" section of this report.

7.2.5 Floor Slab Drains

General recommendations for drainage and vapor barriers under floor slabs are provided in the "Floor Slabs" section of this report.

Specifically, the finish floor grades for the basketball courts at the southwest corner of the site will be as low as elevation 294 feet, resulting in up to a 19-foot cut. In addition, the finished grade of the southern bus pavement area will generally vary between 292 and 293 feet, resulting in up to 9 feet of cut. The main entrance to the school at the southwest corner of the site will also include cuts deeper than 10 feet. Based on the preliminary finished grades and the groundwater depths and several indicators of water toward the southern end and southwest corner of the site, it is our opinion that floor slab drains will be needed for the deeply embedded buildings at the southwest corner of the site. A typical slab drainage detail is provided on Figure 3. The specific extent of the slab drainage should be based on the final grading plan. The drain rock and drainage geotextile should meet the requirements specified in the "Structural Fill" section of this report.

7.2.6 French Drains

We recommend that french drains be installed to intercept groundwater near the deep cuts at the site. The actual alignment and depth of the french drain should be based on the final grading plan. Depending on the extent of cutting required, it may be beneficial to install the french drain prior to excavation to assist in dewatering the site during construction.

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 drain-pipe should not be tied to a stormwater drainage system without backflow provisions. A typical cross section of a french drain is shown on Figure 4. 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 "Structural Fill" section of this report.

7.3 SEISMIC DESIGN CRITERIA

7.3.1 Seismic Design Parameters

Seismic design is prescribed by 2014 SOSSC and the 2012 IBC. Based on the results of our explorations and shear wave velocity testing, the soil profile for the upper 100 feet of the site corresponds to a site class D. Table 2 presents the site design parameters prescribed by the 2012 IBC for the site. The building codes require that seismic design parameters associated with a 2 percent probability of being exceeded in a 50-year period be used in design of the critical structures such as schools. Appendix C includes a site-specific seismic study for the site.

Table 2. IBC Seismic Design Parameters

Seismic Design Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
MCE Spectral Acceleration	S_s = 1.02 g	S_1 = 0.44 g
Site Class	D	
Site Coefficient	F_a = 1.09	F_v = 1.56
Adjusted Spectral Acceleration	S_{MS} = 1.11 g	S_{M1} = 0.69 g
Design Spectral Response Acceleration Parameters	S_{DS} = 0.74 g	S_{D1} = 0.46 g
Design Spectral PGA	0.30 g	

7.3.2 Liquefaction and Lateral Spreading

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

7.4 FOUNDATION SUPPORT RECOMMENDATIONS

7.4.1 General

Based on the results of our explorations and analysis, all structures associated with the proposed school facility can be supported by conventional spread footings resting on undisturbed native soil or structural fill overlying firm native soil. 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.

Fine-grained silt and clay will be present at the base footings and is prone to disturbance during when above the optimum moisture content for compaction and during wet weather. Accordingly, we recommend a minimum of 3 inches of gravel be placed in the base of all footings and compacted until "well keyed" after evaluation of the subgrade by GeoDesign and prior to forming and rebar placement regardless of the time of year construction occurs.

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 values above are net bearing pressures; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and can be doubled for short-term loads resulting from wind or seismic forces.

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

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

7.4.2 Resistance to Sliding

Lateral loads on footings can be resisted by passive earth pressure on the sides of the structures 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 250 pcf, modeled as an 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. A coefficient of friction equal to 0.30 can be used for the resistance to sliding for footings at the project site.

7.4.3 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, and softened subgrades (if present) have been removed. Localized deepening of footing excavations may be required to penetrate debris, fill, or deleterious material, if encountered.

7.5 FLOOR SLABS

Satisfaction subgrade support for building floor slabs supporting floor loads of up to 100 psf areal loading can be obtained provided the subgrade is prepared in accordance with the "Site Preparation" section of this report. 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 "Structural Fill" section of this report. 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 150 pci can be used for slabs-on-grade constructed on subgrade prepared as recommended in the "Construction" section of this report.

The basketball courts at the southwest portion of the site will require up to 20-foot cuts with finished floor grades as low as elevation 294 feet. Groundwater was encountered at elevations of 288 to 290 feet in our explorations. Accordingly, we recommend that slab drains be installed below the slab as discussed in the "Floor Slab Drains" section of this report.

While groundwater is unlikely to be encountered within the slab subgrade material, the native soil is fine grained and will tend to maintain a high moisture content. 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, then we recommend that 10- or 15-mil Stego Wrap be considered for this project. The recommended procedures for installing Stego Wrap are to pour the floor slab concrete directly over the vapor barrier. We recommend that the structural engineer be contacted to determine if the mix design for the concrete should be modified assuming the above-referenced construction sequence. Actual selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team.

7.6 RETAINING STRUCTURES

7.6.1 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 12 feet in height. Feasible wall types for the project may include, but are not limited to, MSE walls, modular block (Ultrablock) walls, and CIP walls. The advantage of MSE walls over CIP walls is generally cost; however, the size of the backfill zone and required excavation for cut MSE walls can usually be minimized with the use of CIP walls. Ultrablock walls are typically economical up to total wall heights of 7.5 to 10 feet. Ultrablock walls above 7.5 to 10 feet in height generally require the use of reinforcing elements or considerably more blocks in a stacking pattern, and other MSE wall types may become more cost effective above this height. The construction of MSE walls should consider that future excavations, such as utility trenches, will be prevented within the reinforced zone of the wall. Segmental block walls, such as Allan Block or Keystone, may be feasible without geogrid reinforcement for smaller walls less than 4 feet in height, depending on the proximity to slopes and distance from traffic loading.

In addition to internal stability, global stability considerations will be necessary, particularly if slopes are present above and/or below the wall or terraced walls are to be included.

7.6.2 Assumptions

Our retaining wall design recommendations are based on the following assumptions: (1) the walls consists of a conventional CIP concrete, prefabricated modular block, conventional segmental block, or MSE retaining walls, (2) the walls will be less than 12.0 feet in height, (3) the backfill is drained and consists of imported granular material, and (4) the appropriate wall surcharges are included in the design as described in this section.

7.6.3 Wall Design Parameters

CIP or gravity 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 $6H^2$ pounds per lineal foot of wall (where H is the height of the wall in feet). The load should be applied as a distributed load with the centroid located at a distance of 0.6H from the base of the wall.

7.6.4 Soil Parameters for Wall Designs

MSE or other walls can be designed using the recommended soil parameters presented in Table 3. The material used to backfill behind the walls is discussed in the wall "Wall Backfill and Drains" section of this report as well as the "Structural Fill" section of the report.

Table 3. Wall Design Parameters

Soil Type	Unit Weight (pcf)	Friction Angle, ϕ (degrees)	Cohesion (psf)
Retained Soil – Granular Wall Backfill[1]	130	35	0
Retained Soil – MSE Granular Backfill[2]	130	35	0
Retained Soil – Fill Walls General Borrow Structural Fill[3]	120	28	50
Retained Soil – Cut Walls Native Soil or New Fill	120	28	50
Foundation Soil – Native Soil or New Fill[4]	120	28	50

1. Wall backfill for prefabricated modular block, conventional segmental block, and cast-in-place walls should meet the specifications provided in OSSC 00510.12 – Granular Wall Backfill.
2. Wall backfill for MSE walls should meet the specifications provided in OSSC 02630 (Base Aggregate)
3. Retained soil should meet the specifications provided in OSSC 00330.12 – Borrow Material.
4. Foundation soil should be dense, native or fill soils prepared in conformance with the "Site Preparation" section of this report.

Seismic forces should be modeled based on the pseudo-static approach developed by the Mononobe-Okobe method. We recommend using a seismic coefficient of half of the PGA provided in Table 2 when analyzing internal stability.

7.6.5 Wall Surcharges

Where traffic loads are located within a horizontal distance from the top of the wall equal to one-half the wall height, the lateral earth pressure shall be increased by a surcharge load equal to 2 feet of soil (assuming a soil density of 125 pcf). The traffic load should not be included for seismic analysis.

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 4) when designing walls that retain sloping soil.

Table 4. 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, then additional pressures may need to be accounted for in the wall design. For alternate surcharge loadings GeoDesign should be contacted to provide appropriate wall surcharges based on the actual magnitude and configuration of the applied loads.

7.6.6 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. The top of temporary slopes should be located at least 5 feet from pavements, utilities, buildings, or other such structures. Sloughing of temporary slopes can be expected and maintenance during construction will likely be required, particularly during wet weather. All temporary slopes should be made and maintained in accordance with applicable OSHA and state regulations.

7.6.7 Wall Foundations

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

7.6.8 Base Excavation

We recommend that walls be constructed on a leveling course placed over the subgrade excavation. The leveling course should consist of crushed rock placed over the subgrade soil. The leveling course of crushed rock should have a minimum layer thickness of 6 inches and meet the requirement of the retaining wall leveling pad in the "Structural Fill" section of this report.

7.6.9 Wall Backfill and Drains

Granular wall backfill materials placed behind modular block, conventional segmental block, and CIP concrete walls should extend at least 1 foot behind the heel of the wall. For conventional segmental block walls that are taller than 4 feet, the granular wall backfill should extend a minimum horizontal distance equal to ½H (where H is the height of the retaining wall) from the back of the wall. The granular wall backfill materials should extend to at least 1 foot below the top of the wall where the backfill is level and at least to the top of the wall where the backfill is sloped. Sloping backfill above the retaining wall may consist of general borrow structural fill. The specifications for the fill materials are discussed in detail in the "Structural Fill" section of this report.

A minimum 12-inch-wide by 12-inch-tall zone of drain rock should be placed at the heel of all prefabricated modular block, conventional segmental block, and CIP concrete retaining walls. For MSE retaining walls, the zone of drain rock should extend the full height of the wall and should be placed at the back of the reinforced zone. For embedded building walls, the drain rock should extend from at least 1 foot below the final slab grade to the full height of the wall and be enveloped by a geotextile fabric.

Perforated collector pipes should be embedded at the base of the drain rock. The drain rock should meet the requirements provided in the "Structural Fill" section of this report. The perforated collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe(s) should not be tied directly into stormwater drain systems unless measures are taken to prevent backflow into the wall's drainage system.

The wall backfill should be compacted as recommended in the "Structural Fill" section of this report. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor).

7.6.10 Construction Considerations

All footing subgrades should be evaluated by the project geotechnical engineer or their representative to confirm suitable bearing conditions. Observations should also confirm that all loose or soft material, organics, unsuitable fill, prior topsoil zones, and softened subgrades (if present) have been removed. Localized deepening of footing excavations may be required to penetrate any deleterious materials.

If construction is undertaken during periods of wet weather, we recommend placing at least 3 inches of imported granular material over the prepared footing subgrades to help protect the subgrade from disturbance due to the elements and foot traffic. The imported material should meet the specifications for "Imported Granular Fill" as discussed in the "Structural Fill" section of this report.

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

7.7 PAVEMENT RECOMMENDATIONS

7.7.1 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. We anticipate that AC pavements will be used for passenger car drive aisles and parking areas. Bus traffic areas could consist of either AC or PCC. Pavements should be installed on firm, undisturbed native subgrade, undisturbed previously placed structural fill, or new structural fill as described in the "Site Preparation" and "Structural Fill" sections of this report. If near-surface soil is cement amended, we should be contacted to revise our recommendations.

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 for AC and 4.5 and 2.5 for PCC pavement.
- Reliability and standard deviations of 85 percent and 0.45 for AC pavement and 85 percent and 0.40 for PCC pavement.
- Structural coefficient of 0.42 and 0.10 for the asphalt 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 and an effective k-value of 160 psi per inch for native or fill subgrade prepared in accordance with the "Site Preparation" section of the report.

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

7.7.2 Flexible AC Pavement Recommendations

Based on the traffic assumptions provided above, we recommend the following AC pavement sections in Table 5.

Table 5. 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 "Structural Fill" section of this report.

7.7.3 PCC Pavement

Based on the traffic assumptions provided above, we recommend the following PCC pavement sections in Table 4.

Table 4. Recommended PCC Sections

Pavement Use	Busses per Day	PCC Thickness (inches)	Aggregate Base Thickness (inches)
Bus Areas	10	7.0	6.0
	20	7.0	6.0
	30	7.5	6.0

PCC should be Class 4000-3/4 (Paving) concrete according to OSSC 02001 (Concrete) with a minimum 28-day flexural strength of 600 psi and placed in general accordance with OSSC 00756 (Plain Concrete Pavement). Dowel bars and placement should conform to OSSC 00756.43 (Placing Dowel Bars and Tie Bars). Joints should be constructed in general accordance with OSSC 00756.48 (Joints) with a maximum transverse joint spacing of 15 feet. In addition, the length-to-width ratio for any panel should be at least 0.75 and should not exceed 1.25.

8.0 CONSTRUCTION

8.1 EROSION CONTROL

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

8.2 SITE PREPARATION

8.2.1 Demolition

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

8.2.2 Stripping

The existing topsoil zone should be stripped and removed from all fill areas. Based on our explorations, the average depth of stripping will be approximately 3 inches, although greater stripping depths may be required to remove localized zones of loose or organic soil. Greater stripping depths should be anticipated in areas with thicker vegetation and along the base of draws. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaped areas.

8.2.3 Subgrade Evaluation

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

8.2.4 Test Pit Locations

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

8.3 SUBGRADE PROTECTION

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

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

As an alternative to thickened crushed rock sections, haul roads and utility work zones may be constructed using cement-amended subgrades overlain by a crushed rock wearing surface. If this approach is used, 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. Cement amendment is discussed in the "Structural Fill" section of this report.

8.4 EXCAVATION

8.4.1 Excavation and Shoring

The soil conditions at the site consist primarily of silt and clay. Cuts in silt and clay should be readily completed with conventional excavation equipment. Temporary excavation sidewalls should stand vertical to a depth of approximately 4 feet provided groundwater seepage is not observed in the sidewalls. Open excavation techniques may be used to excavate trenches with depths between 4 and 8 feet provided the walls of the excavation are cut at a slope of 1H:1V and groundwater seepage is not present. At this inclination, the slopes may slough and require some ongoing repair. Excavations should be flattened to 1½H:1V or 2H:1V if excessive sloughing or raveling occurs. In lieu of large 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, then caving of the sidewalls may occur. The presence of caved material will limit the ability to properly backfill and compact the trenches. The contractor should be prepared to fill voids between the box shoring and the sidewalls of the trenches with sand or gravel before caving occurs.

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

8.4.2 Trench Dewatering

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

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

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

8.4.3 Safety

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

8.5 STRUCTURAL FILL

8.5.1 General

Fill should be placed on subgrade that has been prepared in conformance with the "Site Preparation" section of this report. 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, level benches should be cut into the existing sloping surfaces to remove the surface loose material and should extend into the structural fill of the existing embankment. The benches should be a minimum of 10 feet wide or 1½ times the width of the compaction equipment, whichever is wider.

8.5.2 On-Site Soil

The material at the site should be suitable for use as general structural fill provided it is properly moisture conditioned; free of debris, organic material, and particles over 6 inches in diameter; and meets the specifications provided in OSSC 00330.12 (Borrow Material).

Based on laboratory test results, the moisture content of the on-site soil at the time of our explorations is above the optimum for compaction. Moisture conditioning (drying) will be required to use on-site soil for structural fill. Accordingly, extended dry weather (typically experienced between early July and mid-October) will be required to adequately condition and place the soil as structural fill. It will be difficult, if not impossible, to adequately compact on-site soil during the rainy season or during prolonged periods of rainfall.

When used as structural fill, native soil should be placed in lifts with a maximum uncompacted thickness of 6 to 8 inches and compacted to not less than 92 percent of the maximum dry density for fine-grained soil and 95 percent of the maximum dry density for granular soil, as determined by ASTM D 1557.

8.5.3 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.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557. During the wet season or when wet subgrade conditions exists, the initial lift should be approximately 18 inches in uncompacted thickness and should be compacted by rolling with a smooth-drum roller without using vibratory action.

8.5.4 Stabilization Material

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

8.5.5 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of well-graded granular material with a maximum particle size of 1½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 Sieve and should meet the specifications provided in OSSC 00405.13 (Pipe Zone Material). The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department.
Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of well-graded granular material with a maximum particle size of 2½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 Sieve and should meet the specifications provided in OSSC 00405.14 (Trench Backfill; Class B, C, or D). This material should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department. The upper 3 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D 1557.

Outside of structural improvement areas (e.g., roadway alignments or building pads) trench backfill placed above the pipe zone may consist of general fill material that is free of organics and material over 6 inches in diameter and meets the specifications provided in OSSC 00405.14 (Trench Backfill; Class A, B, C, or D). This general trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department.

8.5.6 Drain Rock

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

8.5.7 Aggregate Base Rock

Imported granular material used as base rock for building floor slabs and pavements should consist of ¾- or 1½-inch-minus material (depending on the application) and meet the requirements in OSSC 00641 (Aggregate Subbase, Base, and Shoulders). In addition, the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 Sieve. The base aggregate should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

8.5.8 Geotextile Fabric

8.5.8.1 Subgrade Geotextile

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

8.5.8.2 Drainage Geotextile

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

8.5.9 Soil Amendment with Cement

8.5.9.1 General

In conjunction with an experienced contractor, the on-site soil can be amended with portland cement to obtain suitable support properties. 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 cement used during treatment should be based on an assumed soil dry unit weight of 100 pcf.

8.5.9.2 Subgrade Stabilization

Specific recommendations based on exposed site conditions for soil amending can be provided if necessary. However, for preliminary design purposes, we recommend a target strength for cement-amended subgrade for building and pavement subbase (below base aggregate) soil of 100 psi. The amount of cement used to achieve this target generally varies with moisture content and soil type. It is difficult to predict field performance of soil to cement amendment due to variability in soil response, and we recommend laboratory testing to confirm expectations. Generally, 4 percent cement by weight of dry soil can be used when the soil moisture content does not exceed approximately 20 percent. If the soil moisture content is in the range of 25 to 35 percent, 5 to 7 percent by weight of dry soil is recommended. The amount of cement added to the soil may need to be adjusted based on field observations and performance. Moreover, depending on the time of year and moisture content levels during amendment, water may need to be applied during tilling to appropriately condition the soil moisture content.

For building and pavement subbase, we recommend assuming a minimum cement ratio of 5 percent (by dry weight). If the soil moisture content is in the range of 25 to 35 percent, 5 to 7 percent by weight of dry soil is recommended.

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

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

8.5.9.3 Other Considerations

On-site soil that because of elevated moisture contents would not otherwise be suitable for structural fill may be amended and placed as fill over a subgrade prepared in conformance with the "Site Preparation" section of this report. The cement ratio for general cement-amended fill can generally be reduced by 1 percent (by dry weight) relative to the recommendations above. 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.

8.5.9.4 Other Considerations

Portland cement-amended soil is hard and has low permeability. This soil does not drain well, nor is it suitable for planting. Future planted areas should not be cement amended, if practical, or accommodations should be made for drainage and planting. Moreover, cement amending soil within building areas must be done carefully to avoid trapping water under floor slabs. We should be contacted if this approach is considered. Cement amendment should not be used if runoff during construction cannot be directed away from adjacent wetlands.

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 cement and soil mixture.

- Spreading Equipment
    - Use a spreader capable of distributing the cement uniformly on the ground to within 5 percent variance of the specified application rate.
    - Use machinery that will not disturb the subgrade, such as using low-pressure "balloon" tires on the spreader vehicle. If subgrade is disturbed, the tilling/treatment depth shall extend the full depth of the disturbance.

- Compaction Equipment
    - Use a static, sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds for initial compaction of fine-grained soil (silt and clay), or an alternate approved by the geotechnical engineer.
    - Use a vibratory, smooth-drum roller with a minimum applied lineal force of 600 pounds per inch for final compaction, or an alternate approved by the geotechnical engineer.

9.0 OBSERVATION OF CONSTRUCTION

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

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

10.0 LIMITATIONS

We have prepared this report for use by Beaverton 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.

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

George Saunders, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
63114PE
OREGON
March 14, 2006
SHAWN DIMKE
EXPIRES: 12/31/15

REFERENCES

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FIGURES

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

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BEAVSCHOOL-45-01

DECEMBER 2014

VICINITY MAP

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE 1

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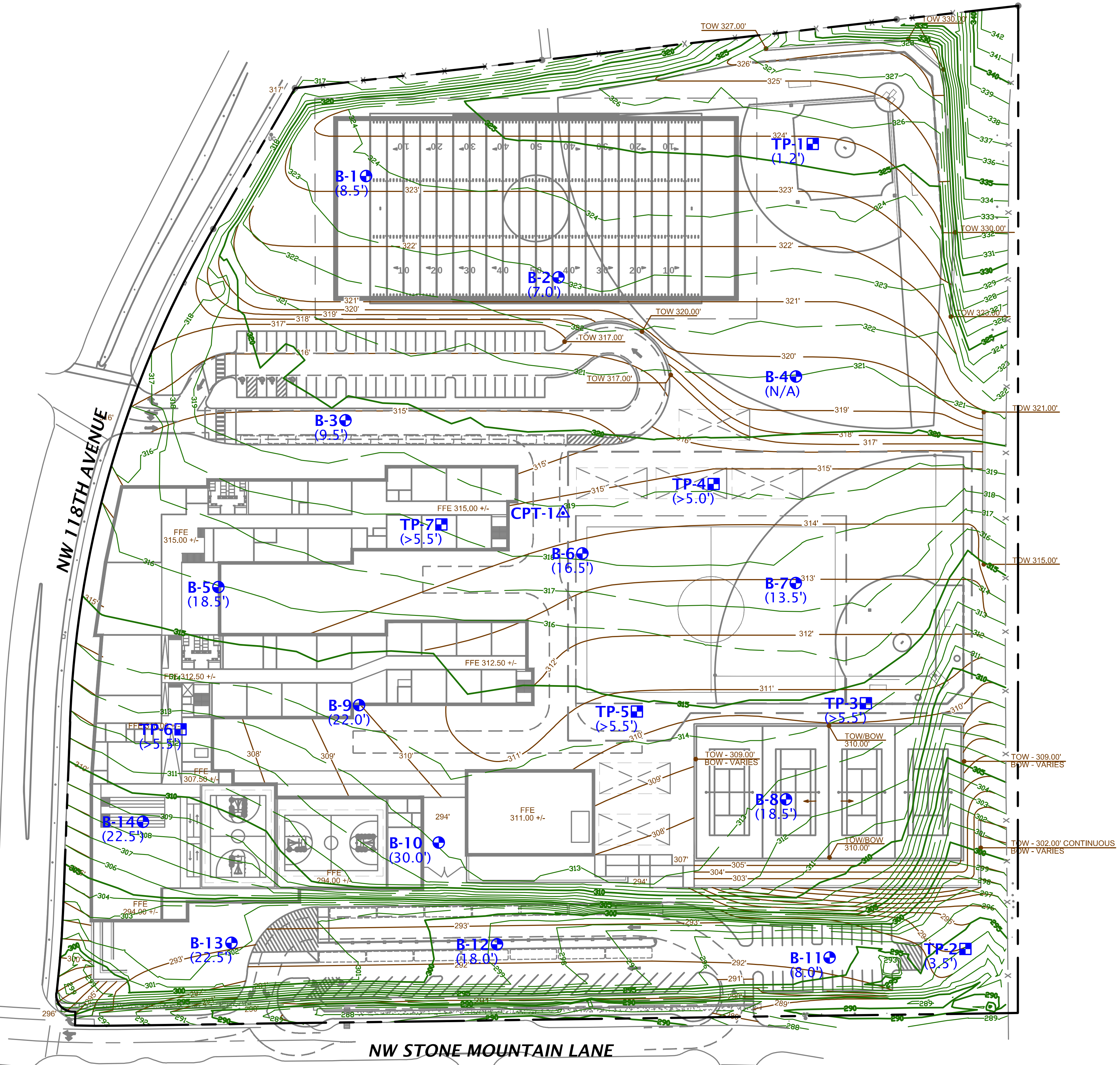
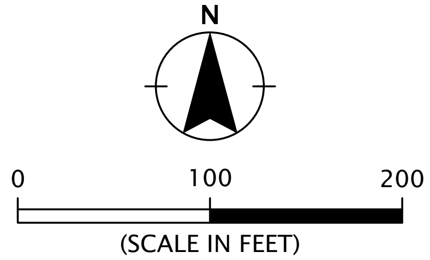


LEGEND:

B-1 ⊕	BORING
TP-1 ◩ (8.5')	TEST PIT DEPTH OF FILL
CPT-1 △	CONE PENETROMETER PROBE
N/A	NOT APPLICABLE
🟩	EXISTING TOPOGRAPHY
🟫	PROPOSED GRADING CONTOURS



NOTES:

1. SITE PLAN BASED ON DRAWING PROVIDED BY CAMERON MCCARTHY, DECEMBER 12, 2014.
2. EXISTING TOPOGRAPHY BASED ON HAGEDORN DRAWING DATED FEBRUARY 21, 2011 PROVIDED BY CARDNO.

FIGURE 2

SITE PLAN

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

BEAVSCHOOL-45-01

DECEMBER 2014

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APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

Our subsurface exploration program included drilling 14 borings (B-1 through B-14) to depths ranging between 16.5 and 36.5 feet BGS and excavating 7 test pits (TP-1 through TP-7) to depths ranging between 5.0 and 8.5 feet BGS at the approximate locations shown on Figure 2. The explorations were completed on November 10, 11, and 12, 2014 by Dan J. Fischer Excavating, Inc. of Forest Grove, Oregon. The borings were advanced with a trailer-mounted drill rig using solid-stem auger drilling techniques, and the test pits were excavated with a trackhoe. The explorations were observed by a member of our geology staff. We obtained representative samples of the various soil encountered in the explorations for geotechnical laboratory testing. Classifications and sampling depths are presented on the exploration logs included in this attachment.

Approximate locations of the explorations are shown on Figure 2. The locations of the explorations were determined in the field by pacing or measuring from existing site features. Surface elevations at the exploration locations were estimated by referencing exploration locations to elevation contours on a Hagedorn, Inc. survey provided by Cardno on November 21, 2014. This information should be considered accurate only to the degree implied by the methods used.

SOIL SAMPLING

We obtained representative samples of the various soil encountered in the explorations for geotechnical laboratory testing. Sampling intervals are presented on the exploration logs included in this appendix. Soil samples were obtained 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 obtained 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 obtained 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 included 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 and sampling intervals are presented on the exploration logs in this appendix.

LABORATORY TESTING

Laboratory tests were conducted on selected soil samples to confirm field classifications and determine the index engineering properties and strength characteristics. Locations of the tested samples are indicated on the exploration logs included in this appendix. Descriptions of the tests and results of the testing completed are presented below.

CLASSIFICATION

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

MOISTURE CONTENT

We tested the natural moisture content of selected samples obtained from the explorations 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 moisture contents are presented on the exploration logs in this appendix.

ATTERBERG LIMITS TESTING

The Atterberg limits (plastic and liquid limits) were performed on selected 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

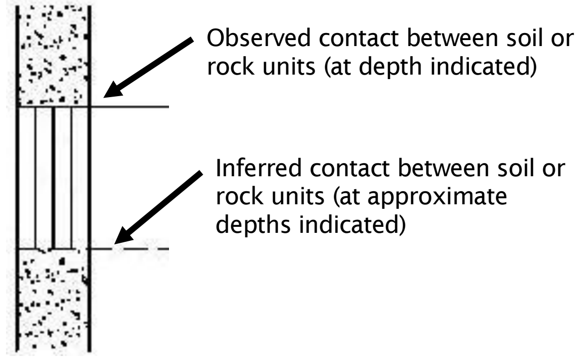
We performed one-dimensional consolidation tests in general accordance with ASTM D 2435 on relatively undisturbed samples obtained from the geotechnical borings. The test measures the volume change of a soil sample under predetermined loads. The results of the consolidation tests are included in this appendix.

GRAIN-SIZE TESTING

Grain-size testing was performed on selected soil samples to determine the distribution of soil particle sizes. The testing consisted of percent fines determination (percent passing the U.S. Standard No. 200 Sieve) analyses completed in general accordance with ASTM C 117 or ASTM D 1140 (P200). 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	PP	Pocket Penetrometer
CBR	California Bearing Ratio	P200	Percent Passing U.S. Standard No. 200 Sieve
CON	Consolidation		
DD	Dry Density	RES	Resilient Modulus
DS	Direct Shear	SIEV	Sieve Gradation
HYD	Hydrometer Gradation	TOR	Torvane
MC	Moisture Content	UC	Unconfined Compressive Strength
MD	Moisture-Density Relationship	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal
P	Pushed Sample		

ENVIRONMENTAL TESTING EXPLANATIONS

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

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EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOILS

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

CONSISTENCY - FINE-GRAINED SOILS

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

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

MOISTURE CLASSIFICATION

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

ADDITIONAL CONSTITUENTS

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

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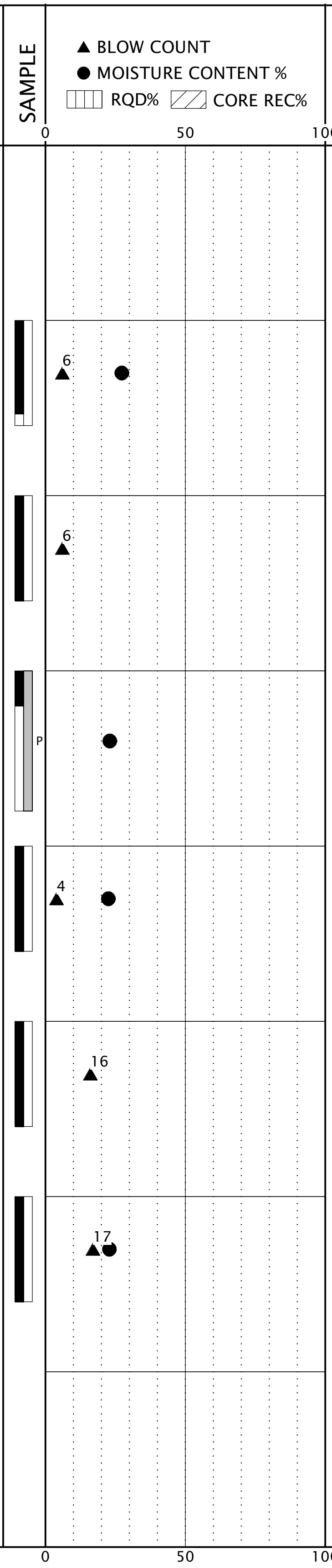
SOIL CLASSIFICATION SYSTEM

TABLE A-2

MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
Medium stiff, brown with red-brown mottled SILT (ML); moist (topsoil to 6 inches, 3-inch-thick root zone) - **FILL**.	324.0		
			DD = 88 pcf
Soft to medium stiff, brown SILT (ML); moist.	315.5 8.5	DD	
very stiff, brown to gray-brown, trace clay at 12.5 feet			
Exploration completed at a depth of 16.5 feet.	307.5 16.5		



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: CMC
COMPLETED: 11/10/14

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

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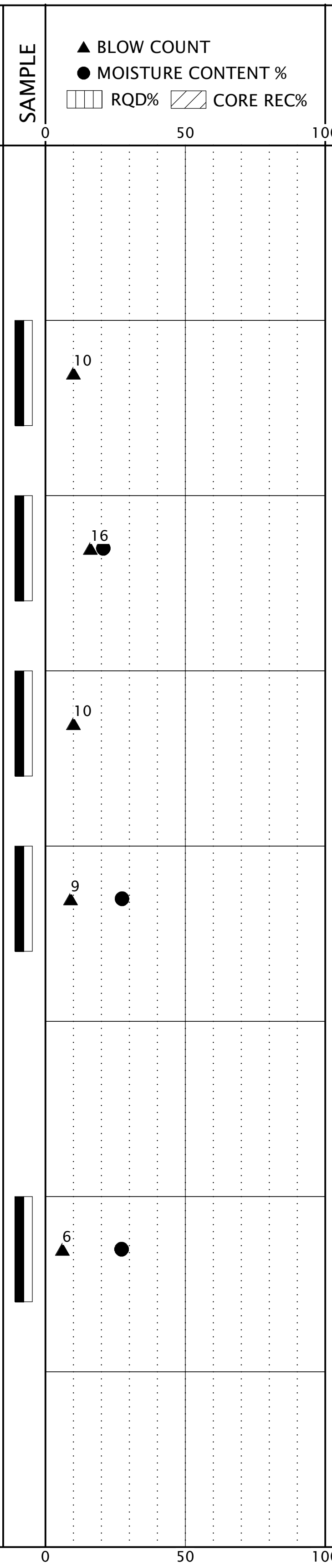
BORING B-1

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH
0.0	Stiff, brown SILT (ML); moist (topsoil to 6 inches, 3-inch-thick root zone) - **FILL**.	323.0
	dark brown at 4.0 feet	
	very stiff, trace organics (charcoal) and gravel at 4.5 feet	
	Stiff, brown SILT (ML); moist.	316.0 7.0
	with dark brown mottles at 10.0 feet	
	medium stiff at 15.0 feet	
	Exploration completed at a depth of 16.5 feet.	306.5 16.5



INSTALLATION AND COMMENTS

DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: CMC

COMPLETED: 11/10/14

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

BORING BIT DIAMETER: 4-inch

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BORING B-2

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-2

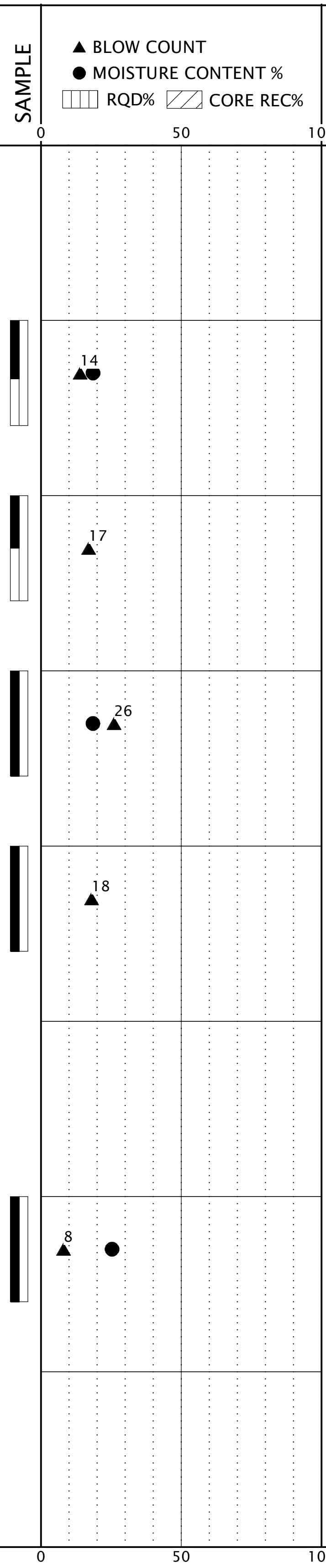
BORING LOG BEAVSCHOOL-45-01-B1_14-TP1_7.GPJ GEODESIGN.GDT PRINT DATE: 12/4/14:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	▲ BLOW COUNT
0.0	Stiff, gray-brown SILT (ML); moist (topsoil to 6 inches, 3-inch-thick root zone) - **FILL**.	320.0	
2.5			14
5.0	very stiff, with sand; sand is fine to coarse and subangular at 5.0 feet		17
7.5	brown with dark gray patches at 7.5 feet		26
9.5	Very stiff, brown SILT (ML); moist.	310.5 9.5	
10.0			18
12.5			
15.0	with dark brown mottles at 15.0 feet		8
16.5	Exploration completed at a depth of 16.5 feet.	303.5 16.5	
17.5			
20.0			

TESTING

SAMPLE

INSTALLATION AND COMMENTS



DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: CMC

COMPLETED: 11/10/14

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

BORING BIT DIAMETER: 4-inch

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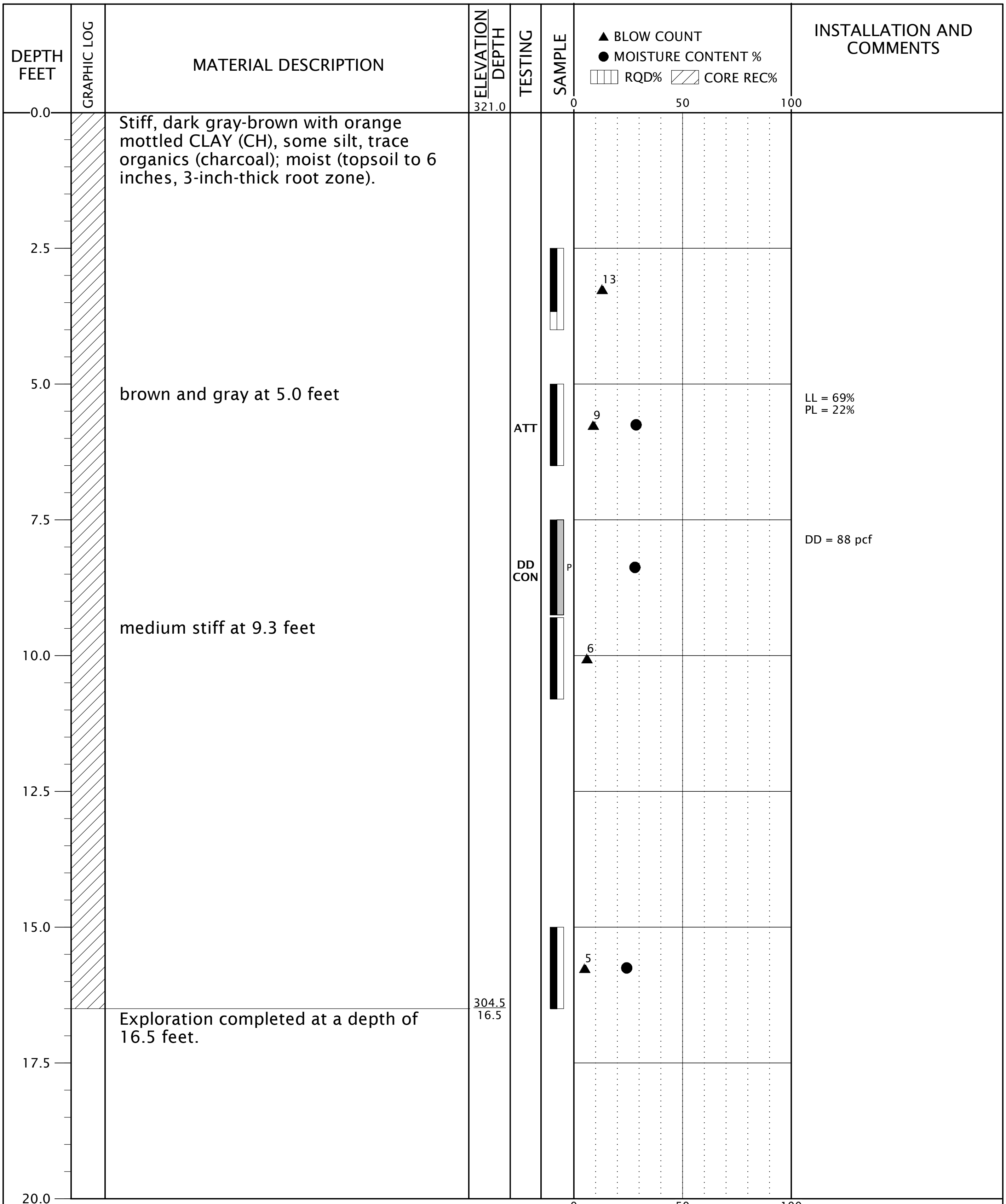
BORING B-3

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-3

BORING LOG BEAVSCHOOL-45-01-B1_14-TP1_7.GPJ GEODESIGN.GDT PRINT DATE: 12/4/14:KT



Stiff, dark gray-brown with orange mottled CLAY (CH), some silt, trace organics (charcoal); moist (topsoil to 6 inches, 3-inch-thick root zone).

brown and gray at 5.0 feet

medium stiff at 9.3 feet

Exploration completed at a depth of 16.5 feet.

LL = 69%
PL = 22%

DD = 88 pcf

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LOGGED BY: CMC
COMPLETED: 11/10/14

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

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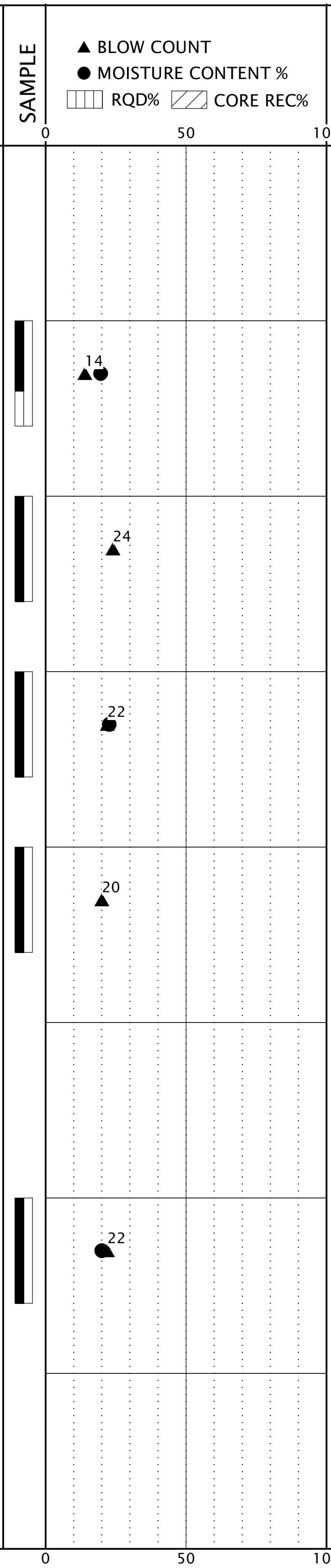
BORING B-4

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-4

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0		Stiff, gray-brown and dark brown SILT (ML), trace sand and organics; moist (topsoil to 6 inches, 2-inch-thick root zone) - **FILL**.	316.0		
2.5					
5.0		very stiff at 5.0 feet			
7.5					
10.0		dark gray-brown, trace straw at 10.0 feet			
12.5					
15.0		brown and gray, trace bark and gravel at 15.0 feet			
17.5					
		Very stiff, brown with gray mottled SILT (ML); moist.	297.5 / 18.5		
20.0					



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: CMC
COMPLETED: 11/10/14

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

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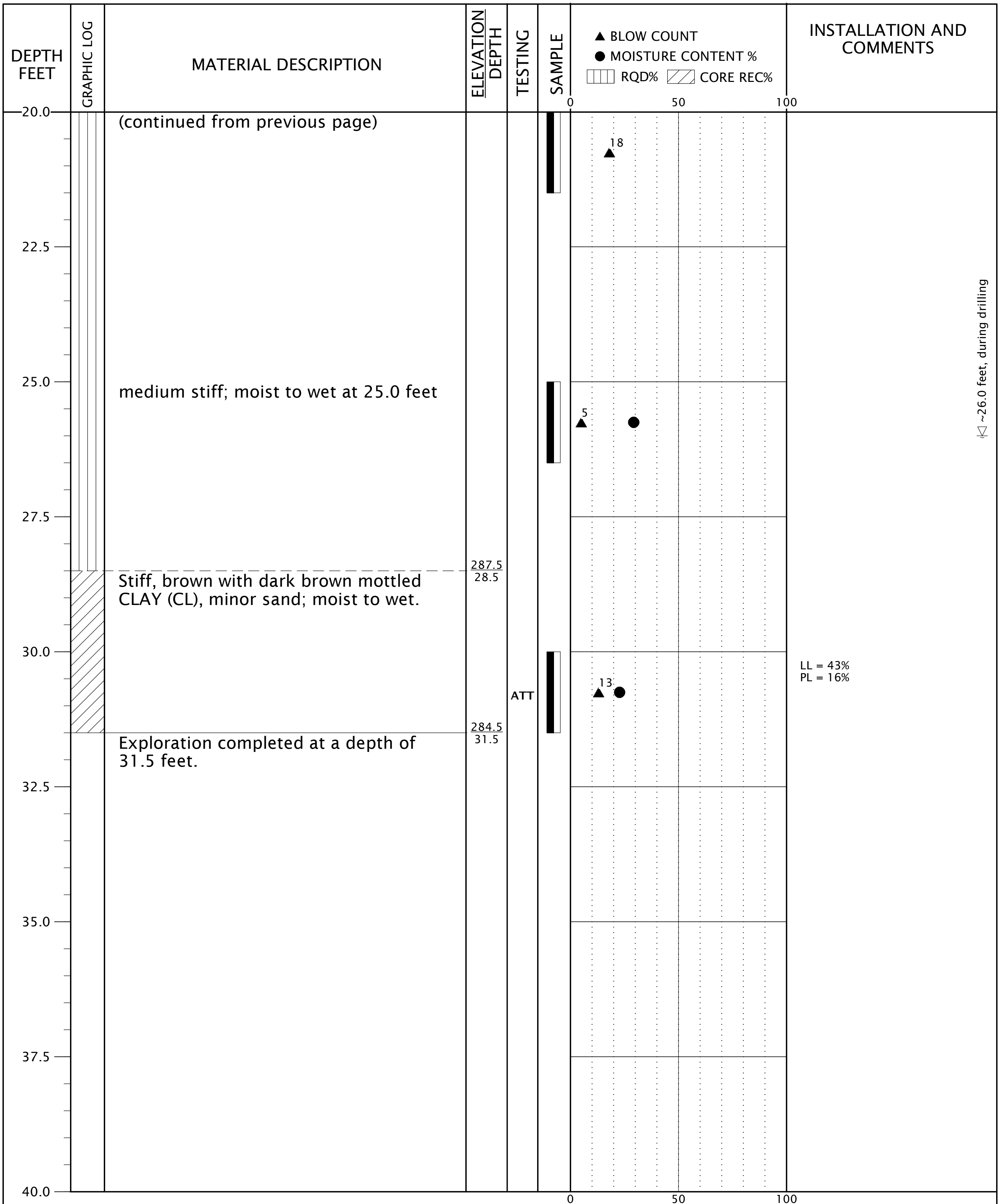
BORING B-5

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-5

BORING LOG BEAVSCHOOL-45-01-B1_14-TP1_7.GPJ GEODESIGN.GDT PRINT DATE: 12/4/14:KT



(continued from previous page)

medium stiff; moist to wet at 25.0 feet

Stiff, brown with dark brown mottled CLAY (CL), minor sand; moist to wet.

Exploration completed at a depth of 31.5 feet.

▽ ~26.0 feet, during drilling

LL = 43%
PL = 16%

DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: CMC

COMPLETED: 11/10/14

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

BORING BIT DIAMETER: 4-inch

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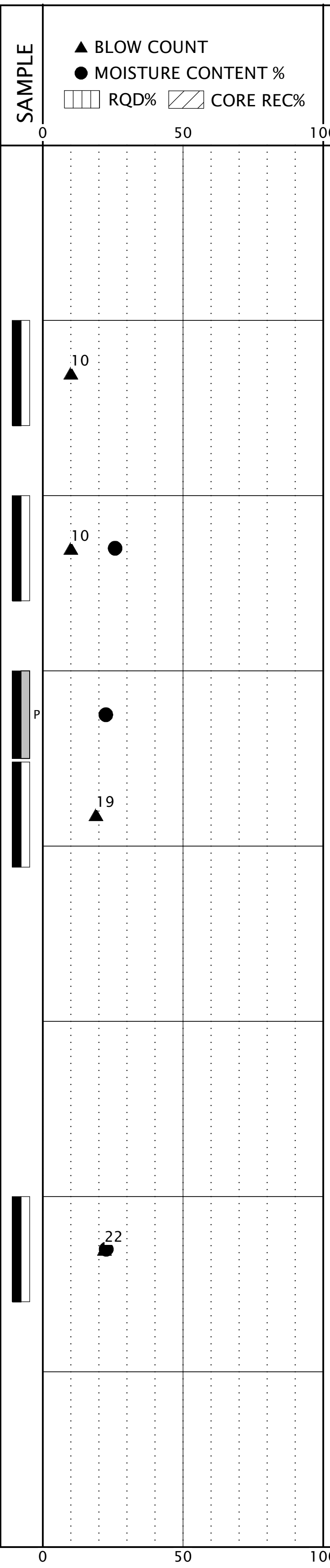
BORING B-5
(continued)

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-5

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0		Stiff, brown SILT (ML); moist (topsoil to 6 inches, 3-inch-thick root zone) - **FILL**.	318.0		
				DD	DD = 99 pcf
		Very stiff, gray-brown SILT (ML), trace sand and clay; moist - **FILL**.	309.5 8.5		
15.0		trace wood fragments and gravel at 15.0 feet			
		Stiff, brown with gray mottled SILT (ML); moist to wet.	301.5 16.5		
20.0					



DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: CMC

COMPLETED: 11/10/14

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

BORING BIT DIAMETER: 4-inch

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BORING B-6

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-6

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% 0 50 100	INSTALLATION AND COMMENTS
20.0	▥	(continued from previous page)			▮	▲ 10	
		Exploration completed at a depth of 21.5 feet.	296.5 21.5				
22.5							
25.0							
27.5							
30.0							
32.5							
35.0							
37.5							
40.0						0 50 100	

DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: CMC

COMPLETED: 11/10/14

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

BORING BIT DIAMETER: 4-inch

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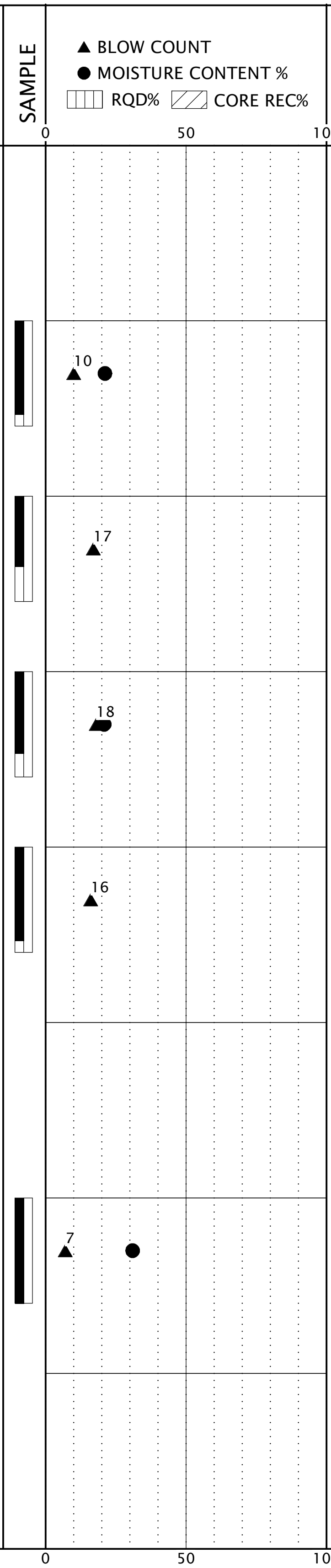
BORING B-6
(continued)

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-6

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0		Stiff, dark gray-brown SILT (ML), trace organics; moist (topsoil to 6 inches, 3-inch-thick root zone) - **FILL**.	317.0		
2.5					
5.0		very stiff at 5.0 feet			
7.5		brown with orange mottles at 7.5 feet			
10.0		with sand; sand is fine to coarse and subangular at 10.0 feet			
12.5					
		Medium stiff, brown with gray mottled SILT (ML); moist.	303.5 / 13.5		
15.0					
17.5					
20.0					



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: CMC
COMPLETED: 11/10/14

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

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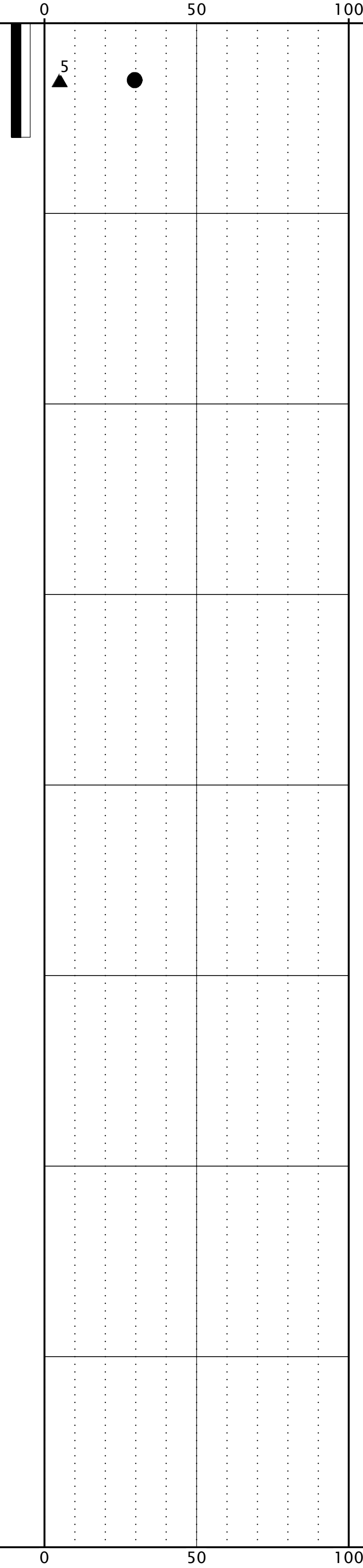
BORING B-7

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-7

BORING LOG BEAVSCHOOL-45-01-B1_14-TP1_7.GPJ GEODESIGN.GDT PRINT DATE: 12/4/14:KT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC%	INSTALLATION AND COMMENTS
20.0		wet at 20.0 feet					
		Exploration completed at a depth of 21.5 feet.	295.5 21.5				
22.5							
25.0							
27.5							
30.0							
32.5							
35.0							
37.5							
40.0							



DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: CMC

COMPLETED: 11/10/14

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

BORING BIT DIAMETER: 4-inch

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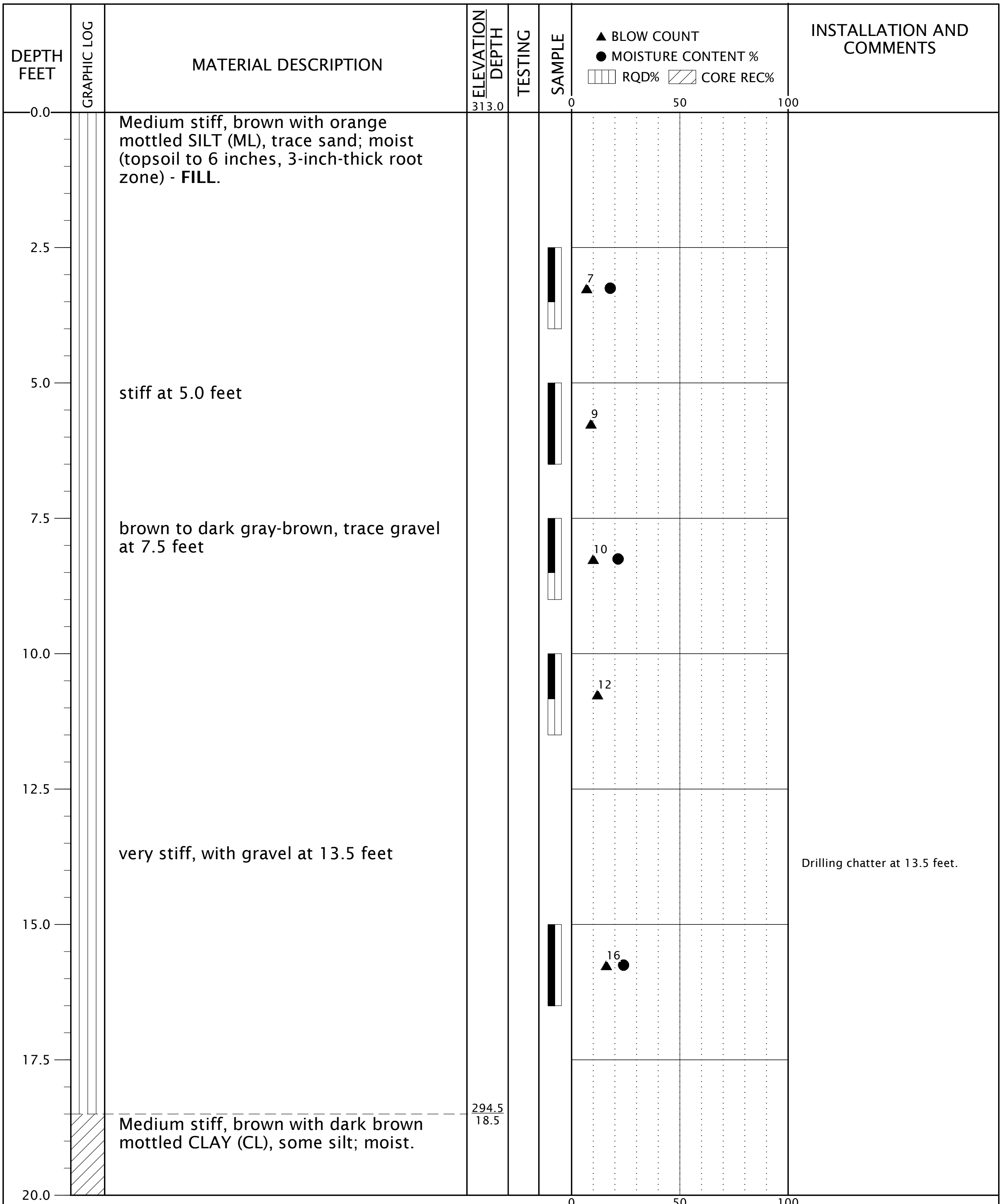
DECEMBER 2014

BORING B-7
(continued)

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-7

BORING LOG BEAVSCHOOL-45-01-B1_14-TP1_7.GPJ GEODESIGN.GDT PRINT DATE: 12/4/14:KT



Medium stiff, brown with orange mottled SILT (ML), trace sand; moist (topsoil to 6 inches, 3-inch-thick root zone) - **FILL**.

stiff at 5.0 feet

brown to dark gray-brown, trace gravel at 7.5 feet

very stiff, with gravel at 13.5 feet

Drilling chatter at 13.5 feet.

Medium stiff, brown with dark brown mottled CLAY (CL), some silt; moist.

DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: CMC
COMPLETED: 11/10/14

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

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BEAVSCHOOL-45-01

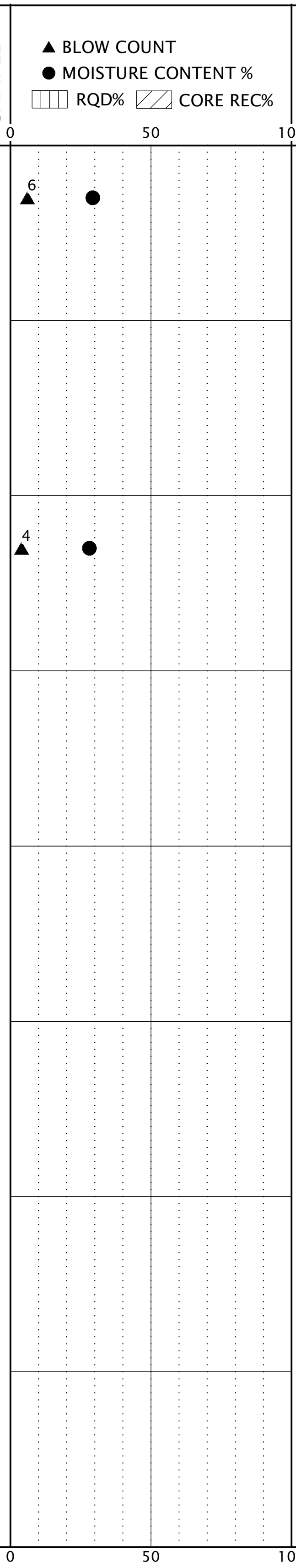
DECEMBER 2014

BORING B-8

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-8

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	INSTALLATION AND COMMENTS
20.0	▨	(continued from previous page)		ATT		LL = 39% PL = 24%
22.5	▨					
25.0	▨	wet at 25.0 feet				▽ ~25.0 feet, during drilling
26.5		Exploration completed at a depth of 26.5 feet.	286.5 26.5			
27.5						
30.0						
32.5						
35.0						
37.5						
40.0						



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: CMC
COMPLETED: 11/10/14

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

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BEAVSCHOOL-45-01

DECEMBER 2014

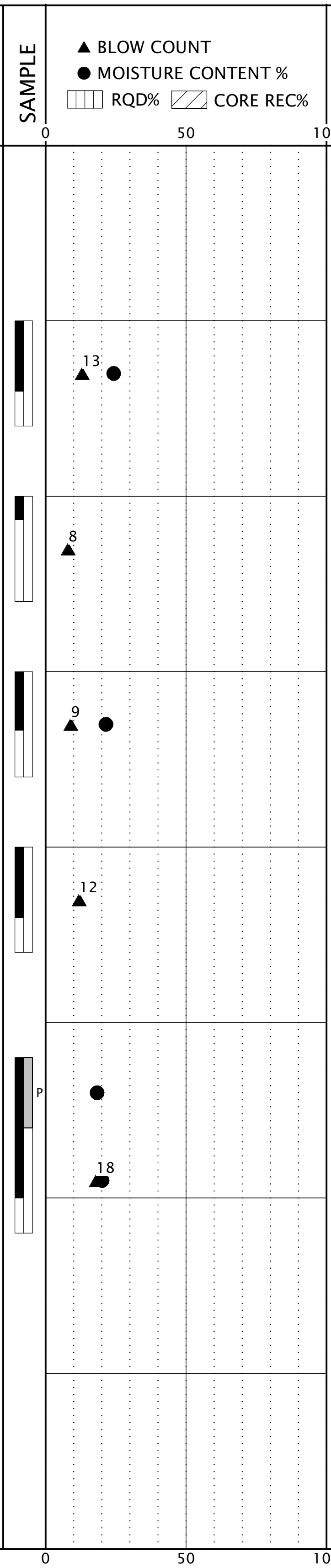
BORING B-8
(continued)

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-8

BORING LOG BEAVSCHOOL-45-01-B1_14-TP1_7.GPJ GEODESIGN.GDT PRINT DATE: 12/4/14:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Stiff, brown CLAY (CL), some silt, trace organics (woody debris); moist, organics are <1/4-inch diameter (3-inch-thick root zone) - **FILL**.	314.0		
2.5			ATT	LL = 44% PL = 23%
5.0	trace sand; sand is fine at 5.0 feet			
7.5				
10.0	trace gravel at 10.0 feet			
12.5			DD	DD = 103 pcf
14.0	very stiff, trace debris (isolated geotextile fabric/roots) at 14.0 feet			
15.0				
17.5				
20.0				



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: JGH
COMPLETED: 11/11/14

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

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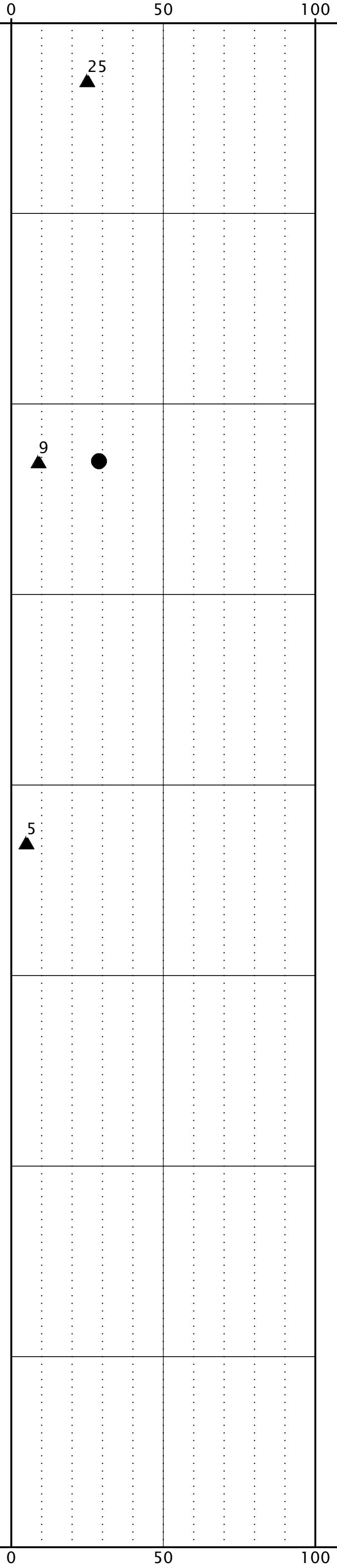
BORING B-9

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-9

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC%	INSTALLATION AND COMMENTS
20.0	▨	dark brown to gray, with gravel, minor sand, without debris; sand is fine at 20.0 feet				25	
22.5	▥	Stiff, light brown SILT (ML), minor sand; moist, sand is fine.	292.0 22.0				
25.0	▥	moist to wet at 26.0 feet				9	
27.5	▥						
30.0	▨	Medium stiff, light brown CLAY (CL/CH), some silt, trace sand; moist.	284.0 30.0			5	
		Exploration completed at a depth of 31.5 feet.	282.5 31.5				
32.5							
35.0							
37.5							
40.0							



BORING LOG BEAVSCHOOL-45-01-B1_14-TP1_7.GPJ GEODESIGN.GDT PRINT DATE: 12/4/14:KT

DRILLED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: JGH COMPLETED: 11/11/14

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

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BEAVSCHOOL-45-01

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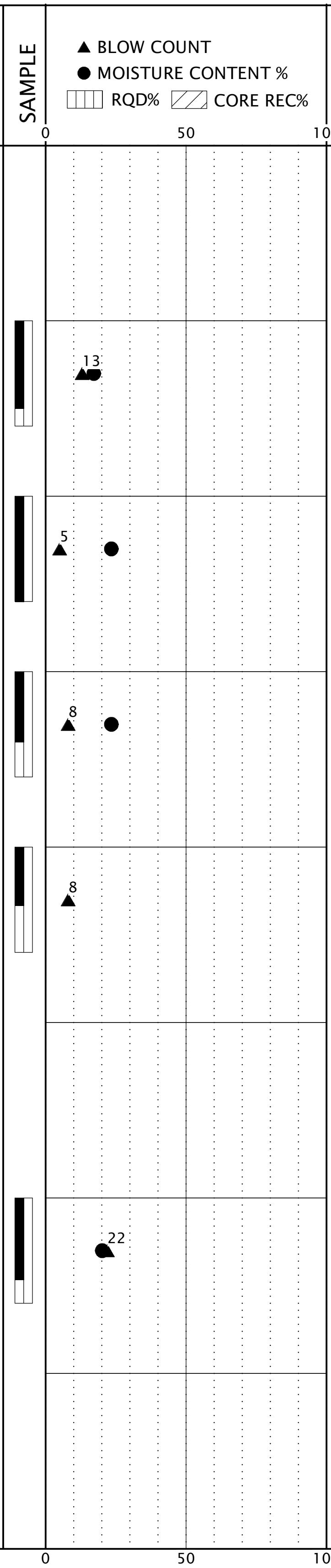
BORING B-9
(continued)

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-9

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0		Stiff, light brown SILT (ML), minor sand, trace clay; moist (topsoil to 8 inches, 2-inch-thick root zone) - **FILL**.	313.0		
2.5					
5.0		medium stiff, light brown-orange at 5.0 feet			
7.5		medium stiff to stiff at 7.5 feet			
10.0		some clay at 10.0 feet			
12.5					
15.0		very stiff, trace organics (woody fragments) at 15.0 feet			
17.5					
20.0					



DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: JGH

COMPLETED: 11/11/14

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

BORING BIT DIAMETER: 4-inch

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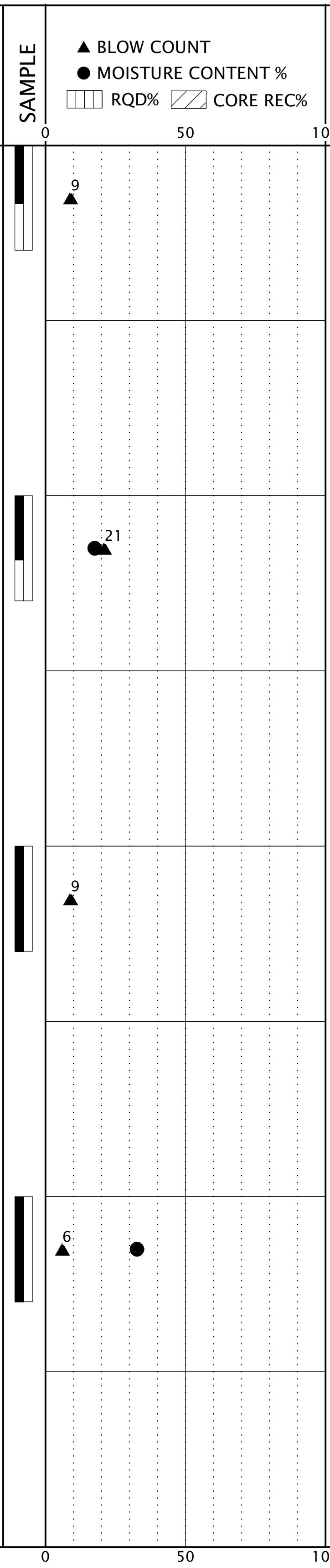
BORING B-10

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-10

BORING LOG BEAVSCHOOL-45-01-B1_14-TP1_7.GPJ GEODESIGN.GDT PRINT DATE: 12/4/14:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	INSTALLATION AND COMMENTS
20.0	stiff, without organics at 20.0 feet			
25.0	very stiff, dark brown, trace organics (roots/woody debris), without clay; organic are 1/8-inch diameter at 25.0 feet			
30.0	Stiff, light brown SILT (ML), trace sand and organics (rootlets); moist, sand is fine.	283.0 30.0		
35.0	medium stiff, minor sand, without organics at 35.0 feet			
36.5	Exploration completed at a depth of 36.5 feet.	276.5 36.5		



DEPTH FEET: 20.0, 22.5, 25.0, 27.5, 30.0, 32.5, 35.0, 37.5, 40.0

DRILLED BY: Dan J. Fischer Excavating, Inc.   LOGGED BY: JGH   COMPLETED: 11/11/14

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

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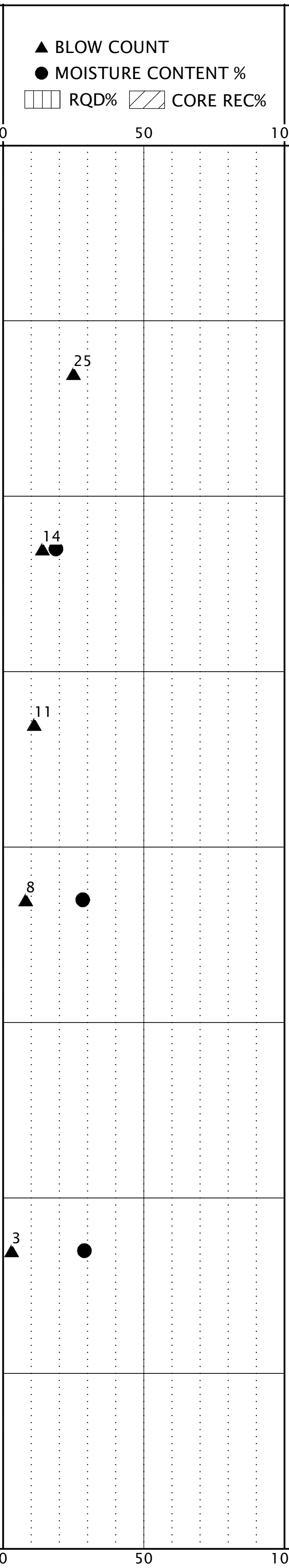
DECEMBER 2014

BORING B-10
(continued)

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-10

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE
0.0		Very stiff, brown SILT (ML), some clay, trace sand and organics (woody debris); moist (topsoil to 5 inches, 2-inch-thick root zone) - **FILL**.	295.0		
2.5					
5.0		stiff, gray-dark gray, minor sand and gravel at 5.0 feet			
7.5					
		Stiff, light brown SILT (ML), minor sand; moist, sand is fine.	287.0 8.0		
10.0		medium stiff to stiff at 10.0 feet			
12.5					
15.0		soft at 15.0 feet			
		Exploration completed at a depth of 16.5 feet.	278.5 16.5		
17.5					
20.0					



INSTALLATION AND COMMENTS

DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: JGH

COMPLETED: 11/11/14

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

BORING BIT DIAMETER: 4-inch

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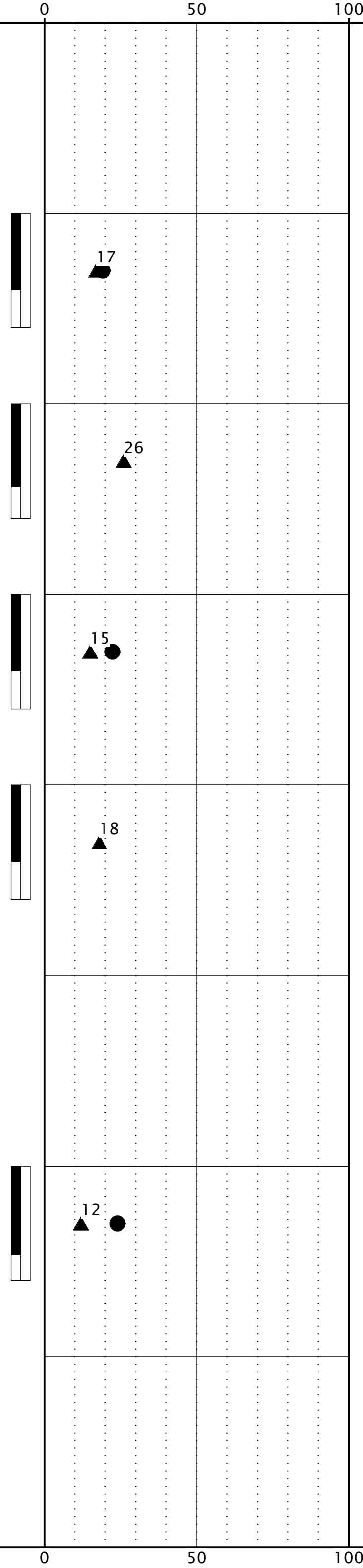
BORING B-11

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-11

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC%	INSTALLATION AND COMMENTS
0.0		Very stiff, brown-light brown SILT (ML), some clay, trace sand and organics (rootlets); moist, sand is fine (3-inch-thick root zone) - **FILL**.	299.0				
2.5							
5.0							
7.5		brown-gray at 7.5 feet					
10.0		brown, minor sand at 10.0 feet					
12.5							
15.0		stiff, light brown to gray, trace sand at 15.0 feet					
17.5							
		Medium stiff, light brown SILT (ML), trace sand; moist to wet, sand is fine.	281.0 18.0				
20.0							



DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: JGH

COMPLETED: 11/11/14

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

BORING BIT DIAMETER: 4-inch

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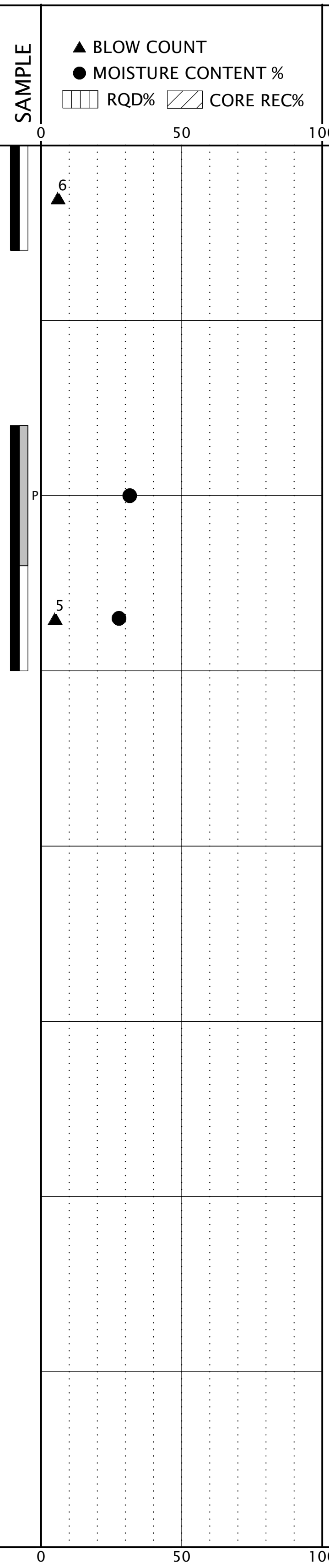
BORING B-12

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-12

BORING LOG BEAVSCHOOL-45-01-B1_14-TP1_7.GPJ GEODESIGN.GDT PRINT DATE: 12/4/14:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
20.0	(continued from previous page)			
22.5				
25.0			DD CON	DD = 89 pcf
27.5	Exploration completed at a depth of 27.5 feet.	271.5 27.5		
30.0				
32.5				
35.0				
37.5				
40.0				



DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: JGH

COMPLETED: 11/11/14

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

BORING BIT DIAMETER: 4-inch

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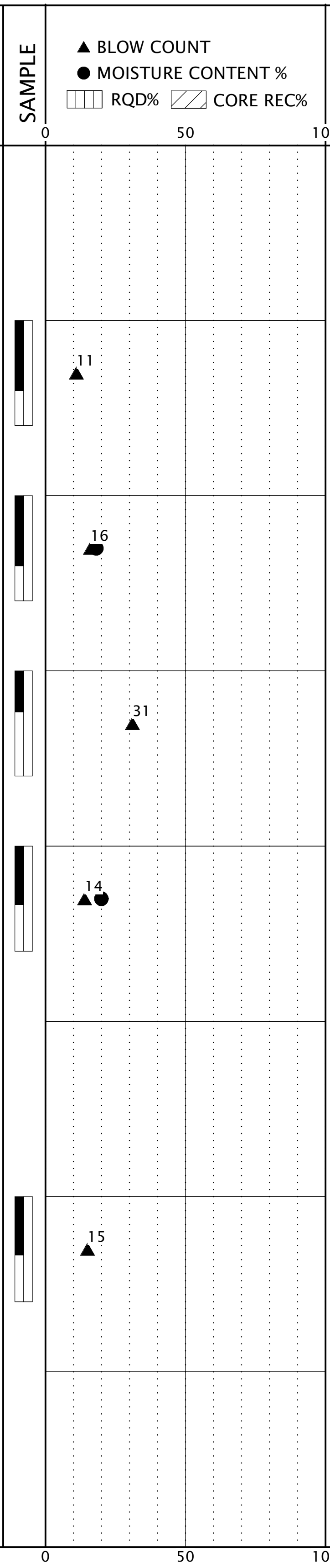
BORING B-12
(continued)

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-12

BORING LOG BEAVSCHOOL-45-01-B1_14-TP1_7.GPJ GEODESIGN.GDT PRINT DATE: 12/4/14:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING
0.0	Stiff, light brown to dark brown SILT (ML), some clay, trace sand and gravel; moist (3-inch-thick root zone) - **FILL**.	302.0	
5.0	very stiff, trace organics (woody fragments) at 5.0 feet		
7.5	hard, brown-gray at 7.5 feet		
10.0	stiff, trace debris (isolated plastic fragments), without gravel at 10.0 feet		
15.0	trace gravel at 15.0 feet		
20.0			



INSTALLATION AND COMMENTS

DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: JGH

COMPLETED: 11/11/14

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

BORING BIT DIAMETER: 4-inch

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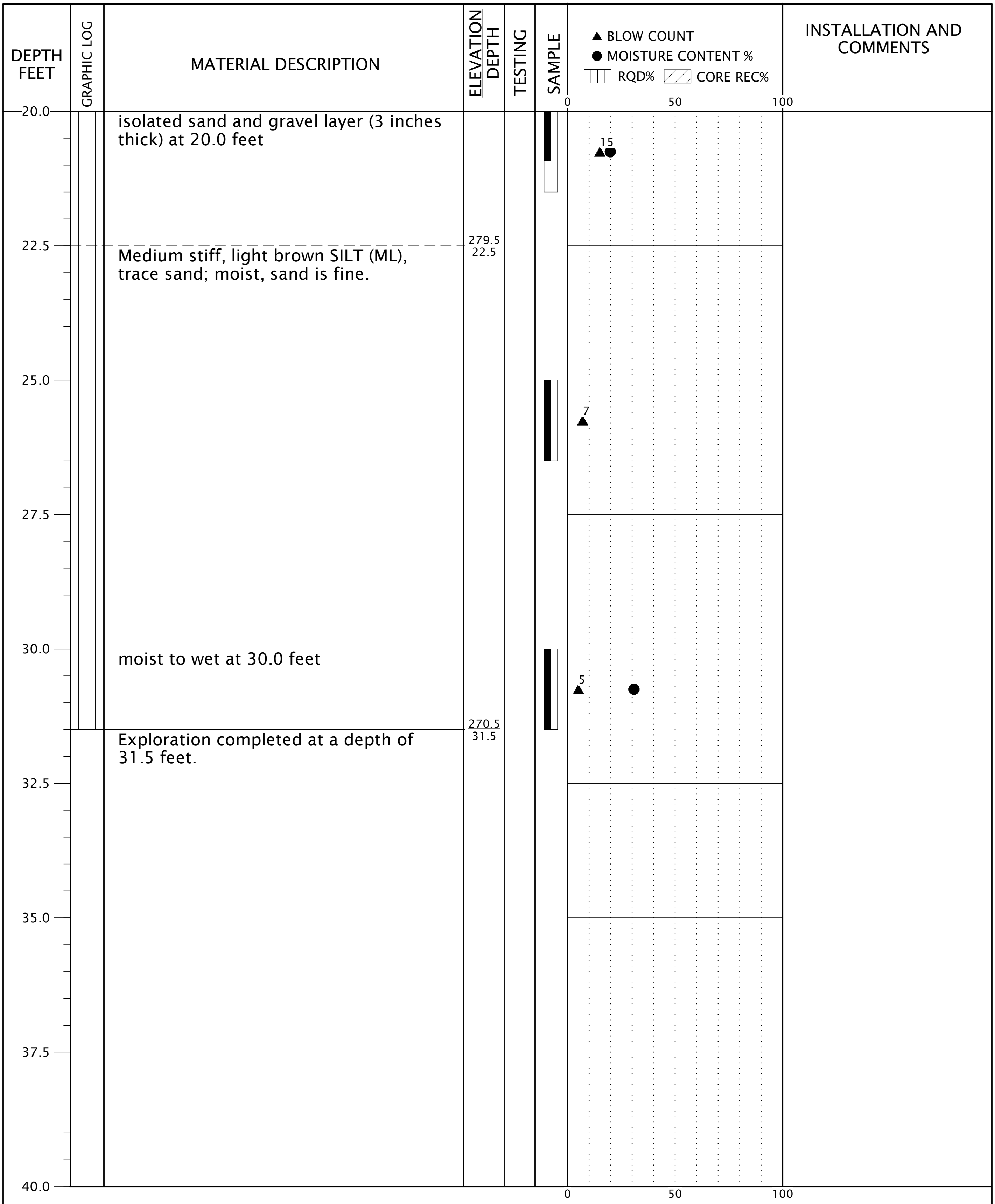
BORING B-13

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-13



isolated sand and gravel layer (3 inches thick) at 20.0 feet

Medium stiff, light brown SILT (ML), trace sand; moist, sand is fine.

moist to wet at 30.0 feet

Exploration completed at a depth of 31.5 feet.

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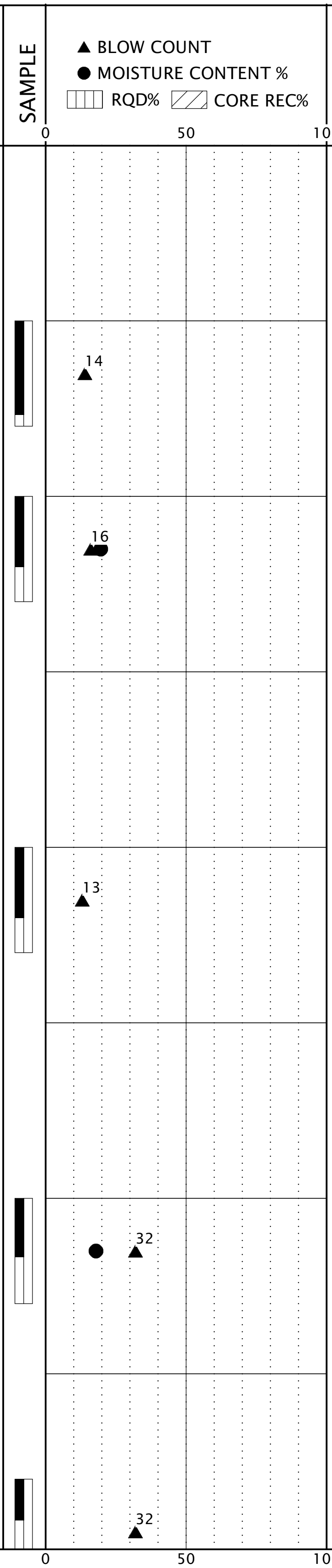
BORING METHOD: solid-stem auger (see document text) BORING BIT DIAMETER: 4-inch

GEODESIGN INC 15575 SW Sequoia Parkway - Suite 100 Portland OR 97224 Off 503.968.8787 Fax 503.968.3068	BEAVSCHOOL-45-01	**BORING B-13** (continued)
	DECEMBER 2014	BEAVERTON MIDDLE SCHOOL AT TIMBERLAND BEAVERTON, OR — **FIGURE A-13**

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
0.0	Stiff, light brown-brown SILT (ML), some clay, trace sand and organics (rootlets); moist, sand is fine, organics are <1/2-inch diameter (3-inch-thick root zone) - **FILL**.	309.0	
5.0	very stiff, light brown to gray, trace organics (woody debris); organics are <1/8-inch diameter at 5.0 feet		
10.0	stiff at 10.0 feet		
15.0	hard at 15.0 feet		
19.0	minor sand at 19.0 feet		
20.0			



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: JGH
COMPLETED: 11/11/14

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

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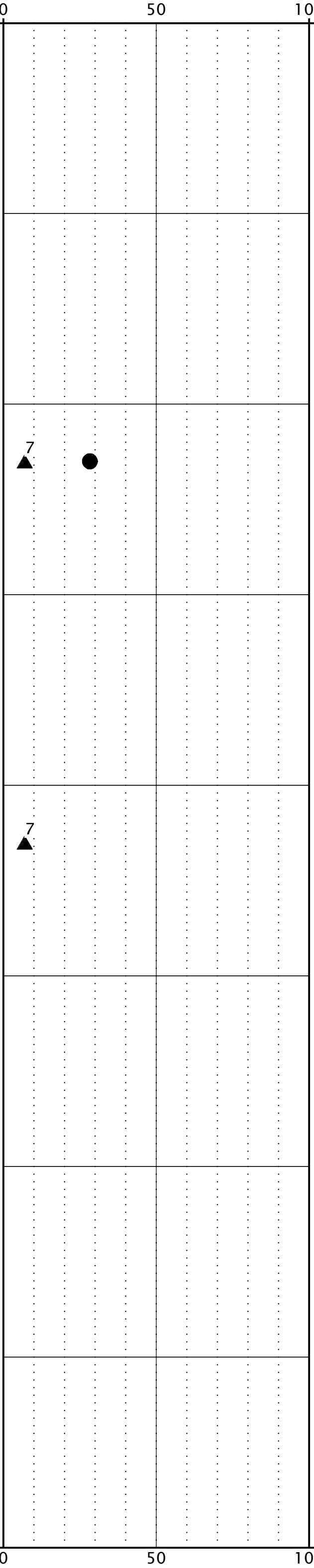
BORING B-14

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-14

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC%	INSTALLATION AND COMMENTS
20.0		(continued from previous page)					
22.5		Medium stiff, brown SILT (ML), trace sand; moist, sand is fine.	286.5 22.5				
25.0						▲ 7 ●	
30.0		some clay at 30.0 feet				▲ 7	
31.5		Exploration completed at a depth of 31.5 feet.	277.5 31.5				
32.5							
35.0							
37.5							
40.0							



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: JGH
COMPLETED: 11/11/14

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

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BORING B-14
(continued)

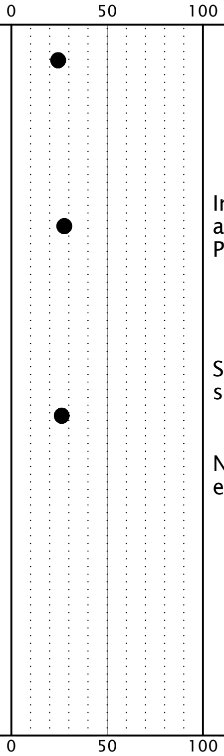
BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-14

BORING LOG BEAVSCHOOL-45-01-B1_14-TP1_7.GPJ GEODESIGN.GDT PRINT DATE: 12/4/14:KT

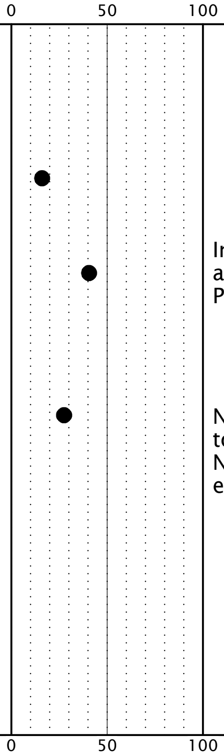
TP-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium stiff, dark brown SILT with gravel (ML), minor sand and organics (rootlets); moist, sand is fine (3-inch-thick root zone) - **FILL**.	326.0		☒	
	Medium stiff, light brown SILT (ML), minor sand; moist, sand is fine.	324.8 1.2		☒	
			P200	☒	Infiltration test: 0.6 inch per hour at 4.0 feet. P200 = 89%
	moist to wet; blocky texture at 7.0 feet				Slow to moderate groundwater seepage observed at 7.5 feet.
				☒	
	Exploration completed at a depth of 8.5 feet.	317.5 8.5			No caving observed to the depth explored.



TP-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium stiff to stiff, brown to gray SILT (ML), some clay, minor gravel, sand, and organics (roots/woody debris); moist, organics are up to 1/2-inch diameter (3-inch-thick root zone) - **FILL**.	294.0		☒	
	isolated cobble (5-inch diameter) at 3.0 feet			☒	
	Medium stiff, light brown SILT (ML), minor sand; moist, sand is fine.	290.5 3.5		☒	
			P200	☒	Infiltration test: 0.4 inch per hour at 5.0 feet. P200 = 82%
				☒	No groundwater seepage observed to the depth explored.
	Exploration completed at a depth of 8.5 feet.	285.5 8.5			No caving observed to the depth explored.



EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: JGH

COMPLETED: 11/12/14

EXCAVATION METHOD: trackhoe (see document text)

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TEST PIT

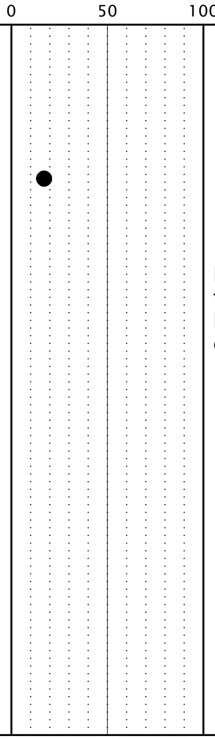
BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-15

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

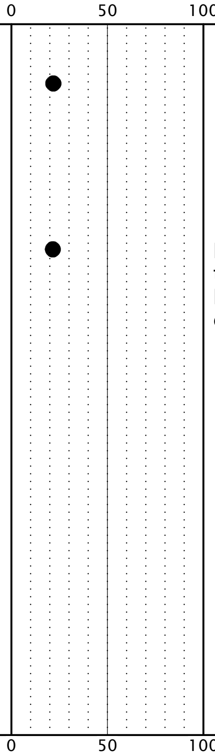
TP-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	SAMPLE	COMMENTS
0.0	Stiff, brown to gray SILT (ML), some clay, minor gravel and sand; moist (2-inch-thick root zone) - **FILL**. with cobbles, trace debris; cobbles are up to 9-inch diameter - total of 5 to 8 observed, debris is concrete pieces up to 4-inch diameter, 4-inch-long flat piece of plastic, and 6-inch by 2 1/4-inch piece of wood at 1.0 foot	314.0	☒ ☒ ☒	
5.5	Exploration completed at a depth of 5.5 feet.	308.5 5.5		No groundwater seepage observed to the depth explored. No caving observed to the depth explored.



TP-4

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	SAMPLE	COMMENTS
0.0	Medium stiff to stiff, brown to gray SILT (ML), some clay, trace gravel, sand, and organics (woody debris); moist, organics are up to 3/8-inch diameter (3-inch-thick root zone) - **FILL**. piece of plastic debris (3 inches long) at 1.5 feet stiff at 3.5 feet	319.0	☒ ☒ ☒	
5.0	Exploration completed at a depth of 5.0 feet.	314.0 5.0		No groundwater seepage observed to the depth explored. No caving observed to the depth explored.



EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: JGH COMPLETED: 11/12/14

EXCAVATION METHOD: trackhoe (see document text)

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TEST PIT

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-16

TP-5

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT (%) 0 50 100	COMMENTS
0.0		Medium stiff to stiff, light brown-brown to gray SILT (ML), some clay, trace gravel, sand, and organics (rootlets/woody debris); moist (3-inch-thick root zone) - **FILL**. stiff, with cobbles (up to 4-inch diameter) at 1.5 feet	315.0		☒		
5.0					☒	●	
5.5		Exploration completed at a depth of 5.5 feet.	309.5 5.5				No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
7.5							
10.0							
12.5							

TP-6

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT (%) 0 50 100	COMMENTS
0.0		Medium stiff to stiff, gray to brown SILT with cobbles (ML), some clay, trace gravel, sand, and organics (woody debris); moist, cobbles are up to 10-inch diameter - total of 6 observed, organics are up to 1/2-inch diameter (3-inch-thick root zone) - **FILL**. trace debris (2-inch PVC pipe) at 2.0 feet stiff, light brown to gray, without cobbles at 2.5 feet	312.0		☒		
2.5					☒	●	
5.0					☒		
5.5		Exploration completed at a depth of 5.5 feet.	306.5 5.5				No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
7.5							
10.0							
12.5							

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: JGH COMPLETED: 11/12/14

EXCAVATION METHOD: trackhoe (see document text)

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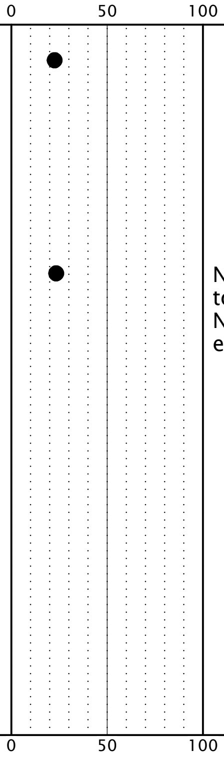
TEST PIT

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-17

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT (%)	COMMENTS
TP-7							
0.0		Medium stiff to stiff, gray to light brown SILT (ML), some clay, minor sand, trace gravel and organics (rootlets and woody debris); moist, organics are up to 1 1/2-inch diameter (2-inch-thick root zone) - **FILL**. trace debris (occasional PVC fragments, plastic bag, twine) at 1.0 foot stiff, with cobbles (isolated, 5-inch diameter) at 3.0 feet	318.0		☒ ☒ ☒		
2.5							
5.0		Exploration completed at a depth of 5.5 feet.	312.5 5.5				No groundwater seepage observed to the depth explored. No caving observed to the depth explored.
7.5							
10.0							
12.5							



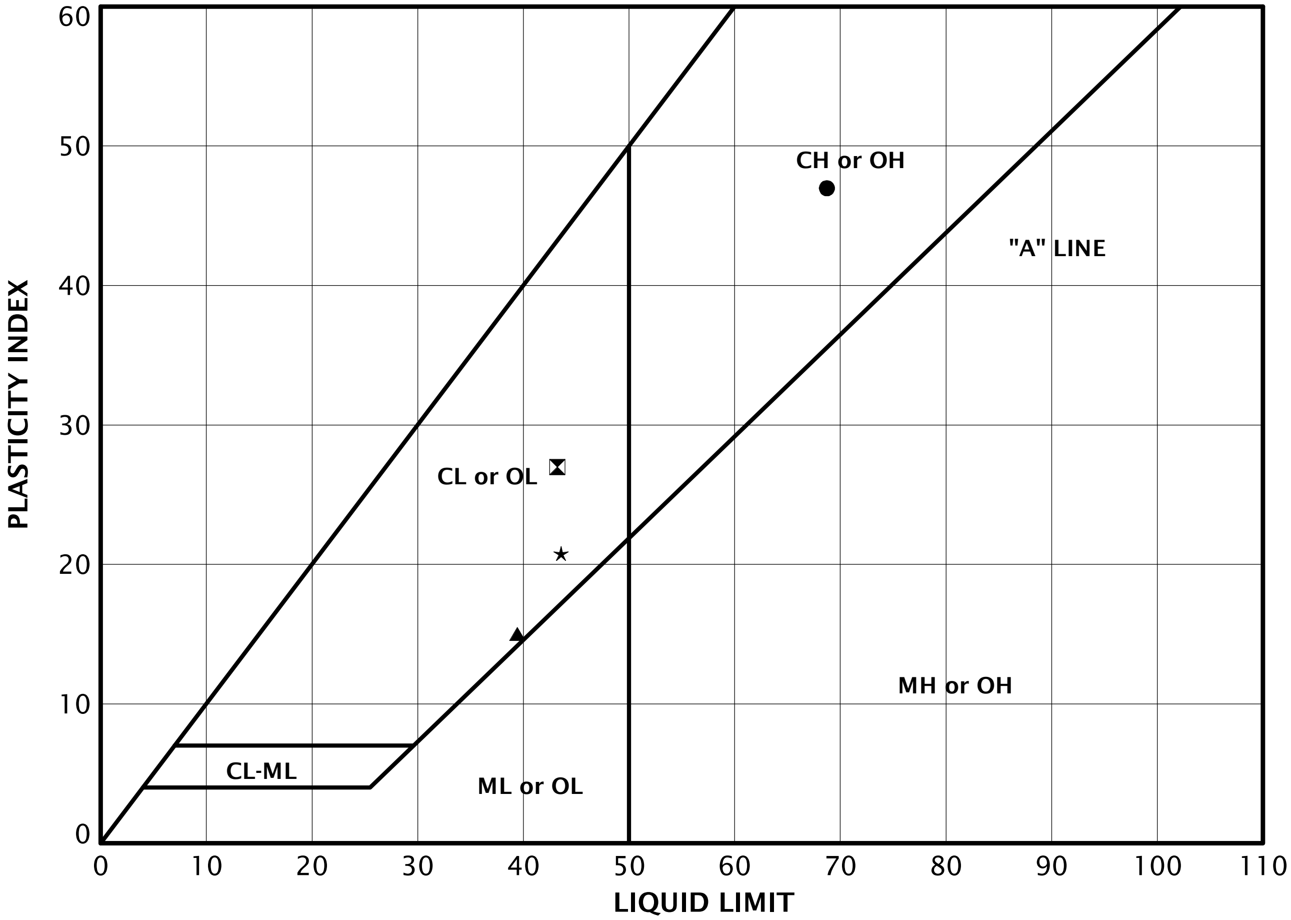
EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: JGH

COMPLETED: 11/12/14

EXCAVATION METHOD: trackhoe (see document text)

GEODESIGN INC 15575 SW Sequoia Parkway - Suite 100 Portland OR 97224 Off 503.968.8787 Fax 503.968.3068	BEAVSCHOOL-45-01	**TEST PIT**	
	DECEMBER 2014	BEAVERTON MIDDLE SCHOOL AT TIMBERLAND BEAVERTON, OR	**FIGURE A-18**



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-4	5.0	29	69	22	47
⧗	B-5	30.0	23	43	16	27
▲	B-8	20.0	29	39	24	15
★	B-9	2.5	24	44	23	21

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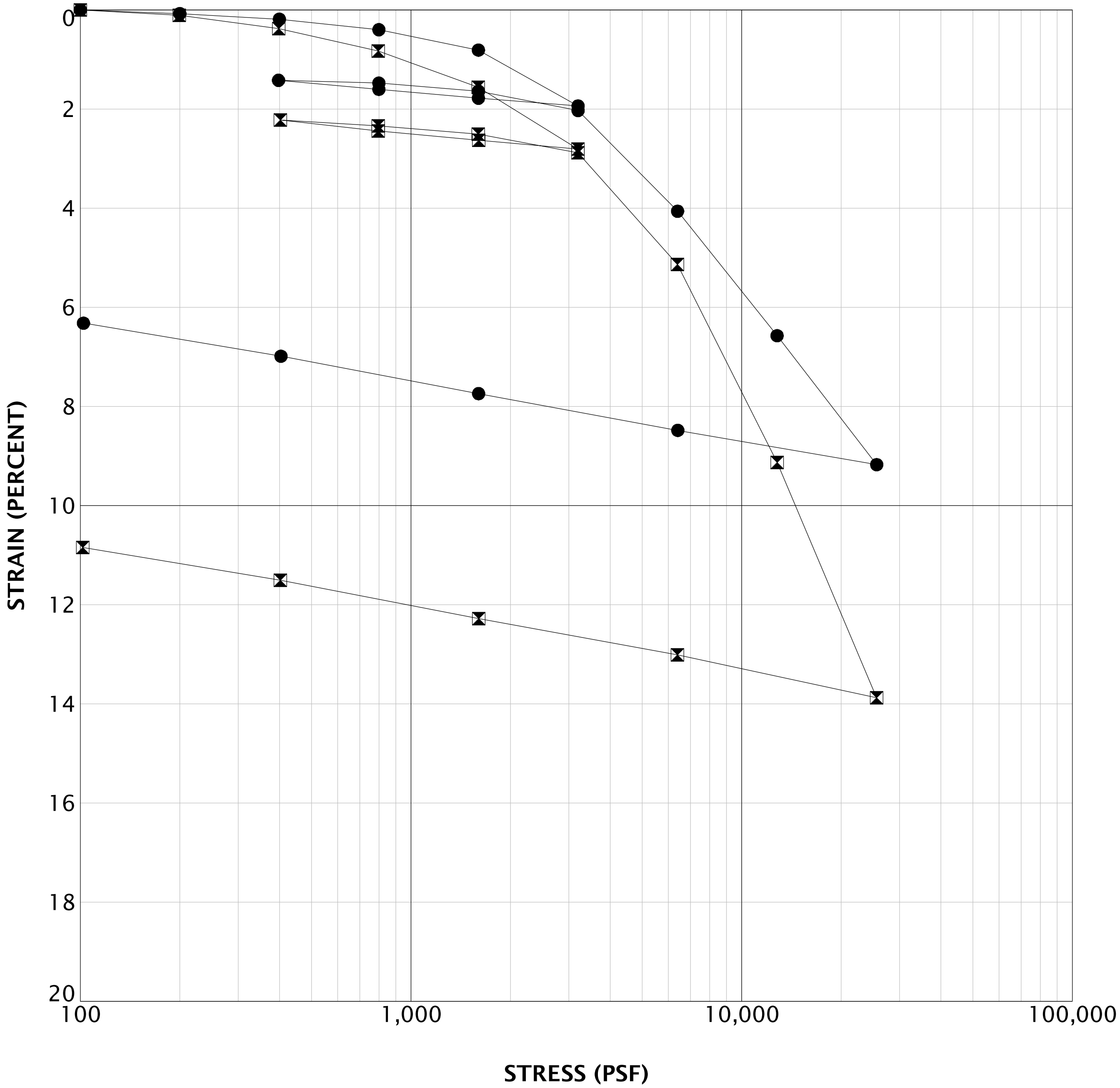
ATTERBERG LIMITS TEST RESULTS

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-19



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)
●	B-4	7.5	28	88
⧗	B-12	24.0	32	89

CONSOL_STRAIN_100K BEAVSCHOOL-45-01-B1_14-TP1_7.GPJ GEODESIGN.GDT PRINT DATE: 12/4/14:KT

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CONSOLIDATION TEST RESULTS

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-20

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	2.5	321.5	27							
B-1	7.5	316.5	23	88						
B-1	10.0	314.0	22							
B-1	15.0	309.0	23							
B-2	5.0	318.0	21							
B-2	10.0	313.0	27							
B-2	15.0	308.0	27							
B-3	2.5	317.5	19							
B-3	7.5	312.5	19							
B-3	15.0	305.0	25							
B-4	5.0	316.0	29					69	22	47
B-4	7.5	313.5	28	88						
B-4	15.0	306.0	24							
B-5	2.5	313.5	20							
B-5	7.5	308.5	23							
B-5	15.0	301.0	20							
B-5	25.0	291.0	29							
B-5	30.0	286.0	23					43	16	27
B-6	5.0	313.0	26							
B-6	7.5	310.5	22	99						
B-6	15.0	303.0	23							
B-7	2.5	314.5	21							
B-7	7.5	309.5	21							
B-7	15.0	302.0	31							
B-7	20.0	297.0	30							
B-8	2.5	310.5	18							
B-8	7.5	305.5	21							

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SUMMARY OF LABORATORY DATA

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-21

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-8	15.0	298.0	24							
B-8	20.0	293.0	29					39	24	15
B-8	25.0	288.0	28							
B-9	2.5	311.5	24					44	23	21
B-9	7.5	306.5	21							
B-9	13.0	301.0	18	103						
B-9	14.0	300.0	20							
B-9	25.0	289.0	29							
B-10	2.5	310.5	17							
B-10	5.0	308.0	23							
B-10	7.5	305.5	23							
B-10	15.0	298.0	20							
B-10	25.0	288.0	18							
B-10	35.0	278.0	33							
B-11	5.0	290.0	19							
B-11	10.0	285.0	28							
B-11	15.0	280.0	29							
B-12	2.5	296.5	19							
B-12	7.5	291.5	22							
B-12	15.0	284.0	24							
B-12	24.0	275.0	32	89						
B-12	26.0	273.0	28							
B-13	5.0	297.0	18							
B-13	10.0	292.0	20							
B-13	20.0	282.0	20							
B-13	30.0	272.0	31							
B-14	5.0	304.0	20							

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SUMMARY OF LABORATORY DATA
(continued)

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-21

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-14	15.0	294.0	18							
B-14	25.0	284.0	28							
TP-1	0.5	325.5	24							
TP-1	4.0	322.0	28				89			
TP-1	8.0	318.0	26							
TP-2	3.0	291.0	16							
TP-2	5.0	289.0	41				82			
TP-2	8.0	286.0	27							
TP-3	3.0	311.0	17							
TP-4	1.0	318.0	22							
TP-4	4.5	314.5	22							
TP-5	5.0	310.0	23							
TP-6	3.0	309.0	19							
TP-7	0.5	317.5	23							
TP-7	5.0	313.0	23							

LAB SUMMARY BEAVSCHOOL-45-01-B1_14-TP1_7.GPJ GEODESIGN.GDT PRINT DATE: 12/4/14:KT

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SUMMARY OF LABORATORY DATA
(continued)

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE A-21

APPENDIX B

APPENDIX B

CONE PENETROMETER TEST

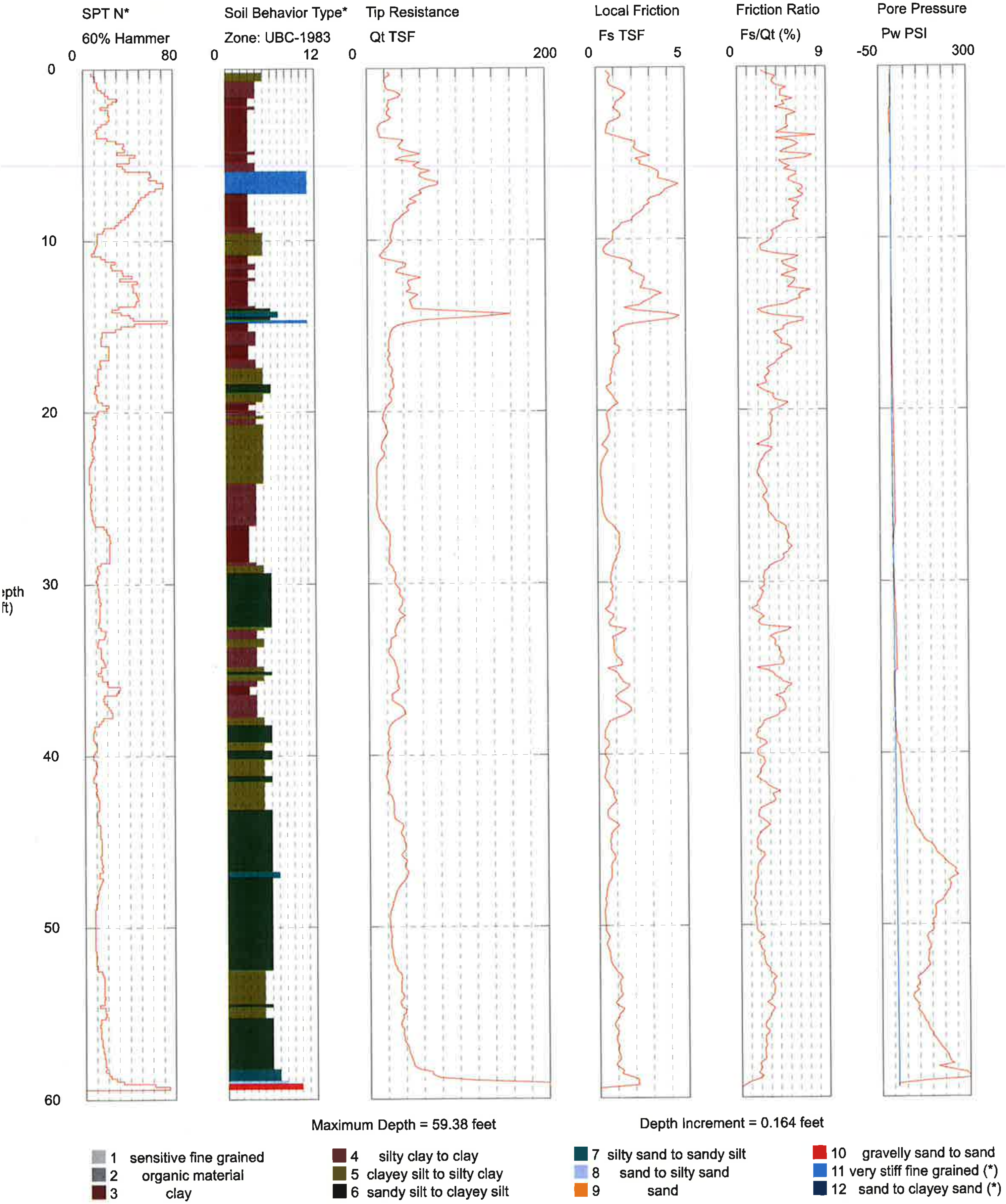
One CPT probe (CPT-1) was advanced to a depth of depth of 59.4 feet BGS. Figure 2 shows the location of the CPT relative to existing site features. The CPT was performed in general accordance with ASTM D 5778 by Oregon Geotechnical Explorations, Inc. of Keizer, Oregon, on November 6, 2014.

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. At selected depths, the CPT advancement was suspended and pore water dissipation rates were measured. Shear wave velocity of the subsurface soil was also measured at 2-meter intervals in CPT-2. This appendix presents the results of the CPT completed for this project.

GeoDesign / CPT-1 / NW 118th & NW Holly Springs Beaverto

Operator: OGE TAJ
Sounding: CPT-1
Cone Used: DSG0736

CPT Date/Time: 11/6/2014 9:48:34 AM
Location: GeoDesign / CPT-1 / NW 118th & Holly Springs Beaverton
Job Number: GeoDesign / CPT-1 / NW 118th & Holly Springs Beave



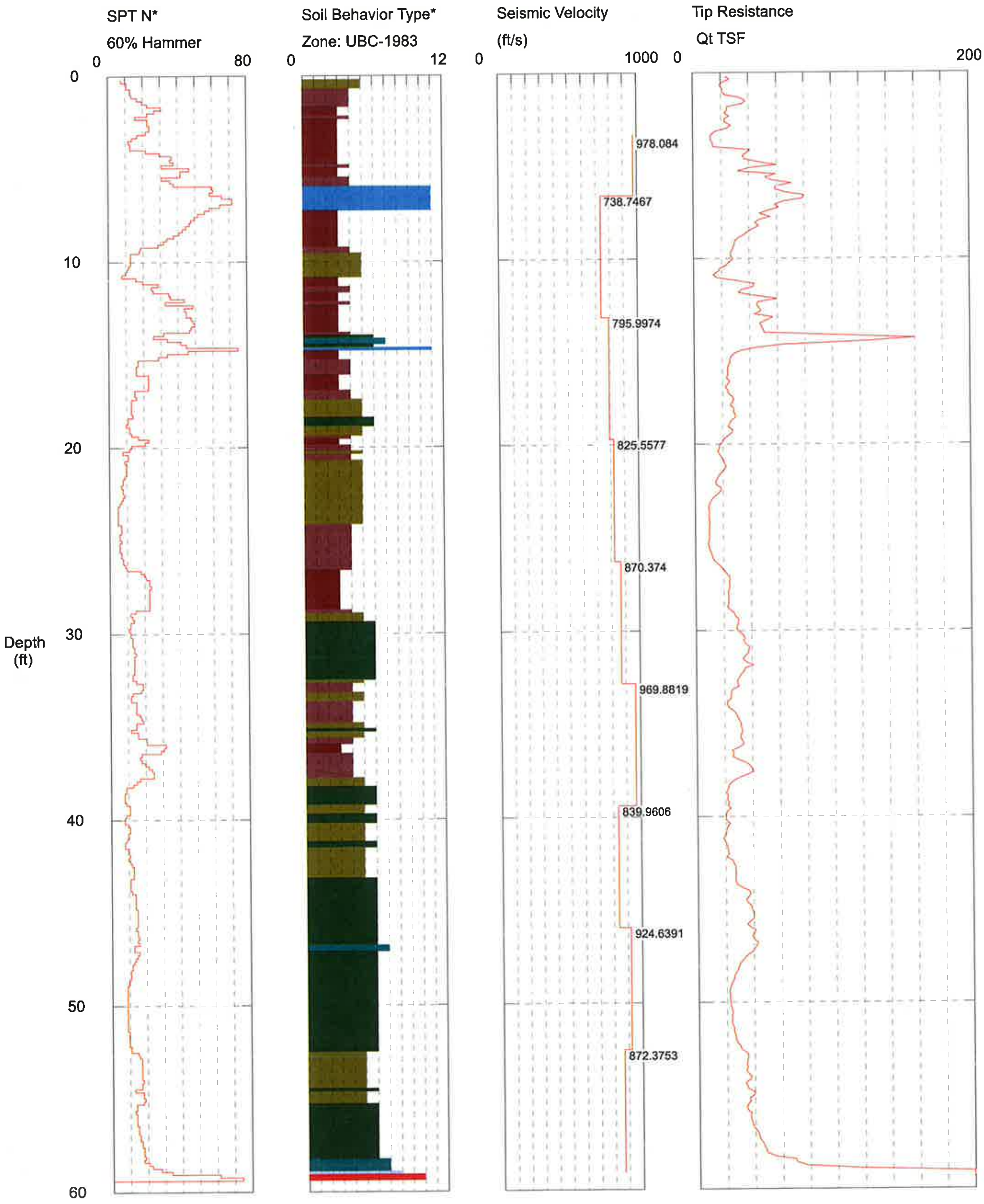
Maximum Depth = 59.38 feet

Depth Increment = 0.164 feet

GeoDesign / CPT-1 / NW 118th & NW Holly Springs Beaverto

Operator: OGE TAJ
Sounding: CPT-1
Cone Used: DSG0736

CPT Date/Time: 11/6/2014 9:48:34 AM
Location: GeoDesign / CPT-1 / NW 118th & Holly Springs Beaverton
Job Number: GeoDesign / CPT-1 / NW 118th & Holly Springs Beave



Maximum Depth = 59.38 feet

Depth Increment = 0.164 feet

■ 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 (*)

APPENDIX C

APPENDIX C

SITE-SPECIFIC SEISMIC HAZARD EVALUATION

INTRODUCTION

The information in this appendix summarizes the results of a site-specific seismic hazard study for the proposed school facility northeast of the intersection between NW 118th Avenue and NW Stone Mountain Lane in Beaverton, Oregon. The site location is shown on Figure 1 of the main report. This seismic hazard evaluation was performed in accordance with the requirements in the 2014 SOSSC and the 2012 IBC.

SITE CONDITIONS

REGIONAL GEOLOGY

The Portland-Vancouver metropolitan area is situated within the Puget-Willamette Trough physiographic province, a north-south structural basin lying between the Coast Ranges to the west and the Cascade Range to the east. The Portland Basin, a major component of the Willamette Trough, is a subsided lowland formed through northeast-directed compression due to large-scale plate movement and subduction and right-lateral extension along a series of faults reaching from central Oregon, across the Cascades, and into the lower Willamette Valley (for general discussion see Burns, 1998; Orr and Orr, 1999).

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

1. An interface event earthquake on the seismogenic part of the interface between the Juan de Fuca Plate and the North American Plate on the CSZ. This source is reportedly capable of generating earthquakes with a moment magnitude of between 8.5 and 9.0.

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

Local Events

A significant earthquake could occur on a local fault near the site within the design life of the facility. Such an event would cause ground shaking at the site that could be more intense than the CSZ events, though the duration would be shorter. Figure C-1 shows the locations of faults with potential Quaternary movement within a 20-mile radius of the site. Figure C-2 shows the interpreted locations of seismic events that occurred between 1833 and 1985 (NGDG, 2010). The most significant faults in the site vicinity are the Oatfield Fault, Beaverton Fault Zone, Portland Hills Fault, East Bank Fault, Helvetia Fault, and Canby-Molalla Fault. Based on seismic deaggregation, the Portland Hills Fault is the major contributing local mapped fault to the overall seismic hazard at the site. A discussion of the faults is provided below.

Oatfield Fault

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

Beaverton Fault Zone

The east-west-striking Beaverton Fault zone forms the southern margin of the main part of the Tualatin Basin, an isolated extension of the Willamette lowland forearc basin in northwestern Oregon. The Beaverton Fault zone is not shown on most published geologic maps of the area, but is marked by a linear aeromagnetic anomaly and has been mapped in the subsurface where it offsets Miocene CRBG rocks and overlying Pliocene to Pleistocene sediments. The late Neogene Tualatin Basin may be a pull-apart basin, with subsidence driven by dextral shear on the nearby Gales Creek Fault zone. The fault trace is buried by a thick sequence of sediment deposited by the 12.7 to 13.3 ka Missoula floods, but offsets middle Pleistocene and possibly younger sediments in the subsurface. Seismic and well data clearly indicate down-to-the-north displacement across the Beaverton Fault zone, but the subsurface data are not detailed enough to determine fault dip direction. Based on seismic deaggregation the Beaverton Fault zone does not significantly contribute to the overall seismic hazard at the site.

Portland Hills Fault

The Portland Hills Fault is mapped approximately 6 km northeast of the site. The northwest-striking Portland Hills Fault forms the prominent linear northeastern margin of the Tualatin Mountains (Portland Hills) and the southwestern margin of the Portland Basin; this basin may be a right-lateral, pull-apart basin in the forearc of the CSZ or a piggyback synclinal basin formed between antiformal uplifts of the Portland fold belt. The fault is part of the Portland Hills-Clackamas River structural zone, which controlled the deposition of Miocene CRBG lavas in the region. The crest of the Portland Hills is defined by the northwest-striking Portland Hills anticline. Sense of displacement on the Portland Hills Fault is poorly known and controversial. The fault was originally mapped as a down-to-the-northeast normal fault. The fault has also been mapped as part of a regional-scale zone of right-lateral oblique slip faults and as a steep escarpment caused by asymmetrical folding above a southwest-dipping blind thrust. Reverse displacement with a right-lateral, strike-slip component may be most consistent with the tectonic setting, mapped geologic relations, aeromagnetic data, and microseismicity in the area. Fault scarps on surficial Quaternary deposits have not been described along the fault trace, but some geomorphic (steep, linear escarpment, triangular facets, over-steepened, and knick-pointed tributaries) and geophysical (aeromagnetic, seismic reflection, and ground penetrating radar) evidence suggest Quaternary displacement (Personius, 2002).

East Bank Fault

The East Bank Fault, mapped approximately 8 km northeast of the site, cuts the Pliocene Age Troutdale Formation; however, recent seismic activity has not been observed on this fault. The East Bank Fault lies in the Portland Basin, which may be a right-lateral, pull-apart basin. The tectonic setting, mapped geologic relations, aeromagnetic data, and microseismicity in the area suggest the East Bank Fault has down-to-the-northeast reverse displacement with a right-lateral, strike-slip component (Personius, 2002).

Helvetia Fault

The northwest-striking Helvetia Fault forms part of the northeastern margin of the Tualatin Basin in northwestern Oregon. The fault primarily is mapped in the subsurface on the basis of water well data, and has little aeromagnetic expression. The fault is expressed in the subsurface with down-to-the-southwest separation, but no data on fault dip or direction have been described. Most of the fault trace is covered by a thick sequence of silty sediment deposited by the Missoula floods, which may bury evidence of pre-latest Quaternary displacement (Personius, 2002). Based on seismic deaggregation, the Helvetia Fault does not significantly contribute to the overall seismic hazard at the site.

Canby-Molalla Fault

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

Table C-1. Significant Crustal Faults

Source	**Closest Mapped Distance[1] (km)**	**Mapped Length[1] (km)**
Oatfield Fault	2.3	24
Beaverton Fault Zone	4.9	15
Portland Hills Fault	5.9	49
East Bank Fault	8.1	29
Helvetia Fault	9.6	7
Canby-Molalla Fault	10.2	50

1. Reported by USGS (USGS, 2008)

SEISMIC RESPONSE ANALYSIS

We determined a probabilistic acceleration response spectra that incorporates the four postulated scenarios discussed above using the NSHMP Hazard Curve Application[1]. The NSHMP Hazard Curve Application provides access to all pre-computed hazard curves for the conterminous United States. The following sections provide a description of our analyses.

RISK TARGETED SITE RESPONSE SPECTRUM

We determined the hazard curve for the site assuming an average shear wave velocity equal to 300 m/s in the upper 30 meters of the soil profile based on the measured velocities from the CPT probe for the silt and clay and a conservative estimate of the velocity for the underlying basalt rock. The indicated average shear wave velocity corresponds to a Site Class D.

ASCE 7-10 requires that the ground motions be defined In terms of the maximum direction of horizontal response. The maximum direction was adopted as the ground motion intensity parameter for use in lieu of explicit consideration of directional effects. The maximum horizontal response may reasonably be estimated by factoring the average response period by period dependent factors. ASCE 7-10 recommends a factor of 1.1 at short periods and 1.3 at a period of 1 second and greater. Linear interpolation was used to compute factors between periods of 0.2 and 1.0 second.

---

[1] http://geohazards.usgs.gov/hazardtool/application.php

The risk targeted bedrock spectrum, MCE_R, target spectrum was computed using Method 1 outlined in ASCE 7-10 Section 21.2.1.2. A risk coefficient of $C_{RS} = 0.895$ was applied to the spectrum at periods of 0.2 second or less and a risk coefficient of $C_{R1} = 0.871$ 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 structural in a 50-year period. Table C-3 presents a summary of values used to compute the MCE_R response spectrum.

Table C-3. Risk Targeted Bedrock Spectrum

Period (seconds)	MCE Response Spectrum (300 m/s)	Maximum Direction Factor	C_R	MCE_R Response Spectrum (300 m/s)
0.0	0.440	1.100	0.900	0.436
0.1	0.690	1.100	0.900	0.683
0.2	1.011	1.100	0.900	1.001
0.3	0.955	1.125	0.896	0.962
0.5	0.814	1.175	0.888	0.849
0.75	0.593	1.238	0.878	0.644
1.0	0.481	1.300	0.868	0.543
2.0	0.221	1.300	0.868	0.249
3.0	0.124	1.300	0.868	0.140
4.0	0.081	1.300	0.868	0.091
5.0	0.052	1.300	0.868	0.059

Figure C-3 shows the site-specific risk targeted response spectrum. For comparison we have also plotted the risk targeted bedrock response spectrum and the response spectrum prescribed by ASCE-7-10 Section 11.4.3 for site class D, consistent with the site soil profile observed in the explorations.

DETERMINISTIC MCE RESPONSE SPECTRUM

The deterministic response spectrum as prescribed by ASCE-7-10 Section 21.2.2 is controlled by the deterministic lower limit. Figure C-4 shows the deterministic lower bound.

SITE-SPECIFIC MCE RESPONSE SPECTRUM

As outlined in ASCE-7-10 Section 21.2.3, the site-specific MCE shall be taken as the lesser of the probabilistic MCE and the deterministic MCE. Figure C-4 shows the site-specific design response spectrum.

DESIGN RESPONSE SPECTRUM

Section 21.3 of ASCE-7-10 prescribes that the site-specific MCE response spectrum to be 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 Section 11.4.5 of ASCE 7-10.

DESIGN ACCELERATION PARAMETERS

To develop the final design response spectrum, the lesser of the values obtained from the probabilistic MCE and the deterministic MCE are taken at each period. 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 C-5 shows the design response spectrum.

GEOLOGIC HAZARDS

In addition to ground shaking, site-specific geologic conditions can influence the potential for earthquake damage. Deep deposits of loose or soft alluvium can amplify ground motions, resulting in increased seismic loads on structures. Other geologic hazards are related to soil failure and permanent ground deformation. Permanent ground deformation could result from liquefaction, lateral spreading, landsliding, and fault rupture. The following sections provide additional discussion regarding potential seismic hazards that could affect the planned facility.

FAULT SURFACE RUPTURE

There are no mapped faults within 2 km of the site; therefore the potential for fault surface rupture is very low.

LIQUEFACTION AND LATERAL SPREADING

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

GROUND MOTION AMPLIFICATION

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

LANDSLIDE

Earthquake-induced landsliding generally occurs in steeper slopes comprised of relatively weak soil deposits. The site and surrounding area are moderately stable, and seismically induced landslides are not considered a site hazard.

SETTLEMENT

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

SUBSIDENCE/UPLIFT

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

LURCHING

Lurching is a phenomenon generally associated with very high levels of ground shaking, which cause localized failures and distortion of the soil. The anticipated ground accelerations shown on Figure C-3 are below the threshold required to induce lurching of the site soil.

SEICHE AND TSUNAMI

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

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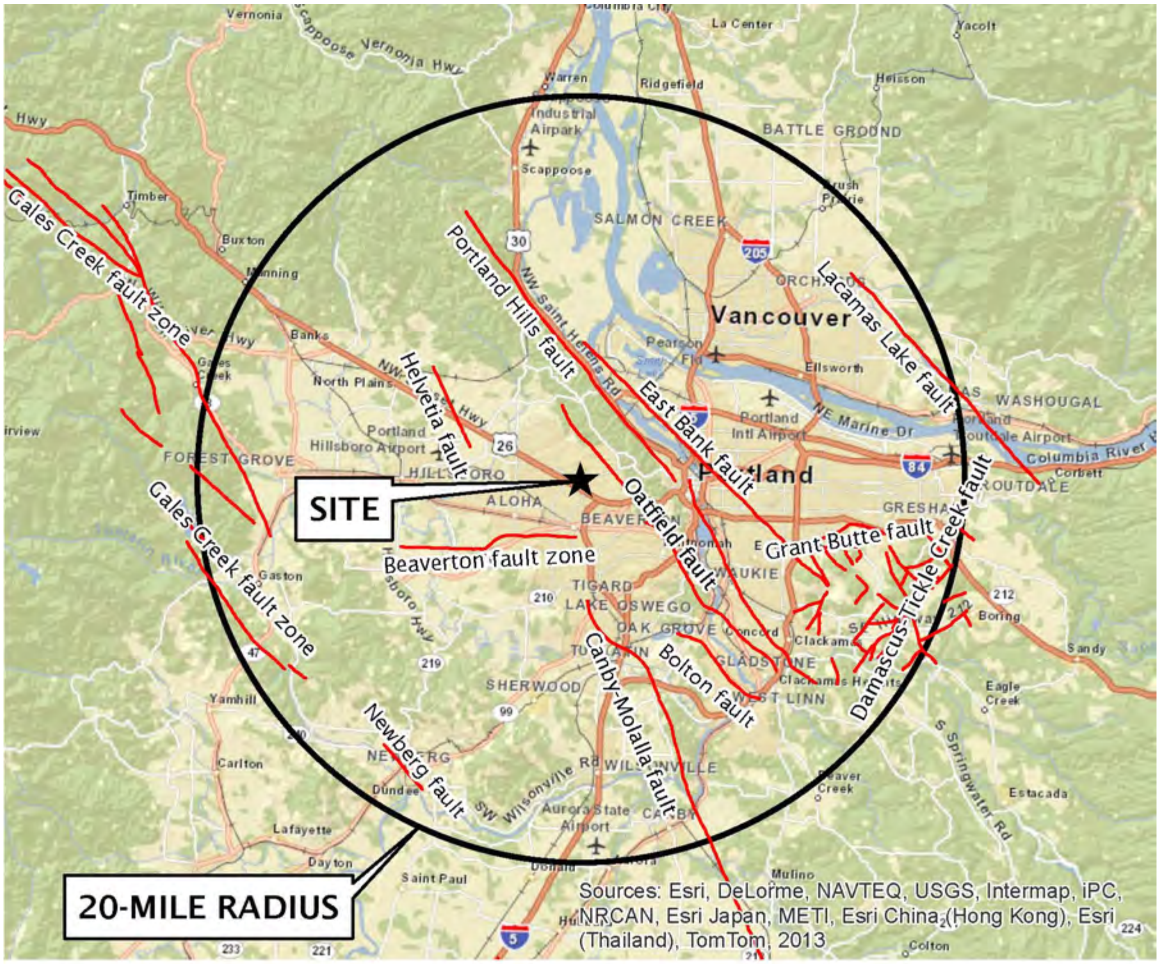
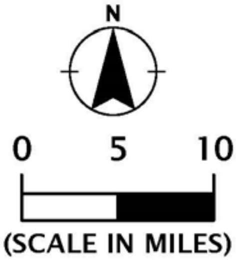


Sources: Esri, DeLorme, NAVTEQ, USGS, Intermap, iPC, NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand), TomTom, 2013

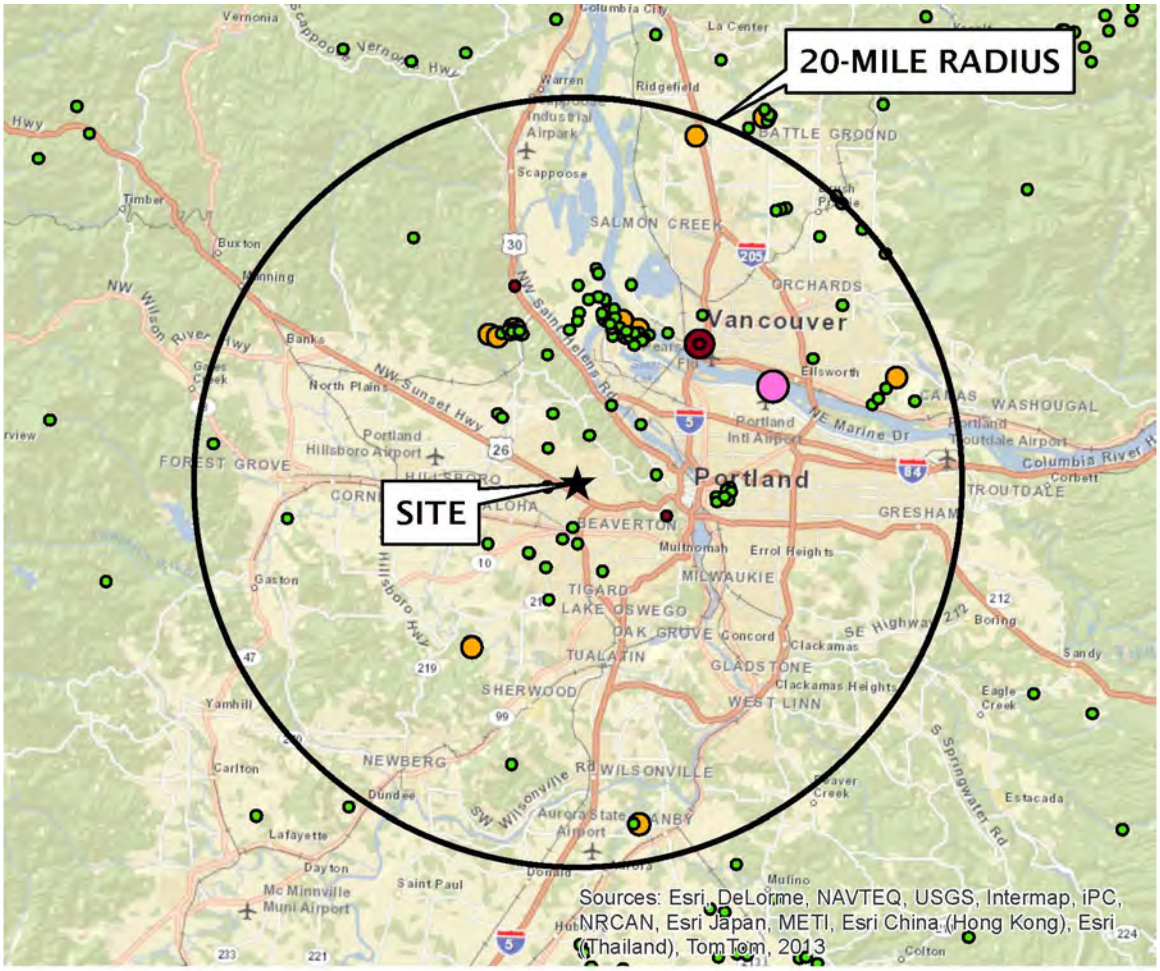
LEGEND

— QUATERNARY FAULT



FAULTS PROVIDED BY THE USGS QUATERNARY FAULT AND FOLD DATABASE (2011)

GEODESIGN INC 15575 SW Sequoia Parkway - Suite 100 Portland OR 97224 Off 503.968.8787 Fax 503.968.3068	BEAVSCHOOL-45-01	**QUATERNARY FAULT MAP**	
	DECEMBER 2014	BEAVERTON MIDDLE SCHOOL AT TIMBERLAND BEAVERTON, OR	**FIGURE C-1**



LEGEND

MAXIMUM MODIFIED MERCALLI INTENSITY (MMI)

- ● VI
- ● VII
- ● VIII

EARTHQUAKE MAGNITUDE

- ● 2.0 - 3.0
- ● 3.0 - 4.0
- ● 4.0 - 5.0
- ● > 6.0

N

0 5 10

(SCALE IN MILES)

HISTORICAL MMI DATA FROM NGDG (2010)

INSTRUMENTAL MAGNITUDE FROM USGS (2009), PNSN (2014)

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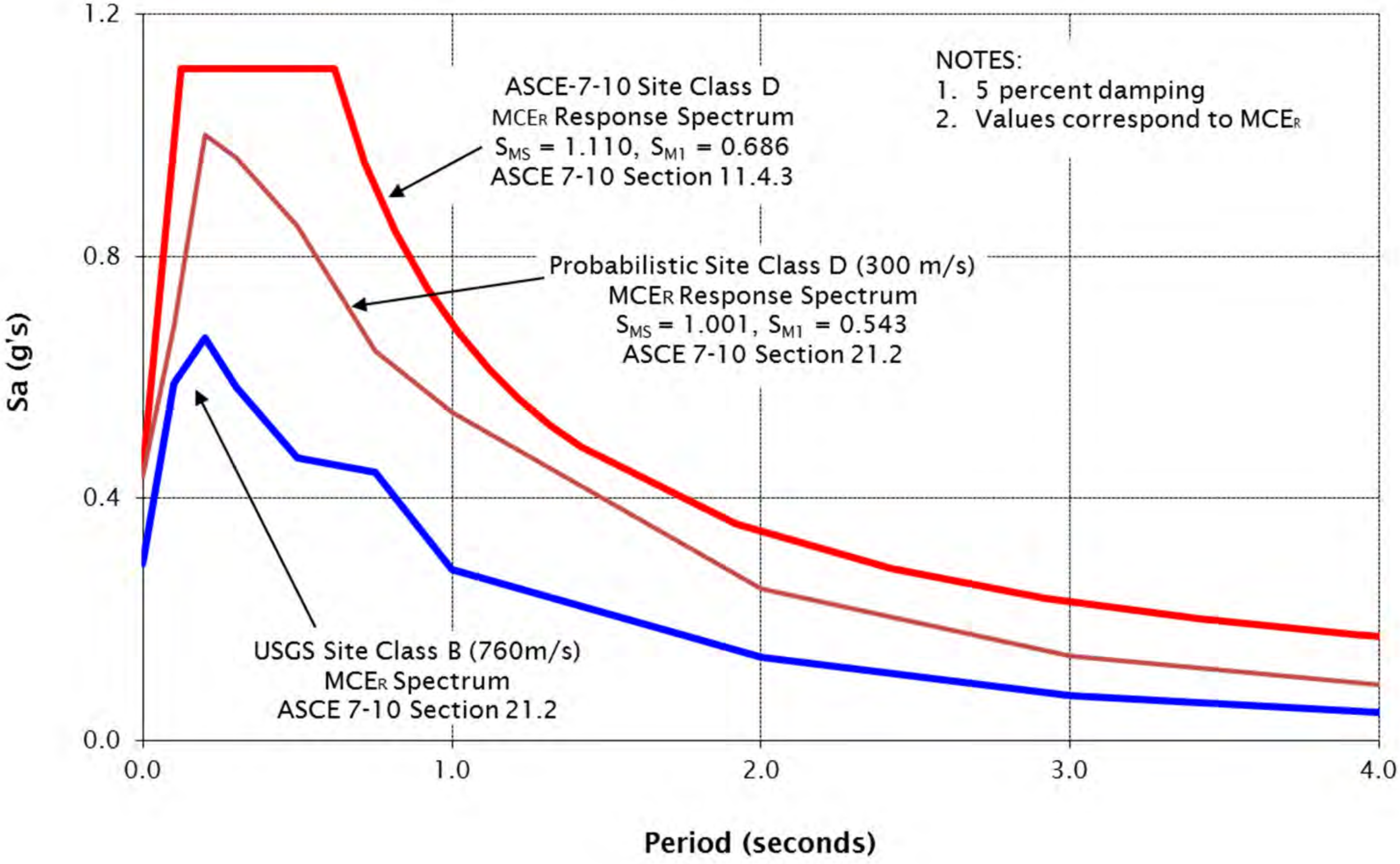
BEAVSCHOOL-45-01	**HISTORICAL SEISMICITY MAP**	
DECEMBER 2014	BEAVERTON MIDDLE SCHOOL AT TIMBERLAND BEAVERTON, OR	**FIGURE C-2**



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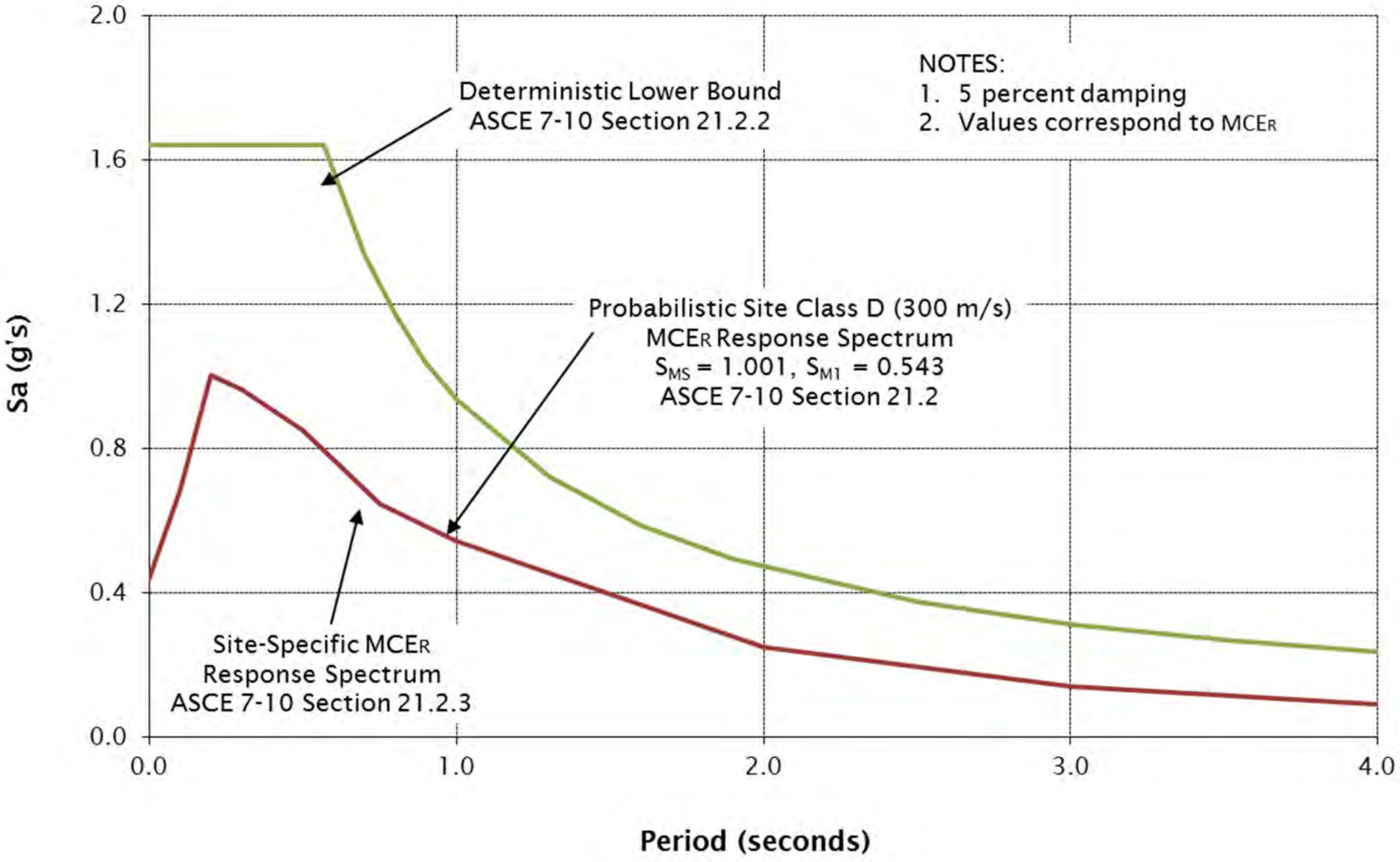
SITE RESPONSE SPECTRA

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE C-3



NOTES:
1. 5 percent damping
2. Values correspond to MCE$_R$

Deterministic Lower Bound
ASCE 7-10 Section 21.2.2

Probabilistic Site Class D (300 m/s)
MCE$_R$ Response Spectrum
S_{MS} = 1.001, S_{M1} = 0.543
ASCE 7-10 Section 21.2

Site-Specific MCE$_R$
Response Spectrum
ASCE 7-10 Section 21.2.3

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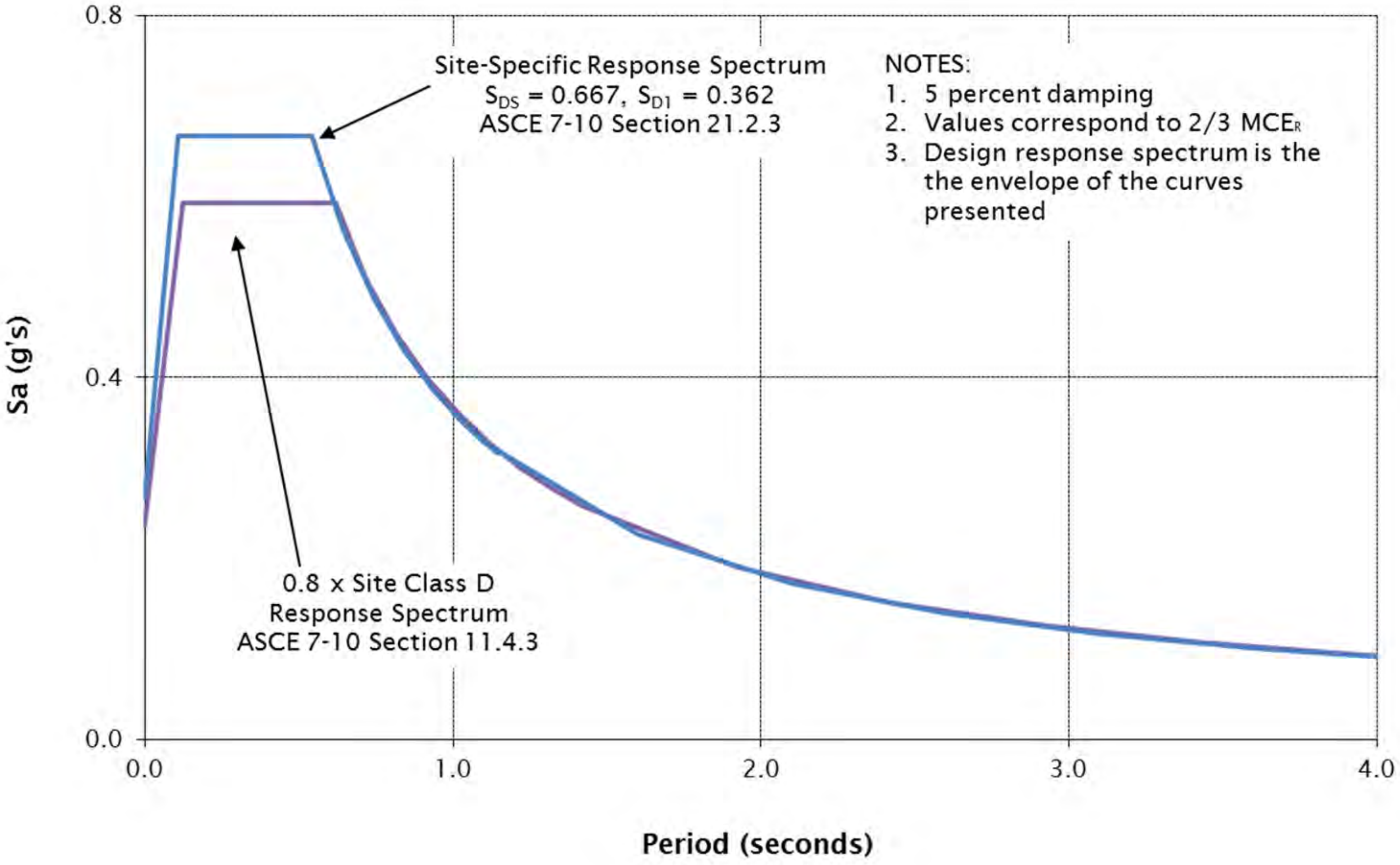
SITE-SPECIFIC RESPONSE SPECTRA

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE C-4



NOTES:

1. 5 percent damping
2. Values correspond to 2/3 MCE_R
3. Design response spectrum is the the envelope of the curves presented

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DESIGN RESPONSE SPECTRA

BEAVERTON MIDDLE SCHOOL AT TIMBERLAND
BEAVERTON, OR

FIGURE C-5

ACRONYMS

ACRONYMS

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
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
m/s	meters per second
MSE	mechanically stabilized earth
NSHMP	National Seismic Hazard Mapping Program
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
PGA	peak ground acceleration
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

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