Carlson Geotechnical

A Division of Carlson Testing, Inc.
Phone: (503) 601-8250
Fax: (503) 601-8254

Bend Office	**(541) 330-9155**
Eugene Office	**(541) 345-0289**
Salem Office	**(503) 589-1252**
Tigard Office	**(503) 684-3460**

Report of
Preliminary Geotechnical Investigation
OSU Cascades 46-Acre Site
1707 & 1757 SW Simpson Avenue
Bend, Oregon

CGT Project Number G1303959.A

Prepared for:

OSU-Cascades
Attn: Ms. Kelly Sparks / AVP Finance & Strategic Planning
650 SW Columbia Street, Suite 7250
Bend, Oregon 97702

July 25, 2014

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Dear Ms. Sparks:

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing our geotechnical investigation for the proposed OSU Cascades 46-acre development site. The project site spans two parcels located at 1707 and 1757 SW Simpson Avenue in Bend, Oregon. This report was prepared in general accordance with CGT Proposal GP6169.A, originally dated October 31, 2013 and revised December 10, 2013. Verbal authorization for our services was provided on November 21, 2013. Written authorization for our services was received on December 11, 2013, as part of the "Retainer Contract Supplement, OUS Retainer Contract for Professional Consultants, Supplement No. OSU-433-P-13-90, Cascades Campus Geotechnical Engineering". A draft version of this report was previously issued on January 30, 2014.

We appreciate the opportunity to work with you on this project. Please contact us at 503.601.8250 if you have any questions regarding this report.

Respectfully Submitted,
CARLSON GEOTECHNICAL

Brad M. Wilcox, P.E., G.E.
Senior Geotechnical Engineer
bwilcox@carlsontesting.com

Jeff A. Jones, R.G., C.E.G.
Project Engineering Geologist
jjones@carlsontesting.com

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ATTACHMENTS

1.0 INTRODUCTION

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing our preliminary geotechnical investigation for the proposed OSU Cascades 46-acre development site. The subject project site spans two parcels located at 1707 and 1757 SW Simpson Avenue in Bend, Oregon, as shown on the attached Site Location, Figure 1.

1.1 Project Description

CGT developed an understanding of the proposed project based on our correspondence with OSU Cascades and review of provided conceptual plans for the site layout, dated November 18-19, 2013. At the time of this report, the project was in the preliminary stages of planning, but will likely include the following:

- Construction of several, concrete-framed, slab-on-grade, academic and residential building(s) spaced throughout the site. Preliminary plans indicate the buildings will range from 2- to 5-stories in height. Depending on finalized locations and grading, some buildings may incorporate one or several below-grade levels. The below-grade level(s) may be fully below-grade (full basements) or daylight at the downslope end (daylight basement). Finished first floor elevations of the buildings have not been determined. Although no structural loading has been provided, we have assumed structural loads will be typical for this type of construction, with maximum column, continuous wall, and uniform floor slab loads less than 250 kips, 6 kips per lineal foot (klf), and 200 pounds per square foot (psf).
- Construction of new drive lanes and passenger car parking lots to serve the new buildings. We anticipate new pavements will be surfaced with asphaltic concrete (AC), while isolated aprons and loading docks will be surfaced with rigid (concrete) pavement.
- Construction of hardscaping features along the sides of the proposed buildings.
- Installation of underground utilities to serve the new buildings. Although no utility plans have been provided, we have assumed utility trench cuts will be up to 8 feet in depth.
- Conceptual plans include collection and diversion of stormwater into on-site infiltration facilities in accordance with the Central Oregon Stormwater Manual (COSM). The type(s), depth(s), and locations of infiltration facilities were not determined at the time of this report. No infiltration testing was performed as part of this assignment recognizing preliminary design concepts.
- Grading plans have not been developed at the time of this report. We understand finalized layout and grading of the site will be determined by OSU Cascades and the design team based, in part, on the results of the preliminary geotechnical investigation.

1.2 Scope of Work

The purpose of our work was to explore subsurface conditions at the site in order to provide preliminary geotechnical engineering recommendations for design and construction of the proposed project. This report is considered preliminary as site layout and grading plans have not been developed. Our scope of work included the following:

- Contact the Oregon Utilities Notification Center to mark the locations of public utilities at the site within a 15-foot radius of our planned explorations.

- Explore subsurface soil conditions at the site by advancing eighteen drilled borings and thirty-five test pits to depths up to about 61½ feet below ground surface (bgs).
- Classify the materials encountered in the explorations in general accordance with American Society for Testing and Materials (ASTM) D2488 (Visual-Manual Procedure).
- Collect representative, disturbed and relatively undisturbed samples of the soils encountered within the explorations in order to perform laboratory testing and to confirm our field classifications.
- Provide a technical narrative describing surface and subsurface deposits, and local geology of the site, based on the results of our explorations and published geologic mapping.
- Provide a site vicinity map and a site plan showing the locations of the explorations relative to existing site features.
- Provide logs of the explorations, including results of laboratory testing on selected soil samples.
- Provide preliminary geotechnical recommendations for site preparation and earthwork, including stripping depths, temporary excavations, subgrade preparation, wet/dry weather earthwork, utility trench excavation and backfill, general grading considerations, fill type for imported materials, use of on-site soils as structural fill, and fill compaction criteria.
- Provide preliminary geotechnical engineering recommendations for design and construction of shallow spread foundations, floor slabs, and pavements.
- Provide preliminary recommendations for the Seismic Site Class, mapped maximum considered earthquake spectral response accelerations, and site seismic coefficients.
- Provide a *qualitative* evaluation of seismic hazards at the site, including earthquake-induced settlement and landsliding, and surface rupture due to faulting or lateral spread.
- Provide this written report summarizing the results of our preliminary geotechnical investigation and recommendations for the project.

2.0 SITE DESCRIPTION

2.1 Geologic & Seismic Setting

A description of regional geology, site geology, local topography, and technical narrative describing faults within a 40-kilometer (25-mile) radius of the project is provided in the attached Appendix A.

2.2 Site Surface Conditions

Existing surface features and site topography are shown on the attached Site Plan, Figure 2. Photographs of the site taken at the time of our field investigation are shown in the attached Appendix B. The approximate 46-acre site is bordered by grass fields and landfill area to the north, a grass field and wooded parcel (proposed OSU-Cascades development property) to the east, SW Chandler Avenue to the south, and Mt. Washington Drive to the west.

The central and eastern portions of the site consisted of a sunken graded area associated with previous mining activities and are hereafter referred to as the “pit”. At the time of our fieldwork, the pit bottom was generally uneven and ascended to the west. The pit sidewalls ranged in height from about 30 to 80 feet, with slope gradients ranging from near-vertical to about 1H:1V (horizontal:vertical). No obvious signs of recent or on-going instability were noted during our reconnaissance of the sidewalls of the pit. The south

border of the pit contained an approximate 10- to 20-foot tall, fill embankment (berm). The embankment was lightly vegetated with grasses and small shrubs. The top of the berm was primarily soil-covered and was being utilized as a pedestrian access (walking) path.

The western portion of the site, hereafter referred to as the "wooded parcel" was relatively undeveloped and vegetated with grasses, understory shrubs, and ponderosa pines and other evergreen trees. Several, unimproved (primarily soil-covered) access roads were present on the site. Anecdotal evidence suggests this portion of the site was previously used for staging and fill stockpiling operations at some point in the past, likely associated with mining activities at the nearby pumice mine. In terms of topography, the southwest portion of the site was relatively level to gently ascending to the north. The northwest portion of the site was somewhat hummocky, indicative of fill berms and other grading activities.

3.0 FIELD INVESTIGATION

3.1 Geotechnical Investigation

Our geotechnical investigation was performed at the site between December 20, 2013, and January 16, 2014, and included eighteen drilled soil borings (B-1 through B-17, B-10A) and thirty-five test pits (TP-1 through TP-34, TP-7A). The approximate locations of our explorations are shown on the attached Site Plan, Figure 2. Additional details of the field investigation are presented in the attached Appendix C.

3.2 Geological Reconnaissance

CGT Certified Engineering Geologist, Jeff Jones, CEG, performed a geologic reconnaissance of the pit on December 18, 2013. The purpose of the geologic site reconnaissance was to observe site surface conditions and to characterize the geologic materials and features exposed within the pit.

3.2.1 Geologic Materials

Published geologic mapping of the site vicinity is described in the attached Appendix A. For the purposes of this report, the geologic materials encountered at the site were assigned to established geologic units on the basis of visual examination only. Color, texture, grain size/shape, and stratigraphic relations provided the primary bases for geologic classification. No radiometric dating or mineralogical analysis was performed.

3.2.2 Slope Conditions

The walls of the pit generally consisted of cut slopes, with heights ranging from about 30 to 80 feet and slope gradients ranging from near-vertical to about 1H:1V (Horizontal:Vertical). The dominant materials exposed in the slopes were pyroclastic deposits, including Tumalo Tuff (Qtu), Bend Pumice (Qb) and Desert Spring Tuff (Qds). Photographs taken during the geological reconnaissance are presented in the attached Appendix B.

In general, the site slopes did not exhibit obvious signs of recent or on-going instability, spring activity, or excessive erosion. However, areas of faulting and jointing were observed within the northern and southern slopes, as discussed below.

Within the central portion of the south pit wall, extensive jointing of the rock (Qds) was observed. Joint separation ranged from less than about 1 inch to in excess of about 1 foot. Several of the joints had previously been filled with concrete, in an apparent attempt to stabilize the individual blocks and reduce the risk of rockfall during operation of the pit. No obvious signs of rockfall were apparent. Photograph 13 in Appendix B shows the area of jointing and concrete patchwork.

Within the western portion of the north pit wall, several faults were observed. The faults appeared to be steeply dipping (near-vertical), with an apparent northwest-trending strike. The sense of motion across the faults was not consistent, with evidence of both down-to-the-west and down-to-the-east offset. Based on offset of geologic contacts across the faults, the vertical component of motion ranged from less than about 1 inch to on the order of 5 feet. It was not readily apparent whether or not the faults offset the surficial soils at the top of the slope, and detailed mapping and measuring of the faults was not performed. Photograph 14 in Appendix B shows an example of the observed faulting.

It should be noted the ground surface, both within the pit and along the rim of the pit, had been extensively modified by previous earthmoving (mining) activity. Signs of past instability, rockfall, or surface expression of faulting, may have been obscured by these activities.

4.0 LABORATORY TESTING

Laboratory testing was performed on soil samples collected in the field to refine our initial field classifications and determine in-situ properties. Details related to the number and type of laboratory tests are presented in the attached Appendix C. Graphical plots of selected laboratory tests are shown in the attached Appendix D.

5.0 SUBSURFACE CONDITIONS

5.1 Soils

Recognizing the size of the project site and variability in subsurface conditions, and for discussion purposes, we divided the project site into four regions, as shown on the attached Site Plan, Figure 2. The first region, Region 1, represents the approximate east half of the pit. The second region, Region 2, represents the approximate west half of the pit. The third region, Region 3, represents the approximate south half of the wooded parcel. The fourth region, Region 4, represents the approximate north half of the wooded parcel. The following sections provide a summary of the subsurface materials encountered within each region. As discussed in Section 3.2.1 above, for the purposes of this report, the geologic materials encountered at the site were assigned to established geologic units on the basis of visual examination only.

5.1.1 Soils – Region 1

The following table presents a "checklist" of the subsurface materials encountered in the explorations performed within Region 1. Adjacent to those materials, the tabulation presents an indicator (X) whether that subsurface material was encountered within the depth explored in the subject exploration.

Table 1: Subsurface Material "Checklist" within Area of Site Designated as Region 1

Subsurface Material[1]	USCS	Subsurface Exploration																
		B-1	B-2	B-3	B-5	B-6	B-7	TP-1	TP-2	TP-3	TP-4	TP-5	TP-6	TP-7	TP-7A	TP-8	TP-9	TP-10
Undocumented Fill	GP FILL, SM FILL	X				X	X	X										
Undocumented Rubble Fill	GP-GM FILL								X	X	X	X	X	X	X	X	X	X
Silty Sand	SM		X	X	X													
Tumalo Tuff	RX {Qtu}	X	X					X										
Desert Spring Tuff	RX {Qds}		X	X			X				X		X		X		X	X
Bend Pumice	RX {Qb}			X	X	X						X		X		X		

[1]Descriptions of each subsurface material are described below.

The following paragraphs provide a summary of the subsurface materials encountered within Region 1.

Soil Type	USCS	Geologic Interpretation
Undocumented Gravel/Sand Fill	GP FILL, SM FILL	Man-Made Fill

Undocumented fill, ranging from gravel, sandy gravel, and silty sand, was encountered at the surface of the referenced explorations and extended to depths of about 5½ to 12½ feet bgs. The gravel fill to sandy gravel fill was generally brown to gray to black, damp to moist, and fine- to coarse-grained. The silty sand fill was generally brown to dark brown, damp to moist, fine- to medium-grained, and contained no to some gravel, cobbles, and boulders (up to 3 feet in diameter). Raw (unfactored) N-values obtained from the SPTs in these soils ranged from 9 to 50+, indicating loose to very dense relative densities.

Soil Type	USCS	Geologic Interpretation
Undocumented Rubble Fill	GP-GM FILL	Man-Made Fill

Undocumented rubble fill was encountered at the surface of the referenced explorations and extended to depths of about 2 to 25+ feet bgs. The rubble fill generally consisted of gravel, cobbles, and boulders (up to about 4 feet in diameter) in a matrix of silty sand. The rubble fill was generally brown to gray and damp. In some cases, the rubble fill contained scattered concrete debris, rebar pieces, plastic debris, and/or asphalt debris. In TP-2 and TP-3, the rubble fill extended to the full depths explored (about 25 feet bgs). This depth represented the maximum reach of the referenced track-mounted excavator.

Soil Type	USCS	Geologic Interpretation
Silty Sand	SM	Volcaniclastic Sediments

Silty sand was encountered either at the surface of, or beneath an overlying layer of tuff, within the referenced explorations and extended to depths ranging from about 8 to 31½ feet bgs. The silty sand was generally medium dense to dense, brown to orange-brown, damp, fine- to coarse-grained, and contained fine gravel and occasional pumice and tuff particles. This material exhibited very low shrink-

swell properties. Raw (unfactored) N-values obtained from the SPTs in this soil ranged from 15 to 32, indicating medium dense to dense relative densities.

Soil/Rock Type	USCS	Geologic Interpretation
Tumalo Tuff, Bend Pumice, Desert Spring Tuff	RX	Pyroclastic Deposits

Tumalo Tuff, Bend Pumice, and Desert Spring Tuff were encountered below existing fill materials or the silty sand within the referenced explorations. These materials have been lumped together for discussion purposes recognizing their similar index properties and geologic origin as pyroclastic deposits. These materials extended to depths ranging from about 10 to 27 feet bgs. The Tumalo tuff was generally unconsolidated (loose to medium dense) to extremely soft (R0) to very soft (R1), damp to moist, light gray to brown to orange-brown, and contained varying amounts of pumice and welded tuff particles. The Bend Pumice was generally unconsolidated (medium dense), light gray to brown, subangular to angular, dry to moist, pumiceous, and contained varying amounts of ash. The Desert Spring Tuff was generally extremely soft (R0) to very soft (R1), slightly weathered, dark brown to black, damp to moist, and contained varying amounts of pumice, lithics, and scoria.

5.1.2 Soils – Region 2

The following table presents a "checklist" of the subsurface materials encountered in the explorations performed within Region 2. Adjacent to those materials, the tabulation presents an indicator (X) whether that subsurface material was encountered within the depth explored in the subject exploration.

Table 2: Subsurface Material "Checklist" within Area of Site Designated as Region 2

Subsurface Material[1]	USCS	Subsurface Exploration																
		B-4	B-8	B-9	B-12	TP-11	TP-12	TP-13	TP-14	TP-15	TP-16	TP-17	TP-18	TP-19	TP-20	TP-21	TP-22	TP-23
Undocumented Fill	GP FILL, SM FILL		X		X						X	X	X		X	X	X	X
Silty Sand	SM	X			X													
Desert Spring Tuff	RX {Qds}		X	X		X	X	X	X	X	X			X				X
Bend Pumice	RX {Qb}	X	X												X			
Gravel Conglomerate	RX										X							

[1]Descriptions of each subsurface material are described below.

The following paragraphs provide a summary of the subsurface materials encountered within Region 2.

Soil Type	USCS	Geologic Interpretation
Undocumented Gravel/Sand Fill	GP FILL, SM FILL	Man-Made Fill

Undocumented fill, ranging from gravel to silty sand, was encountered at the surface of the referenced explorations and extended to depths of about 12½ to 27½ feet bgs. The gravel fill was generally brown to gray, damp, coarse-grained, and contained cobbles and boulders (up to 2 feet in diameter). The silty sand fill was generally brown to black, damp to moist, dark brown, fine- to coarse-grained, and contained no to some pumice, gravel, cobbles, boulders (up to 4 feet in diameter), and scattered concrete debris, wood debris (branch, 12-inch diameter tree stump), PVC pipe debris, asphaltic concrete debris, metal debris, glass debris, and insulation debris. Raw (unfactored) N-values obtained from the SPTs in these

soils ranged from 8 to 50+, indicating loose to very dense relative densities. In several cases, we anticipate the N-values determined in the field were overstated due to the presence of coarse particles. In TP-17, TP-18, TP-21, and TP-22, the silty sand fill extended to the full depths explored (about 25 feet bgs). This depth represented the maximum reach of the referenced track-mounted excavator.

Soil Type	USCS	Geologic Interpretation
Silty Sand	SM	Volcaniclastic Sediments

Silty sand was encountered beneath an overlying layer of fill or tuff within the referenced explorations and extended to depths up to about 31½ feet bgs. The silty sand was generally brown to orange-brown, damp to moist, fine- to coarse-grained, and contained fine gravel and occasional pumice. Raw (unfactored) N-values obtained from the SPTs in this soil ranged from 32 to 50+, indicating medium dense to dense relative densities.

Soil/Rock Type	USCS	Geologic Interpretation
Bend Pumice, Desert Spring Tuff, Gravel Conglomerate	RX	Pyroclastic Deposits

Bend Pumice, Desert Spring Tuff, and Gravel Conglomerate were encountered below existing fill materials within the referenced explorations. These materials have been lumped together for discussion purposes recognizing their similar index properties and geologic origin as pyroclastic deposits. These materials extended to the full depths explored in the referenced explorations, up to about 46½ feet bgs. The Bend Pumice was generally unconsolidated (loose to medium dense), light gray to brown, subangular to angular, dry to moist, pumiceous, and contained varying amounts of ash. The Desert Spring Tuff was generally extremely soft (R0) to very soft (R1), slightly weathered, dark brown to red-brown to black, damp to moist, and contained varying amounts of pumice, lithics, and scoria. The Gravel Conglomerate was generally very soft (R1), slightly weathered, light brown to orange, moist, and contained pumice in a silt/ash matrix.

5.1.3 Soils – Region 3

The following table presents a "checklist" of the subsurface materials encountered in the explorations performed within Region 3. Adjacent to those materials, the tabulation presents an indicator (X) whether that subsurface material was encountered within the depth explored in the subject exploration.

Table 3: Subsurface Material "Checklist" within Area of Site Designated as Region 3

Subsurface Material[1]	USCS	Subsurface Exploration						
		B-10	B-10A	B-11	B-13	TP-32	TP-33	TP-34
Silty Sand	SM	X	X	X	X	X	X	X
Tumalo Tuff	RX {Qtu}	X	X	X		X		
Desert Spring Tuff	RX {Qds}				X			
Bend Pumice	RX {Qb}						X	

[1]Descriptions of each subsurface material are described below.

The following paragraphs provide a summary of the subsurface materials encountered within Region 3.

Soil Type	USCS	Geologic Interpretation
Silty Sand	SM	Volcaniclastic Sediments

Silty sand was encountered at the surface of each of the referenced explorations and extended to depths ranging from about 3 to 12 feet bgs. The silty sand was generally brown, dry to moist, fine- to coarse-grained, and contained fine gravel and occasional pumice and cobbles. Raw (unfactored) N-values obtained from the SPTs in this soil ranged from 4 to 13, indicating loose to medium dense relative densities.

Soil/Rock Type	USCS	Geologic Interpretation
Tumalo Tuff, Bend Pumice, Desert Spring Tuff	RX	Pyroclastic Deposits

Tumalo Tuff, Bend Pumice, and Desert Spring Tuff were encountered below the silty sand within the referenced explorations. These materials have been lumped together for discussion purposes recognizing their similar index properties and geologic origin as pyroclastic deposits. These materials extended to the full depths explored in the referenced explorations, up to about 61½ feet bgs. The Tumalo tuff was generally unconsolidated (loose to medium dense) to extremely soft (R0) to very soft (R1), damp to moist, light gray to brown, and contained varying amounts of pumice and welded tuff particles. The Bend Pumice was generally unconsolidated (loose to medium dense), light gray to brown, and dry to damp. The Desert Spring Tuff was generally extremely soft (R0) to very soft (R1), fresh to slightly weathered, dark brown, damp to moist, and contained varying amounts of lithics.

5.1.4 Soils – Region 4

The following table presents a "checklist" of the subsurface materials encountered in the explorations performed within Region 4. Adjacent to those materials, the tabulation presents an indicator (X) whether that subsurface material was encountered within the depth explored in the subject exploration.

Table 4: Subsurface Material "Checklist" within Area of Site Designated as Region 4

Subsurface Material[1]	USCS	Subsurface Exploration									
		B-14	B-17	TP-24	TP-25	TP-26	TP-27	TP-28	TP-29	TP-30	TP-31
Undocumented Fill	SM FILL	X	X	X	X		X		X	X	
Silty Sand	SM		X		X	X		X			X
Tumalo Tuff	RX {Qtu}			X				X	X	X	
Desert Spring Tuff	RX {Qds}					X					
Bend Pumice	RX {Qb}	X									X

[1]Descriptions of each subsurface material are described below.

The following paragraphs provide a summary of the subsurface materials encountered within Region 4.

Soil Type	USCS	Geologic Interpretation
Undocumented Silty Sand/Pumice Fill	SM FILL	Man-Made Fill

Undocumented fill, ranging from silty sand to pumice, was encountered at the surface of the referenced explorations and extended to depths of about 7½ to 31½ feet bgs. The silty sand fill was generally brown to pink-gray, damp to moist, fine- to coarse-grained, and contained no to some pumice, gravel, cobbles,

boulders (up to 2 feet in diameter). The pumice fill was generally light brown to pink to white, fine- to coarse-grained, and dry to damp. Raw (unfactored) N-values obtained from the SPTs in these soils ranged from 10 to 24, indicating medium dense relative densities. In TP-25, the silty sand fill extended to the full depth explored (about 25 feet bgs). This depth represented the maximum reach of the referenced track-mounted excavator. Based on testing of a remolded specimen, the silty sand fill exhibited "moderate" degree of specimen collapse per ASTM D5333-03.

Soil Type	USCS	Geologic Interpretation
Silty Sand	SM	Volcaniclastic Sediments

Silty sand was encountered beneath the existing fill or at the surface of the referenced explorations. The silty sand extended to depths ranging from about 6 to 36½ feet bgs. The silty sand was generally loose to medium dense, brown, dry to moist, fine- to coarse-grained, and contained fine gravel and occasional tuff fragments, gravel, and cobbles.

Soil/Rock Type	USCS	Geologic Interpretation
Tumalo Tuff, Bend Pumice, Desert Spring Tuff	RX	Pyroclastic Deposits

Tumalo Tuff, Bend Pumice, and Desert Spring Tuff were encountered below beneath the fill or the silty sand within the referenced explorations. These materials have been lumped together for discussion purposes recognizing their similar index properties and geologic origin as pyroclastic deposits. These materials extended to depths of about 9 to 30½ feet bgs in the referenced explorations. The Tumalo tuff was generally extremely soft (R0) to very soft (R1), slightly weathered, damp to moist, gray to brown to pink, and contained varying amounts of pumice and welded tuff particles. Based on testing of a remolded specimen, the Tumalo Tuff exhibited "slight" degree of specimen collapse per ASTM D5333-03. The Bend Pumice was generally unconsolidated (loose to medium dense), light gray to brown, and dry to damp. The Desert Spring Tuff was generally extremely soft (R0) to very soft (R1), fresh to slightly weathered, dark brown to black, and damp to moist.

5.2 Groundwater

We did not encounter groundwater within the depths explored at the site between December 20, 2013, and January 16, 2014. A review of well logs and water level data available at the Oregon Water Resources Department (OWRD) website[1] for wells located within about 1½ miles of the site indicates groundwater levels in excess of 240 feet bgs. It should be noted that groundwater levels vary with local topography. In addition, the groundwater levels reported on the OWRD logs often reflect the purpose of the well, so water well logs may only report deeper, confined groundwater, while geotechnical or environmental borings will often report any groundwater encountered, including shallow, unconfined groundwater. Therefore, the levels reported on the OWRD well logs referenced above are considered generally indicative of local water levels and may not reflect actual groundwater levels at the site. We anticipate that groundwater levels will fluctuate due to seasonal and annual variations in precipitation, changes in site utilization, or other factors. However, such fluctuation (if it occurs) is anticipated to be at depths well below that considered of concern for this project.

[1] Oregon Water Resources Department, 2014. Water Level Data and Hydrographs http://www.oregon.gov/owrd/pages/gw/well_data.aspx

6.0 SEISMIC CONSIDERATIONS

6.1 Seismic Hazards

The complete results of our seismic hazard evaluation for this project site are presented in Section A.1.5 of the attached Appendix A. The following highlights the results of our evaluation:

- We conclude there is a negligible risk of liquefaction at the site.
- We conclude there is a negligible risk of surface rupture from lateral spread.
- We conclude there is at least a moderate risk of slope instability from a design-level earthquake. Refer to Section 7.3 of this report for additional discussion.
- We conclude there is a low risk of surface rupture from faulting.

6.2 Seismic Site Class

Based on the results of the explorations and review of geologic mapping, we have assigned the site as Site Class D for the subsurface conditions encountered in accordance with Table 1613.5.2 of the 2010 Oregon Structural Specialty Code (2010 OSSC). Preliminary recommendations for seismic ground motion values at the site are presented in Section 10.4 of this report.

7.0 GEOTECHNICAL REVIEW & DISCUSSION

7.1 Overview

Based on the results of our field explorations and analyses, the site may be developed as conceptually described in Section 1.1 of this report. We conclude the primary geotechnical considerations for the currently planned project are:

(1) the presence of uncontrolled fill materials within portions of the site intended for development.
(2) the presence of relatively steep, tall cut slopes along the north, south, and west rims of the pit.

These considerations are described in more detail in the following sections.

7.2 Presence of Uncontrolled Fill Materials

As indicated in Sections 5.1.1, 5.1.2, and 5.1.4 of this report, uncontrolled fill materials were encountered within several of our subsurface explorations advanced within “Region 1”, “Region 2”, and “Region 4” of the project site. The attached Figure 2 shows the depths of the uncontrolled fill encountered at our exploration locations. To further help illustrate areas of the site containing significant uncontrolled fills, we have prepared a supplemental site plan (attached as Figure 4) that shows areas of the site underlain by at least 10 feet of uncontrolled fill. This plan has been prepared principally for illustrative purposes and was based on the results of our explorations, site observations, and review of topographic irregularities.

To the best of our knowledge, there is no available documentation detailing the placement and compaction of the existing fill materials at the project site. Our explorations showed the fill materials were highly variable in terms of type, composition, and relative density/compaction. The fill materials ranged

from silty sand, pumice, gravel, sandy gravel, and rubble fill (cobbles, and boulders up to 4 feet in diameter). In some cases, the existing fill contained debris, including concrete, asphaltic concrete, metal, pipe fragments, glass, and/or insulation. In isolated cases, we encountered discrete organic debris, including branches and a 12-inch-diameter tree stump. No organic layers or organic-laden materials were encountered during our investigation.

Recognizing the variability in relative density/compaction, the presence of over-sized particles (over 12 inches in diameter), and in some cases, the presence of construction material debris, it is our opinion the existing uncontrolled fill materials were not placed in accordance with typical code requirements for structural fill. If relied upon for subgrade support of planned buildings, pavements, hardscaping, and/or other structural improvements, we conclude there is a moderate to high risk of uneven subgrade response below, and potential for excessive, post-construction settlements of, those features. We recommend the existing uncontrolled fill be mitigated where present within finalized locations for new structural features at the site. Given the preliminary nature of layout, grading, and design of the proposed project, it is difficult to develop "blanket" recommendations for mitigation of the existing uncontrolled fill across the site. Accordingly, we have identified options that we anticipate should be effective for mitigation of the existing fill materials in building and pavement areas for OSU Cascades' consideration. These options are presented in the following table.

Table 5: Options for Mitigation of Existing Uncontrolled Fill

Type of Fill Present	Mitigation Options	
	Building Areas	**Pavement/Exterior Hardscaping Areas**
Fill Soil Primarily Free of Boulders and Large Debris	Full Removal & Replacement[1]	Full Removal & Replacement[1]
	Displacement (Driven) Piles[2]	Ground Improvement Techniques[4]
	Non-Displacement (Drilled) Piles[3]	Relocation of Feature (away from fill area)[6]
	Ground Improvement Techniques[4]	
	Earth Stabilization Columns[5]	
	Relocation of Building (away from fill area)[6]	
Fill Soil with Moderate to Heavy Concentration of Boulders and/or Large Debris	Full Removal & Replacement[1]	Full Removal & Replacement[1]
	Relocation of Building (away from fill area)[6]	Relocation of Feature (away from fill area)[6]
		Partial Removal & Geo-Grid Reinforcement (provision for increased risk)[7]

[1] Depths of fill removal and replacement will be a function of the finalized grading plan per civil engineer.
[2] Such as steel H-piles, steel pipe piles, pre-cast concrete piles, micro-piles, etc. Considered applicable for areas of relatively deep fill.
[3] Such as drilled piers. Considered applicable for areas of moderately deep fill (up to 20 feet deep).
[4] Such as deep in-place soil mixing, jet grouting, etc. Considered applicable for areas of relatively deep fill.
[5] Such as stone columns (vibro-replacement), granular piers, etc. Considered applicable for areas of relatively deep fill.
[6] Subject to preferences of owner and review of design team.
[7] This approach should help reduce, but not eliminate, the potential for post-construction settlements of fill left in-place below these features. The owner (OSU Cascades) would need to recognize and accept an increased risk of area-wide settlements from consolidation/densification of the underlying, uncontrolled fill.

As project plans are developed, we recommend the geotechnical engineer be consulted to review the plans in an effort to help determine the most practical and economic option(s) for supporting new structural features at the site. Specific geotechnical recommendations for use in design and construction of foundations, floor slabs, pavements, hardscaping, and other site features can be reasonably developed after layout and grading criteria have been established. Additional geotechnical investigation (drilled borings and/or test pits) may be recommended in some cases to further characterize subsurface

conditions for the purposes of developing recommendations, particularly in areas of the site containing relatively deep, uncontrolled fill.

7.3 Presence of Relatively Steep, Tall Cut Slopes Along North, South, and West Rims of Pit

As indicated in Section 2.2 of this report, the central and eastern portions of the site consist of a former pumice mine ("pit") and contain near-vertical sidewalls ranging from about 30 to 80 feet in height. Conceptual site layout plans provided by OSU Cascades show buildings and drive lanes may be placed above, within (by construction of fill embankments), and near the toe of the west and north sidewalls of the pit within "Region 2". The plans also show a building and appurtenant features may be placed relatively near the toe of the south sidewall of the pumice pit within "Region 1".

Based on the results of our explorations and geological field reconnaissance, the sidewalls of the pit consist of pyroclastic deposits, ranging from ash to lapilli tuff, ash flow tuff, or pumice. To help illustrate stratigraphy of the sideslopes, we developed seven geologic cross sections of the sidewalls (A-A' through G-G') using subsurface information collected from the explorations and the geological reconnaissance, presented in Appendix E. The locations of the slope cross sections are shown on Figure E1 contained within the attached Appendix E.

No areas of moderate- or large-scale (deep-seated) past or ongoing instability were evident at the site during our investigation and geological reconnaissance. Notwithstanding the preceding statement, we conclude the pit sidewalls, considering their heights, gradients, and composition, are susceptible to slope instability and rockfall, particularly if subjected to seismic loading. Generally speaking, ground shaking from design-level seismic events can induce slope failures, including landslides or rockfall, on otherwise stable (or marginally stable) slopes. As noted in Appendix A, we identified several potential sources for earthquakes in the region of the site. Accordingly, it is our opinion that the hazard level associated with slope instability and/or rockfall at the site is at least moderate.

Given the preliminary nature of layout, grading, and design of the proposed project, it is difficult to develop "blanket" geotechnical recommendations for addressing this slope hazard. Accordingly, we have identified options for OSU Cascades' consideration. These options are presented in the following table, in order of anticipated increased cost for the project.

Table 6: Options for Consideration for Development Near Steep Site Slopes

Option	Discussion[1]
1 – Avoid Hazard through Proper Setback	• This option would include locating buildings, drive lanes, parking areas, and other structural features at a sufficient distance away from on-site slopes exceeding 1H:1V in gradient. To help illustrate setback of buildings per current OSSC requirements, the attached Figure 2 includes shading indicating minimum setback of building clearances from ascending (and descending) slopes. • Pedestrian access would be restricted from the top and bottom of the slopes through fencing or other barrier system. Draping of the slopes could also be considered to help provide protection. • This option is subject to owner preferences and review of design team.
2 – Quantitatively Analyze Slope Stability	• This option would include evaluating the stability of the existing slopes using slope stability software. The intent of this analysis would (conceptually) be to determine safety factors against slope instability and determine whether reduced slope setbacks could be achieved. • The owner is advised proceeding with quantitative stability analyses may not lead to reduction in slope setbacks.
3 – Re-grade Slopes	• This option would (conceptually) include re-grading the site slopes, where practical to do so. • Re-grading of the north, west, and south side slopes could include placement of structural fill near the toe of the cut slope to serve as a buttress. • Re-grading of the west side slope could also take the form of pulling back (flattening) the slope to achieve a flatter gradient. • This option would be subject to owner preferences and review of project civil engineer and architect.
4 – Install Retaining Structures	• This option would (conceptually) include design and installation of retaining structures to retain the existing side slopes of the pit. • Steel sheet pile walls, tieback/anchored walls, or other wall systems could be considered.

[1] The owner and design team may consider one, or a combination of the, above options to address development near steep slopes.

As project plans are developed, we recommend the geotechnical engineer be consulted to review the plans in an effort to help determine the most practical and economic option(s) for addressing the slope stability and rockfall hazard presented by the sidewalls of the pit. Depending on the option selected, the owner is advised additional geotechnical investigation and analyses may be recommended to further characterize subsurface conditions above, within, or below site slopes.

8.0 PRELIMINARY RECOMMENDATIONS: SITE PREPARATION & EARTHWORK

The preliminary recommendations presented below are provided for general planning purposes and are subject to revision once layout and grading plans for the project are further developed. Our preliminary recommendations are based on the information provided to us, results of the field investigation, laboratory data, and professional judgment. CGT has observed only a small portion of the pertinent subsurface conditions. The recommendations are based on the assumptions that the subsurface conditions do not deviate appreciably from those found during the field investigation.

8.1 Site Preparation

8.1.1 Stripping

Surface vegetation and rooted soils should be removed from within, and for a 5-foot margin around, the proposed structural fill, building, and pavement locations. Based on the results of the field explorations, stripping depths at the site are anticipated to be about ¼- to ½-foot bgs. These materials may be shallow or deeper away from the exploration locations. The geotechnical engineer or his representative should provide recommendations for actual stripping depths based on observations during site stripping. Stripped surface vegetation and rooted soils should be transported off-site for disposal, or stockpiled for later use in landscaped areas.

8.1.2 Grubbing

Grubbing of shrubs and trees should include the removal of the root mass, and roots greater than 1-inch in diameter. Grubbed materials should be transported off-site for disposal. Where root masses are removed, the resulting excavation should be properly backfilled with imported granular structural fill in conformance with Section 8.4.3.2 of this report, as needed to achieve finished subgrade elevations.

8.1.3 Existing Utilities & Below-Grade Structures

All existing utilities at the site should be identified prior to excavation. Abandoned utility lines beneath new buildings and pavements should be completely removed or grouted full. Soft, loose, or otherwise unsuitable soils encountered in utility trench excavations should be removed and replaced with imported granular structural fill in conformance with Section 8.4.3.2 of this report. No below-grade structures were encountered during our field investigation of the site. If encountered during site preparation, buried structures, including but not limited to, footings, foundation walls, slabs-on-grade, tanks, or pavements, should be completely removed and disposed of off-site.

8.1.4 Erosion Control

Erosion and sedimentation control measures should be employed in accordance with applicable City, County, and State regulations regarding erosion control.

8.2 Temporary Excavations

8.2.1 Overview

All excavations should be in accordance with applicable OSHA and state regulations. It is the contractor's responsibility to select the excavation methods, to monitor site excavations for safety, and to provide any shoring required to protect personnel and adjacent improvements. A “competent person”, as defined by OR-OSHA, should be on-site during construction in accordance with regulations presented by OR-OSHA. CGT’s current role on the project does not include review or oversight of excavation safety.

8.2.2 Dewatering

Recognizing the depth to regional groundwater at this site (in excess of 200 feet bgs), we do not anticipate that site excavations will require area-wide dewatering. If groundwater seepage is encountered

on temporary cut slopes during construction, provisions may be required to collect and divert the water from the cut slope and reduce the potential of instability. The geotechnical engineer should be consulted in the event groundwater seepage emerges within cut slopes.

8.2.3 Utility Trenches

Temporary trench cuts should stand near vertical to depths of approximately 4 feet in the native silty sand (SM), tuff (RX), and Bend Pumice (RX) encountered at the site. Depending on time of year, some instability may occur in excessively dry, cohesionless soils within the upper few feet of the site surface. In the event that caving of the sidewalls is observed during excavation, the sidewalls should be flattened or shored. Although not anticipated, trench dewatering may be required in order to maintain dry working conditions, particularly if significant perched water and seepage is encountered. If groundwater is present at the base of utility excavations, we recommend placing trench stabilization material at the base of the excavations. Trench stabilization material should be in conformance with Section 8.4.5 of this report.

8.2.4 OSHA Soil Type

8.2.4.1 Silty Sand

Conventional earthmoving equipment in proper working condition should be capable of making cuts within this soil. For use in the planning and construction of temporary excavations at the site, an OSHA soil type "C" should be used for this soil. We anticipate some instability of the silty sand may occur if seepage occurs, particularly during or after heavy rains. If seepage is encountered that undermines the stability of the excavation, or caving of the sidewalls is observed during excavation, the sidewalls should be flattened or shored.

8.2.4.2 Tumalo Tuff, Bend Pumice & Desert Springs Tuff

Conventional earthmoving equipment in proper working condition should be capable of making cuts within the on-site tuff and pumice. Recognizing their primarily sandy nature, an OSHA soil type "C" should be used when considering temporary excavations into these materials.

8.2.5 Excavations Near Foundations

Excavations near footings should not extend within a 1H:1V (horizontal:vertical) plane projected out and down from the outside, bottom edge of the footings. In the event that excavation needs to extend below the referenced plane, temporary shoring of the excavation and/or underpinning of the subject footing may be required. The geotechnical engineer should be consulted to review proposed excavation plans for this design case to provide specific recommendations.

8.3 Wet Weather Considerations

Notwithstanding the generally arid conditions of the Bend area, soil conditions should be evaluated in the field by the geotechnical engineer or his representative at the initial stage of site preparation to determine whether the recommendations within this section should be incorporated into construction.

8.3.1 Overview

Trafficability of the near-surface, silty sand (SM) and tuff (RX) may be difficult, and significant damage to subgrade soils could occur, if earthwork is undertaken without proper precautions at times when the exposed soils are more than a few percentage points above optimum moisture content. Site preparation activities may need to be accomplished using track-mounted equipment, loading removed material onto trucks supported on granular haul roads, or other methods to limit soil disturbance. The geotechnical engineer or his representative should evaluate the subgrade during excavation by probing rather than proof rolling. Soils that have been disturbed during site preparation activities, or soft or loose areas identified during probing, should be over-excavated to firm, stable subgrade, and replaced with imported granular structural fill in conformance with Section 8.4.3.2 of this report.

8.3.2 Geotextile Separation Fabric

A geotextile separation fabric should be placed to serve as a barrier between fine-grained subgrades and imported fill in areas of repeated or heavy construction traffic. The geotextile fabric should be in conformance with Section 02320 of the most current Oregon Department of Transportation (ODOT) Standard Specification for Construction. In accordance with Table 02320-1 of ODOT specifications, the separation fabric should have minimum puncture strength (ASTM D4833) of 80 pounds and an apparent opening size (ASTM D4751) no larger than the U.S. Standard No. 30 sieve. Examples of products that currently meet these requirements include Propex Geotex 200ST and US Fabrics US200. Other products meeting the requirements set forth by ODOT specifications may be considered for separation geotextile fabric.

8.3.3 Granular Working Surfaces

Haul roads subjected to repeated heavy, tire-mounted construction traffic (e.g. dump trucks, concrete trucks, forklifts, etc.) will require a minimum of 18 inches of imported granular material. For light staging areas subjected to light, tire-mounted equipment (e.g. pickups) or track-mounted equipment, 12 inches of imported granular material should be sufficient. Additional granular material, geo-grid reinforcement, or cement amendment may be recommended based on site conditions and/or loading at the time of construction. The imported granular material should consist of imported granular structural fill in conformance with Section 8.4.3.2 of this report and have less than 5 percent passing the U.S. Standard No. 200 Sieve. The prepared subgrade should be covered with geotextile fabric prior to placement of the imported granular material. The imported granular material should be placed in a single lift and compacted using a smooth-drum, non-vibratory roller until achieving a well-keyed condition.

8.3.4 Footing Subgrade Protection

A minimum of 3 inches of imported granular material is recommended over fine-grained, foundation subgrades in order to provide protection from foot traffic during inclement weather. The imported granular material should be in conformance with Section 8.4.3.2 of this report, contain a maximum particle size of 1 inch, and have less than 5 percent passing the U.S. Standard No. 200 Sieve. The imported granular material should be placed in one lift over the prepared, undisturbed subgrade, and compacted using non-vibratory equipment until well-keyed.

8.4 Structural Fill

8.4.1 Overview

On-site or imported materials intended for use as structural fill at the site should be reviewed by the geotechnical engineer prior to placement. The geotechnical engineer or his representative should be contacted to evaluate compaction of structural fill as the material is being placed. Evaluation of compaction may take the form of in-place density tests, deflection (proof roll) tests, or other testing methods accepted by the geotechnical engineer. The following table presents recommended guidelines for frequency of density testing (where practical) of various fill designations.

Table 7: Recommended Guidelines for Frequency of Density Testing

Fill Designation	Recommended Frequency of Density Tests	
	Maximum Depth Interval	Area-Wide
General Structural Fill (Mass Grading)	Test every 1 vertical foot	At least one density test per 2,000 feet^2 of fill area
Utility Trench Backfill$^{\alpha}$	Test every 2 vertical feet	At least one density test per 50 feet of trench line
Pavement Base Rock$^{\alpha}$	Test at surface of section	At least one density test per 2,000 feet^2 of base rock area
Floor Slab Base Rock	Test at surface of section	At least one density test per 1,000 feet^2 of base rock area
$^{\alpha}$Testing frequency within the public right-of-way should be in conformance with the local jurisdiction requirements.		

8.4.2 On-Site Materials – General Use

8.4.2.1 Silty Sand, Tumalo Tuff, Bend Pumice & Desert Springs Tuff

Re-use of these materials as structural fill is feasible, provided they are kept free of organic matter, debris, and particles larger than about 2 inches. When used as structural fill, these soils should be placed in lifts with a maximum thickness of about 9 inches at moisture contents within –1 and +3 percent of optimum, and compacted to not less than 95 percent of the material's maximum dry density as determined in accordance with ASTM D1557 (Modified Proctor).

8.4.2.2 Silty Sand Fill, Sandy Gravel Fill, Gravel Fill & Pumice Fill

Re-use of these fill materials as structural fill is feasible, provided they can be kept free of organics, debris, and other deleterious materials, and processed ("picked") free of large particles (cobbles and boulders) in excess of 4 inches in diameter. When used as structural fill, these soils should be placed in lifts with a maximum thickness of about 9 inches at moisture contents within –1 and +3 percent of optimum, and compacted to not less than 95 percent of the material's maximum dry density as determined in accordance with ASTM D1557 (Modified Proctor). Where the material contains a high concentration of over-sized particles, evaluation of relative compaction should be performed by deflection (proof roll) testing in accordance with ODOT Test Method TM 158.

8.4.2.3 Existing Rubble Fill

Re-use of the existing rubble fill (primarily consisting of cobbles, concrete debris, and boulders up to about 4 feet in diameter) as structural fill will require extensive processing (removal or crushing) of over-sized particles and debris. Due to the concentration of boulders and debris, the economics of processing this material for re-use as structural fill should be weighed. The processed/crushed material should be

prepared to achieve a fill that is fairly well graded between coarse and fine. The maximum particle size should be limited to 4 inches. As a guideline, grading of this material with particles up to about 4 inches in diameter may follow that presented in the following table.

Table 8: Guideline Gradation of Processed/Crushed Rubble Fill

Sieve Size	% Passing
4 inches	100
3 inches	88 – 100
¾-inch	70 – 90
U.S. Standard No. 4	40 – 60
U.S. Standard No. 40	20 – 40
U.S. Standard No. 200	Dry Weather: Less than 12 | Wet Weather: Less than 5

When used as structural fill, this material should be placed in lifts with a maximum thickness of about 9 inches at moisture contents within –1 and +3 percent of optimum, and compacted to not less than 95 percent of the material's maximum dry density as determined in accordance with ASTM D1557 (Modified Proctor). Where the material contains a high concentration of over-sized particles, evaluation of relative compaction should be performed by deflection (proof roll) testing in accordance with ODOT Test Method TM 158. Proof roll tests should be performed at maximum intervals of every 1 vertical foot as the fill is being placed.

If the on-site soils cannot be properly moisture-conditioned and/or processed, we recommend using imported granular material for structural fill.

8.4.3 Imported Fill (General Use)

8.4.3.1 Imported Material(s) with Appreciable Fines Content

Imported fill materials with a relatively high concentration of fines (e.g. clay- to silt-sized particles) may be considered for use as structural fill during mass grading. For the purposes of discussion, a fill material containing more than 12 percent passing the U.S. Standard No. 200 Sieve constitutes a material with relatively high concentration of fines. Subject to the review of the geotechnical engineer, fill material(s) meeting this designation may be used as structural fill (general use) at the site, provided they can be moisture-conditioned and compacted in conformance with the recommendations presented in Section 8.4.2.1 of this report, and are free of organic matter, debris, and particles larger than 4 inches. Fill materials with a high concentration of fines are best suited for use during dry weather conditions, as they inherently are sensitive to changes in moisture content and are difficult, if not impossible, to adequately compact during wet weather. Specific recommendations for placement and compaction of imported fill materials with appreciable fines content can be provided by the geotechnical engineer on a case-by-case basis.

8.4.3.2 Imported Granular Fill with Low Fines Content

Imported granular fill should consist of angular pit or quarry run rock, crushed rock, or crushed gravel that is fairly well graded between coarse and fine particle sizes. The granular fill should contain no organic matter, debris, or particles larger than 4 inches, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. The percentage of fines can be increased to 12 percent of the material passing

the U.S. Standard No. 200 Sieve if placed during dry weather, and provided the fill material is moisture-conditioned, as necessary, for proper compaction. As a guideline, grading of this material with particles up to about 4 inches in diameter may follow that presented in Table 8 above. Imported granular fill material should be placed in lifts with a maximum thickness of about 12 inches at moisture contents within –1 and +3 percent of optimum, and compacted to not less than 95 percent of the material's maximum dry density as determined in accordance with ASTM D1557 (Modified Proctor). Granular fill materials with high percentages of particle sizes in excess of 1½ inches are considered non-moisture-density testable materials. As an alternative to conventional density testing, compaction of these materials should be evaluated by periodic deflection (proof roll) testing in accordance with ODOT Test Method 158. Proof roll tests should be performed at maximum intervals of every 1 vertical foot as the fill is being placed.

8.4.4 Floor Slab Base Course

Floor slab base course should consist of well-graded granular material (crushed rock or gravel) containing no organic matter or debris, have a maximum particle size of ¾ inch, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. The material should be placed in one lift and compacted to not less than 95 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor). As a guideline, the material should consist of a well-graded, ¾-inch minus crushed aggregate meeting the requirements of the most recent Oregon Standard Specifications for Construction, Section 2630.10, Table 02630-1 "Grading Requirements for Dense-Graded Aggregate", for ¾-inch minus rock. A guideline base course gradation criterion is presented in the following table.

Table 9: Guideline Gradation for Floor Slab Base Course

Sieve Size	% Passing (See Note 1)
1 inch	100
¾ inch	90 – 100
½ inch	-----
⅜ inch	55 – 75
¼ inch	40 – 60
U.S. Standard No. 4	-----
U.S. Standard No. 8	-----
U.S. Standard No. 10	See Note 2
U.S. Standard No. 16	-----
U.S. Standard No. 200	0 – 5

Note 1: Gradation should conform to the most current, ODOT specifications, for ¾-inch minus rock.
Note 2: Of the fraction passing the ¼-inch sieve, 40% to 60% shall pass the No. 10 sieve.

8.4.5 Trench Base Stabilization Material

If groundwater is present at the base of utility excavations, trench base stabilization material should be placed. Trench base stabilization material should consist of 1 foot of well-graded granular material with a maximum particle size of 4 inches and less than 5 percent material passing the U.S. Standard No. 4

Sieve. The material should be free of organic matter and other deleterious material, placed in one lift, and compacted until well-keyed.

8.4.6 Utility Trench Backfill

Trench backfill for the utility pipe base and pipe zone should consist of granular material as recommended by the utility pipe manufacturer. Trench backfill above the pipe zone should consist of well-graded granular material containing no organic matter or debris, have a maximum particle size of ¾ inch, and have less than 8 percent material passing the U.S. Standard No. 200 Sieve. As a guideline, trench backfill should be placed in maximum 12-inch thick lifts. The earthwork contractor may elect to use alternative lift thicknesses based on their experience with specific equipment and fill material conditions during construction in order to achieve the required compaction. The following table presents recommended relative compaction percentages for utility trench backfill.

Table 10: Utility Trench Backfill Compaction Recommendations

Backfill Zone	Recommended Minimum Relative Compaction	
	Structural Areas[1]	Landscaping Areas
Pipe Base and Within Pipe Zone	90% ASTM D1557 or pipe manufacturer's recommendation	88% ASTM D1557 or pipe manufacturer's recommendation
Above Pipe Zone	92% ASTM D1557	90% ASTM D1557
Within 3 Feet of Design Subgrade	95% ASTM D1557	90% ASTM D1557

[1]Includes proposed structural fill areas, buildings, pavements, hardscaping, etc.

8.4.7 Controlled Low-Strength Material (CLSM)

CLSM is a self-compacting, cementitious material that is typically considered when backfilling localized areas. CLSM is sometimes referred to as "controlled density fill" or CDF. Due to its flowable characteristics, CLSM typically can be placed in restricted-access excavations where placing and compacting fill is difficult. If chosen for use at this site, we recommend the CLSM be in conformance with Section 00442 of the most recent, State of Oregon, Standard Specifications for Highway Construction. The geotechnical engineer's representative should observe placement of the CLSM and obtain samples for compression testing in accordance with ASTM D4832. As a guideline, for each day's placement, two compressive strength specimens from the same CLSM sample should be tested. The results of the two individual compressive strength tests should be averaged to obtain the reported 28-day compressive strength.

8.5 Additional Considerations

8.5.1 Drainage Considerations

Subsurface drains should be connected to the nearest storm drain, on-site stormwater infiltration facilities (designed by others), or other suitable discharge point. If on-site infiltration of stormwater is considered, the geotechnical engineer should be consulted to review the proposed construction. Paved surfaces, and ground near or adjacent to buildings, should be sloped to drain away from the buildings. Surface water

from pavement surfaces and open spaces should be collected and routed to a suitable discharge point. Surface water should not be directed into foundation drains or onto site slopes.

8.5.2 Freezing Weather Considerations

For construction that occurs during extended periods of sub-freezing temperatures, the following special provisions are recommended:

- Structural fill should not be placed over frozen ground.
- Frozen soil should not be placed as structural fill.
- Fine-grained soils should not be placed as structural fill during sub-freezing temperatures.

Identification of frozen soils at the site should be in accordance with ASTM D4083-01 "Standard Practice for Description of Frozen Soils (Visual-Manual Procedure)" or other method approved by the geotechnical engineer. The geotechnical engineer can aid the contractor with supplemental recommendations for earthwork that will take place during extended periods of sub-freezing weather, as required.

8.6 Permanent Slopes

Permanent cut or fill slopes constructed at the site should be graded at 2H:1V or flatter. Constructed slopes should be overbuilt by a few feet depending on their size and gradient so that they can be properly compacted prior to being cut to final grade. The surface of all slopes should be protected from erosion by mulching, seeding, sodding, or other acceptable means. Construction of fill slopes on surfaces exceeding 5H:1V in declivity should be keyed and benched into the sloped surface. The geotechnical engineer should be consulted to review proposed fill slopes and provide supplemental geotechnical recommendations for site preparation and construction as grading plans are being developed.

8.7 Foundation Setback from Ascending Slopes

Section 1808.7.1 of the 2010 OSSC requires that foundations be a sufficient depth to provide horizontal setback from an ascending slope exhibiting gradients in excess of 1H:1V. As stated therein, the required setback shall be measured by assigning "*…the toe of the slope shall be assumed to be at the intersection of a horizontal plane drawn from the top of the foundation and a plane drawn tangent to the slope at an angle of 45 degrees to the horizontal*." Using this criterion, and the provided topographic plan (prepared by others), we approximated the minimum setback by providing an overlay (orange shading) within the attached Supplemental Site Plan, Figure 4. For preliminary planning, we recommend buildings be setback beyond the overlay shown on the attached Figure 4. If final layout includes placing building(s) within this overlay, the geotechnical engineer should be consulted to review the proposed construction.

8.8 Foundation Setback from Descending Slopes

Section 1808.7.2 of the 2010 OSSC requires that foundations be a sufficient depth to provide horizontal setback from a descending slope exhibiting gradients in excess of 1H:1V. As stated therein, "*…the required setback shall be measured from an imaginary plane 45 degrees to the horizontal, projected upward from the toe of the slope*." Using this criterion, and the provided topographic plan (prepared by others), we approximated the minimum setback from the west sidewall of the pit by providing an overlay

(red shading) within the attached Supplemental Site Plan, Figure 4. For preliminary planning, we recommend buildings be setback beyond the overlay shown on the attached Figure 4. If final layout includes placing building(s) within this overlay, the geotechnical engineer should be consulted to review the proposed construction.

9.0 PRELIMINARY RECOMMENDATIONS: PAVEMENT DESIGN

The following recommendations are provided assuming the native silty sand (SM) is encountered at design subgrade elevation for new pavements. As mentioned previously, portions of the site containing relatively deep, uncontrolled fill will require special consideration for developing subgrade support of new pavements. In these cases, the geotechnical engineer should be consulted to develop specific supplemental recommendations for pavements once layout and grading plans are being developed.

9.1 Subgrade Preparation

After site stripping as recommended above, but prior to placement of base course material or structural fill, the prepared native subgrade should be scarified to a depth of 12 inches below design subgrade elevation, moisture-conditioned to +/- 2 percent of optimum moisture content, and re-compacted with suitable equipment. The subgrade should be compacted to not less than 95 percent of the material's maximum dry density as determined by ASTM D1557 (Modified Proctor). The geotechnical engineer or his representative should perform in-place density testing of the compacted subgrade to confirm proper compaction and moisture-conditioning. In addition, a proof roll test of the compacted subgrade should be performed with a fully-loaded, 10- to 12-cubic yard, dump truck (or equivalent loaded water truck) in order to identify areas of excessive yielding. The geotechnical engineer or his representative should witness the proof roll test(s). If areas of soft soil or excessive yielding are identified, the affected material should be over-excavated to firm, stable subgrade, and replaced with imported granular structural fill in conformance with Section 8.4.3.2 of this report. Pavement subgrade surfaces should be crowned (or sloped) for proper drainage in accordance with specifications provided by the project civil engineer.

9.2 Flexible Pavements

9.2.1 Input Parameters

Our pavement section designs were based on the American Association of State Highway and Transportation Officials (AASHTO) 1993 "Design of Pavement Structures" manual. A number of design assumptions and variables were required in order to develop design sections for pavements proposed at the site. The following table presents the input parameters assumed for the design:

Table 11: Input Parameters Used in Asphalt Pavement Design

Input Parameter	Design Value[1]		Input Parameter		Design Value[1]
Pavement Design Life	20 years		Resilient Modulus	Subgrade[3]	6,000 psi
Annual Percent Growth	0 percent			Crushed Aggregate Base	22,500 psi
Serviceability	4.2 initial, 2.2 terminal		Structural Coefficient	Crushed Aggregate Base	0.10
Reliability	85 percent			Asphalt	0.42
Standard Deviation	0.49		Vehicle Traffic[4] (range in ESAL)	APAO Level I (Very Light)	Less than 10,000
Drainage Factor[2]	1.0			APAO Level II (Light)	Less than 50,000

1 If any of the above parameters are incorrect, please contact us so that we may revise our recommendations, if warranted.

2 Assumes good drainage away from pavement, base, and subgrade is achieved by proper crowning of subgrades.

3 Values based on experience with similar soils and assumes subgrade is prepared in conformance with Section 9.1 of this report.

4 ESAL = Total 18-Kip equivalent single axle load. Traffic levels taken from Table 3.1 of APAO manual. If actual traffic levels will be above those identified above, the geotechnical engineer should be consulted.

9.2.2 Recommended Minimum Sections

The following table presents the minimum asphalt pavement sections for various traffic loads indicated in the preceding table, based on the referenced AASHTO procedures.

Table 12: Recommended Minimum Asphalt Pavement Sections

Material	APAO Traffic Loading	
	Level I (Passenger Car Parking)	Level II (Entrance/Service Drive Lanes)
Asphalt Pavement (inches)	3	3½
Crushed Aggregate Base (inches)[1]	6	8
Subgrade Soils	Prepared in conformance with Section 9.1 of this report.	

1 Thickness shown assumes dry weather construction. A granular sub-base section and/or a geotextile separation fabric may be required in wet conditions in order to support construction traffic and protect the subgrade. Refer to Section 8.3 for additional discussion.

Asphalt pavement and base course material should conform to the most current State of Oregon, Standard Specifications for Highway Construction. Place aggregate base in one lift, and compact to not less than 95 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor). Asphalt pavement should be compacted to at least 91 percent of the material's theoretical maximum density as determined in general accordance with ASTM D2041 (Rice Specific Gravity).

9.3 Rigid (Concrete) Pavements

9.3.1 Input Parameters

Design of the rigid pavement sections presented below was based on the assumed parameters presented in the following table and the referenced AASHTO design manual. If any of the items listed need revision, please contact us and we will reassess the provided design sections. Jointing, reinforcement, and surface finish should be performed in accordance with the project civil engineer, architect, and owner requirements.

Table 13: Input Parameters Used in Concrete Pavement Design

Parameter / Discussion		Design Value
Standard Deviation		0.39
Load Transfer Devices incorporated?		Yes; Load Transfer Coefficient = 3.2
Minimum Concrete Modulus of Rupture		600 psi
Concrete Elastic Modulus		5.0 x 10^6 psi
Minimum Air-Entrained Concrete Comp. Strength		4,000 psi
Vehicle Traffic[1] (range)	APAO Level I (Very Light)	Less than 10,000 ESAL
	APAO Level II (Light)	Less than 50,000 ESAL

[1] ESAL = Total 18-Kip equivalent single axle load. If actual traffic levels will be above those identified above, the geotechnical engineer should be consulted.

9.3.2 Recommended Minimum Sections

The following table presents the recommended minimum concrete pavement sections based on the referenced AASHTO procedures.

Table 14: Recommended Minimum Concrete Pavement Sections

Material	APAO Traffic Loading	
	Level I (Passenger Car Traffic Only)	Level II (Entrance/Service Drive Lanes)
Portland Cement Concrete, PCC[1] (inches)	5	6
Leveling Course, Sand or All-Weather Base[2,3] (inches)	2	2
Subgrade Soils	Prepared in conformance with Section 9.1 of this report	

[1] Concrete strength and other properties should be in conformance with Table 13 above.
[2] Leveling course thickness should be a minimum of four times the maximum particle size. Example. If crushed rock up to ¾ inch in diameter is used, the leveling course should be at least 3 inches thick.
[3] Assumes dry weather construction. Increased base rock sections and/or a geotextile separation fabric may be required in wet conditions in order to support construction traffic and protect the subgrade. Refer to Section 8.3 for additional discussion

10.0 PRELIMINARY RECOMMENDATIONS: STRUCTURAL DESIGN

The following recommendations are provided assuming the native silty sand (SM), native tuff (RX), or the native Bend Pumice (RX) are encountered at design subgrade elevation for new foundations. As mentioned previously, portions of the site containing relatively deep, uncontrolled fill will require special consideration for developing subgrade support of new buildings. In these cases, the geotechnical engineer should be consulted to develop specific supplemental recommendations for foundations and floor slabs once layout and grading plans are being developed.

10.1 Shallow Spread Foundations

10.1.1 Subgrade Preparation

Satisfactory subgrade support for shallow foundations can be obtained from the native, medium dense to better, silty sand (SM), the native tuff (RX), the native Bend Pumice (RX), or structural fill that is properly placed and compacted on these materials during construction. The geotechnical engineer or his representative should be contacted to observe subgrade conditions prior to placement of forms, reinforcement steel, or structural backfill (if required). If soft, loose, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with imported granular structural fill in conformance with Section 8.4.3.2 of this report. The maximum particle size of foundation granular pads should be limited to 1½ inches. All granular pads for footings should be constructed a minimum of 6 inches wider on each side of the footing for every vertical foot of over-excavation.

10.1.2 Minimum Footing Width & Embedment

Minimum footing widths should be in conformance with the most recent, Oregon Structural Specialty Code (OSSC). Individual spread footings should have a minimum width of 24 inches. Subject to review of the structural engineer, we recommend continuous wall footings have a minimum width of 18 inches. To help mitigate potential frost action, all perimeter footings should be founded a minimum of 18 inches below the lowest adjacent grade. Interior footings should be founded a minimum of 12 inches below the interior surfacing element (e.g. concrete slab).

10.1.3 Bearing Pressure & Settlement

Footings founded as recommended above should be proportioned for a maximum allowable soil bearing pressure of 2,000 pounds per square foot (psf). This bearing pressure is a net bearing pressure, applies to the total of dead and long-term live loads, and may be increased by one-third when considering seismic or wind loads. For foundations founded as recommended above, and assumed maximum loads indicated in Section 1.1, total settlement of foundations is anticipated to be less than 1½ inches. Differential settlements between adjacent columns and/or bearing walls should not exceed ¾ inch. If an increased soil bearing pressure is desired, and/or the estimated foundation settlements need to be reduced, the geotechnical engineer should be consulted.

10.1.4 Lateral Capacity

A maximum passive (equivalent fluid) earth pressure of 150 pounds per cubic foot (pcf) is recommended for design of footings confined by the native soils described above, or imported granular structural fill that is properly placed and compacted during construction. The recommended earth pressure was computed using a factor of safety of 1½, which is appropriate due to the amount of movement required to develop full passive resistance. In order to develop the above capacity, the following should be understood:

1. Concrete must be poured neat in excavations or the foundations must be backfilled with imported granular structural fill,
2. The adjacent grade must be level,

3. The static ground water level must remain below the base of the footings throughout the year.
4. Adjacent floor slabs, pavements, or the upper 12-inch-depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

An ultimate coefficient of friction equal to 0.35 may be used when calculating resistance to sliding for footings founded on the native soils described above. An ultimate coefficient of friction equal to 0.45 may be used when calculating resistance to sliding for footings founded on a minimum of 6 inches of imported granular structural fill (crushed rock) that is properly placed and compacted during construction.

10.1.5 Subsurface Drainage

Placement of perimeter foundation drains is recommended at the base elevations of continuous wall footings on the outside of footings. Foundation drains should consist of a minimum 4-inch-diameter, perforated, HDPE (High Density Polyethylene) drainpipe wrapped with a non-woven geotextile filter fabric. The drains should be backfilled with a minimum of 2 cubic feet of open graded drain rock per lineal foot of pipe. The drain rock should be encased in a geotextile filter fabric in order to provide separation from the surrounding soils. Foundation drains should be positively sloped and should outlet to a suitable discharge point. The geotechnical engineer or his representative should be contacted to observe the drains prior to backfilling. Roof drains should not be tied into foundation drains.

10.2 Shallow Mat Foundations

10.2.1 Subgrade Preparation

Satisfactory static subgrade support for shallow mat foundations can be achieved by a minimum of 6 inches of imported granular fill ("crushed rock base") placed on the native, medium dense to better, silty sand (SM), the native Tumalo Tuff (RX/SM), or structural fill that is properly placed and compacted on these materials during construction. The crushed rock base is recommended to provide a more uniform surface for placing concrete and supporting the mat foundation. The crushed rock base should be in conformance with Section 8.4.4 of this report and extend a minimum of 1-foot wider on each side of the mat foundation. The geotechnical engineer or his representative should observe foundation subgrade conditions prior to placement of the crushed rock base. If soft, loose, organic, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the geotechnical representative at the time of construction.

10.2.2 Minimum Embedment

To help mitigate potential frost action, mat foundations should be founded a minimum of 18 inches below the lowest, permanent, adjacent grade.

10.2.3 Allowable Soil Bearing Pressure

For the proposed construction (up to five-story, concrete-framed buildings), we anticipate the maximum uniform contact pressure (from dead and long-term live loads) acting on the respective mat foundation will be less than 1,000 pounds per square foot (psf). This value may be considered the recommended maximum allowable soil bearing pressure for use in preliminary design. This bearing pressure is a net bearing pressure, applies to the total of dead and long-term live loads, and may be increased by ⅓ when

considering seismic or wind loads. If an increased allowable soil bearing pressure is desired, the geotechnical engineer should be consulted.

10.2.4 Modulus of Subgrade Reaction

For mat foundations founded as recommended above, a modulus of subgrade reaction up to 150 pci may be used for design. If an increased subgrade modulus is desired, the geotechnical engineer should be consulted.

10.2.5 Lateral Capacity

The recommendations presented in Section 10.1.4 are applicable for mat foundations confined by the native soils described above or imported granular structural fill that is properly placed and compacted during construction. An ultimate coefficient of friction equal to 0.45 may be used when calculating resistance to sliding for mat foundations founded as recommended.

10.2.6 Post-Construction Settlement (Static Loading)

For the recommended design bearing pressure, total post-construction settlement of mat foundations is anticipated to be less than 1 inch. Similarly, differential settlement (i.e. tilt) across uniformly-loaded mat foundations should not exceed ½ inch.

10.3 Floor Slabs & Exterior Hardscaping

10.3.1 Subgrade Preparation

Satisfactory subgrade support for slabs constructed on-grade, supporting up to 200 psf area loading, can be obtained from the native, medium dense to better, silty sand (SM), the native tuff (RX), the native Bend Pumice (RX), or structural fill that is properly placed and compacted on these materials during construction. The geotechnical engineer or his representative should observe foundation subgrade conditions prior to placement of the crushed rock base. If soft, loose, organic, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the geotechnical representative at the time of construction.

10.3.2 Crushed Rock Base

Concrete floor slabs should be supported on a minimum 6-inch-thick layer of crushed rock (base rock) in conformance with Section 8.4.4 of this report. For design cases where a vapor barrier or retarder is not placed below the slab, we recommend “choking” the surface of the base rock with fine sand just prior to concrete placement. Choking means the voids between the largest aggregate particles are filled with sand, but does not provide a layer of sand above the base rock. Choking the base rock surface reduces the lateral restraint on the bottom of the concrete during curing.

10.3.3 Design Considerations

For floor slabs constructed as recommended, a modulus of subgrade reaction of 150 pounds per cubic inch (pci) is recommended for the design of the floor slab. Floor slabs constructed as recommended will

likely settle less than ½-inch. For general floor slab construction, slabs should be jointed around columns and walls to permit slabs and foundations to settle differentially.

10.3.4 Subgrade Moisture Considerations

Liquid moisture and moisture vapor may be encountered at the subgrade surface. The recommended crushed rock base is anticipated to provide protection against liquid moisture. Where moisture vapor emission through the slab must be minimized, e.g. impervious floor coverings, storage of moisture sensitive materials directly on the slab surface, etc., a vapor retarding membrane or vapor barrier below the slab should be considered. Factors such as cost, special considerations for construction, floor coverings, and end use suggest that the decision regarding a vapor retarding membrane or vapor barrier be made by the architect and owner.

If a vapor retarder or vapor barrier is placed below the slab, its location should be based on current American Concrete Institute (ACI) guidelines, ACI 302 Guide for Concrete Floor and Slab Construction. In some cases, this indicates placement of concrete directly on the vapor retarder or barrier. Please note that the placement of concrete directly on impervious membranes increases the risk of plastic shrinkage cracking and slab curling in the concrete. Construction practices to reduce or eliminate such risk, as described in ACI 302, should be employed during concrete placement.

10.4 Seismic Design

As indicated in Section 6.2 of this report, the site was assigned as Site Class D. Earthquake ground motion parameters for the site were obtained based on the United States Geological Survey (USGS) Seismic Design Values for Buildings - Ground Motion Parameter Calculator[2]. The site Latitude 44.043421° North and Longitude -121.338768° West were input as the site location. The following table shows the recommended seismic design parameters for the site per Section 1613.5 of the 2010 OSSC.

Table 15: Seismic Ground Motion Values

Parameter		Value[1]
Mapped Acceleration Parameters	Spectral Acceleration, 0.2 second (S_s)	0.395g
	Spectral Acceleration, 1.0 second (S_1)	0.165g
Coefficients (Site Class D)	Site Coefficient, 0.2 sec. (F_A)	1.484
	Site Coefficient, 1.0 sec. (F_V)	2.139
Adjusted MCE Spectral Response Parameters	MCE Spectral Acceleration, 0.2 sec. (S_{MS})	0.587g
	MCE Spectral Acceleration, 1.0 sec. (S_{M1})	0.353g
Design Spectral Response Accelerations	Design Spectral Acceleration, 0.2 seconds (S_{DS})	0.391g
	Design Spectral Acceleration, 1.0 second (S_{D1})	0.236g

[1]Value presented for design under 2010 OSSC and is subject to change with building code updates. The geotechnical engineer should be consulted to finalize recommendations for seismic design at the time of building design.

[2] United States Geological Survey, 2014. Seismic Design Parameters determined using:, "U.S. Seismic Design Maps Web Application - Version 3.1.0," from the USGS website http://earthquake.usgs.gov.

11.0 RECOMMENDED ADDITIONAL SERVICES

11.1 Design Review

Geotechnical design review of project plans is of paramount importance, particularly for large or complex projects. As indicated previously, we recommend the geotechnical engineer be consulted to review project plans as they are being developed to provide supplemental recommendations for design and construction.

11.2 Observation of Construction

Satisfactory earthwork, foundation, floor slab, and pavement performance depends to a large degree on the 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 subsurface explorations, and recognition of changed conditions often requires experience. We recommend qualified personnel visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those observed to date and anticipated in this report.

We recommend the geotechnical engineer or his representative attend a pre-construction meeting coordinated by the contractor and/or owner. The geotechnical engineer or his representative should provide observations and/or testing of at least the following earthwork elements during construction:

- Site Stripping and Grubbing
- Subgrade Preparation for Structural Fills, Shallow Foundations, Floor Slabs, and Pavements
- Compaction of Structural Fill, Foundation Backfill, and Utility Trench Backfill
- Placement of Foundation Drains
- Compaction of Floor Slab Base Rock and Pavement Base Rock
- Compaction of Asphaltic Concrete for Pavements

It is imperative that the owner and/or contractor request earthwork observations and testing at a frequency sufficient to allow the geotechnical engineer to provide a final letter of compliance for the earthwork activities.

12.0 LIMITATIONS & CLOSURE

We have prepared this report for use by OSU Cascades and other members of the design and construction team for the proposed development. The opinions and recommendations contained within this report are not intended to be, nor should they be construed as, a warranty of subsurface conditions, but are forwarded to assist in the planning and design process.

We have made observations based on our explorations that indicate the soil conditions at only those specific locations and only to the depths penetrated. These observations do not necessarily reflect soil types, strata thickness, or water level variations that may exist between or away from those explorations.

If subsurface conditions vary from those encountered in the site explorations, CGT should be alerted to the change in conditions so that we may provide additional geotechnical recommendations, if necessary. Observation by experienced geotechnical personnel should be considered an integral part of the construction process.

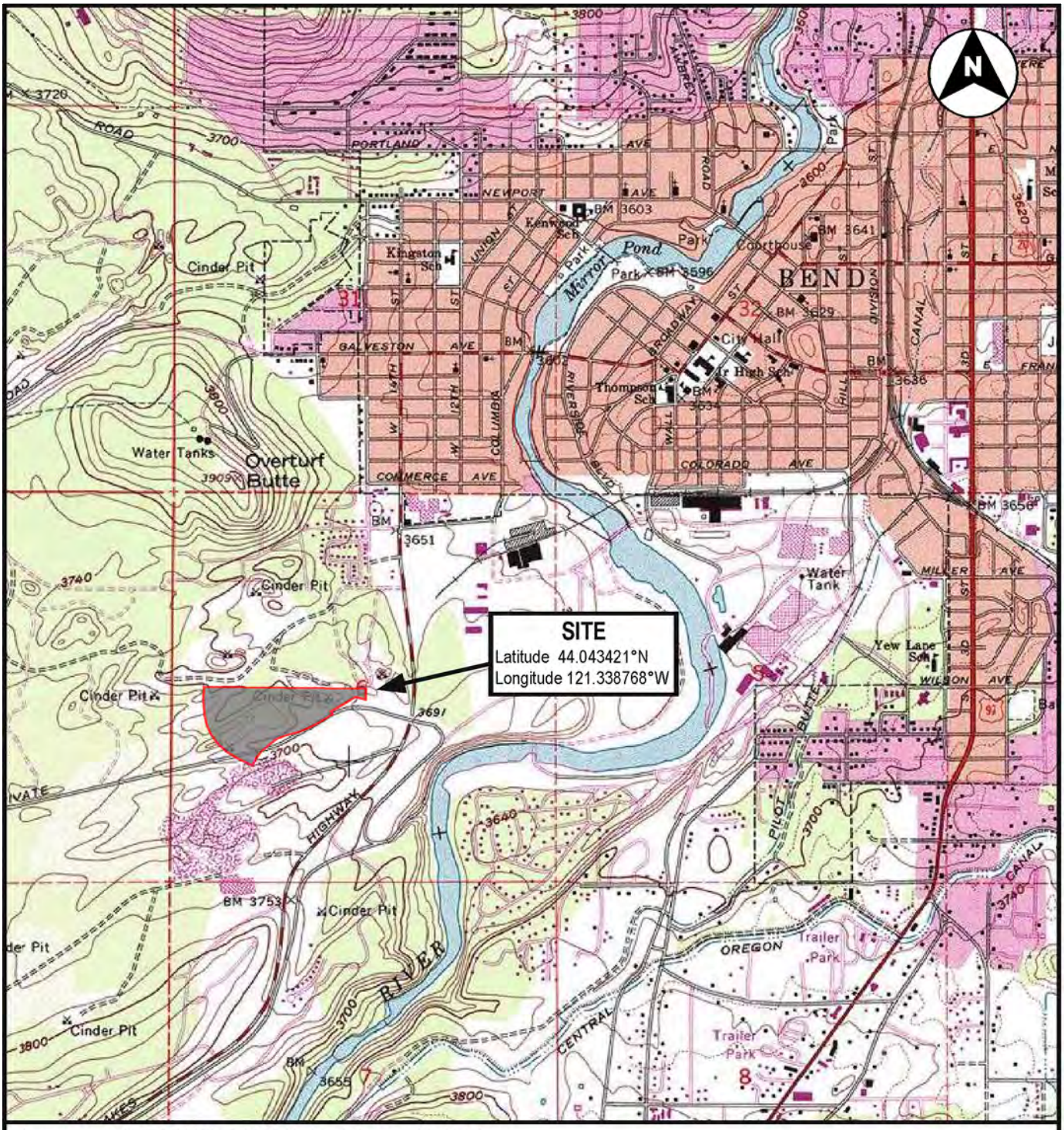
The owner is responsible for insuring that the project designers and contractors implement our recommendations. When the design has been finalized, prior to releasing bid packets to contractors, we recommend that the design drawings and specifications be reviewed by our firm to see that our recommendations have been interpreted and implemented as intended. 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. Design review is beyond the scope of our current assignment, but can be provided for an additional fee.

The scope of our services 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.

Geotechnical engineering and the geologic sciences are characterized by a degree of uncertainty. Professional judgments presented in this report are based on our understanding of the proposed construction, familiarity with similar projects in the area, and on general experience. Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared; no warranty, expressed or implied, is made. This report is subject to review and should not be relied upon after a period of 3 years.

SITE LOCATION

OSU CASCADES 46-ACRE SITE - BEND, OREGON



Map created with TOPO!™, © 2006 National Geographic Holdings
USGS 7.5 Minute Topographic Map Series, Bend, Oregon Quadrangle.

Approximate Scale: 1 inch = 2,000 feet

0 2000 4000

Township 18 South, Range 12 East, Section 6 Willamette Meridian

CARLSON CGT GEOTECHNICAL 503-601-8250

Carlson Geotechnical
P.O. Box 23814
Tigard, Oregon 97281

CGT Job No. G1303959.A

FIGURE 1