Geotechnical Engineering Services Report

DAS Executive Building Renovation
155 Cottage Street Northeast
Salem, Oregon

for
Oregon Department of Administrative Services
c/o Otak, Inc.

May 4, 2022

GeoEngineers
Earth Science + Technology

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File No. 11681-015-00

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1.0 INTRODUCTION

GeoEngineers, Inc. (GeoEngineers) is pleased to submit this geotechnical engineering report for the proposed Department of Administrative Services (DAS) Executive Building Renovation project. The project site is located at 155 Cottage Street NE in Salem, Oregon. The project site is currently developed with existing DAS facilities, generally consisting of an executive building and associated parking garage. The executive building and parking garage extend partially below-grade. An ingress/egress roadway to the lower level of the garage traverses the site between buildings.

Our understanding of the project is based on discussions with the design team. Based on information provided to us, the proposed project consists of site upgrades and remodeling to the existing facilities. Specific improvements include a complete interior renovation of the existing office building, opening up a garden level with additional windows and associated grading and/or retaining walls, and seismic retrofit upgrades.

2.0 SCOPE OF SERVICES

We were provided a copy of the geotechnical investigation requirements prepared by KPFF (project structural engineer), dated January 21, 2022. The purpose of our geotechnical engineering services was to advance subsurface explorations and develop an understanding of soil and groundwater conditions at the project site. Based on conditions encountered in the explorations, we provide geotechnical recommendations and earthwork considerations in this report to support project planning and design. In general, our authorized services included: reviewing selected geotechnical information about the site; advancing subsurface explorations; collecting representative soil samples; completing laboratory testing on selected soil samples; completing geotechnical analyses; and preparing this geotechnical report with our conclusions, findings and recommendations.

Our services have been provided in accordance with our subconsultant agreement with Otak, Inc. dated February 17, 2022 and authorized March 1, 2022.

3.0 SITE CONDITIONS

3.1. Vicinity and Site Limits

The project site is located in downtown Salem, Oregon and shown relative to surrounding physical features on the Vicinity Map, Figure 1 and Site Plan, Figure 2. Predominant natural features in the vicinity consist of several open waterways, including Willamette River (approximately 0.5 miles west), Mill Creek (approximately 0.5 miles north), Mill Stream (approximately 0.2 miles south) and Pringle Creek (approximately 0.35 miles south). The site is generally surrounded with multi-story urban structures (several of which are known to have at least one-story basements), broad city streets, parking and sidewalk areas, and open urban park areas. The site is directly west of the Oregon State Capitol building grounds.

The overall site encompasses one square city block, totaling about 2.65 acres. The site is rectangular in shape and bounded by Church Street NE, State Street, Cottage Street NE and Court Street NE.

3.2. Current Site Conditions

The project site is currently developed with existing DAS facilities, generally consisting of the executive building and an associated parking garage. The existing executive building is a multi-story masonry structure in the western portion of the site with a below-grade level (basement) relative to street level. The existing parking garage is a two-level concrete structure in the eastern portion of the site with a parking deck above surrounding grade and one level of parking below surrounding street grade. An ingress/egress roadway to the lower level of the garage traverses the site between buildings. The low point of the access roadway also allows access to the Executive Building on its west side. Site structures are surrounded by concrete sidewalks, lawn and landscaped islands (including grasses, shrubs and trees), asphalt-paved parking and drive areas, and asphalt-paved city streets. A loading dock drive is located on the north side of the Executive Building.

Based on observations while on site and a review of topography available online, overall grades at the site and extending west towards the Willamette River are relatively level. Roadway elevations surrounding the site vary between about Elevation (EL) 156 and 158 feet. Immediately east of the site, grades in the vicinity generally slope down (from southeast to northwest) towards the Willamette River at an average slope of about 1 percent (100H:1V [horizontal to vertical]) or flatter.

Elevations and ground surface contours referenced in this report were obtained through United States Geologic Survey (USGS) topographic data available online. Elevations refer to North American Vertical Datum of 1988 (NAVD88) and should be considered approximate.

3.3. Literature Review

3.3.1. Geologic Setting

The project site and surrounding downtown Salem vicinity are within the floodplains of the Willamette River and its surrounding tributaries. We reviewed published geologic maps of the project area, including maps by O'Connor et. al. (2001) and Bela (1981). For visual reference, a portion of the O'Connor et. al. (2001) published geologic map is reproduced in this report as Geologic Map, Figure 3.

Based on our review, the project site and surrounding vicinity are mapped as underlain by alluvial sand and gravel deposits. Typical thickness of these deposits varies from about 30 feet to 100 feet or more. The upper foot or so of gravel is noted as compacted and including a fine-grained matrix in the gravel which restricts drainage.

Although not indicated on the reviewed geologic maps, our experience in the downtown Salem area suggests that the uppermost layers of these gravels are often mantled by young silty to sandy alluvium of the Willamette and Santiam Rivers. These silty mantles can be on the order of several feet to several tens of feet thick. Additionally, much of the Salem urban core has also been extensively altered by human activities, including placement of a variable thickness of soil or debris fills.

Other geologic deposits mapped in the vicinity consist of variable unconsolidated clay, silt, sand and gravel originating from the Willamette River and its tributaries (alluvial deposits) or originating from flood deposits of glacial Lake Missoula, which flowed down the Columbia River and backflooded into the Willamette Valley (flood deposits). Upland areas surrounding the Salem vicinity are generally mapped as bedrock consisting of Columbia River Basalt Group or marine sedimentary rocks.

3.3.2. Natural Resources Conservation Service (NRCS) Description

The United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Web Soil Survey (WSS) provides soil survey data and information for local and wide area planning, as well as for general conservation and engineering applications. For visual reference, a map of soil types in the project vicinity as mapped by the NRCS WSS (accessed March 22, 2022) is reproduced in this report as Web Soil Survey - Soil Type, Figure 4.

The site is mapped as underlain by Woodburn Silt Loam, described as deposited in broad valley terraces formed in indistinctly stratified alluvium or lacustrine. This soil type is also described as moderately well drained, with slow to medium runoff and moderately slow permeability.

Other soils mapped in the immediate project vicinity consist of variable alluvial deposits. A summary of soil types and hazards in the project vicinity (as described in the Web Soil Survey) are provided in the table below.

TABLE 1. SUMMARY NRCS SOIL SURVEY

Soil Unit Name	Map Symbol	Parent Material	Permeability	Hydrologic Soil Group
Mapped at Project Site				
Woodburn silt loam	WuA	Silty alluvium	Moderately slow	C
Mapped in Surrounding Areas				
Amity silt loam	Am	Mixed silty alluvium	Moderately slow	C/D
Clackamas gravelly loam	Ck	Gravelly mixed alluvium	Moderately slow	C/D
Cloquato silt loam	Cm	Alluvium	Moderate	B
Salem gravelly silt loam	Sa	Gravelly mixed alluvium	Moderately high	B
Willamette silt loam	WlA	Silty alluvium	Moderately slow	B

3.3.3. Published Boring Logs and Groundwater Well Data

We reviewed online databases from the Oregon State Water Resources Department and USGS National Water Information System (NWIS) for soil and groundwater data in the project vicinity. The typical subsurface profile described in reviewed boring and well logs consisted of "top soil" and/or clay within the upper 9 to 12 feet, underlain by variable sand and gravel. Clay is occasionally noted present within the sand and gravel layers, and intermittent discrete clay layers are also noted. Top of bedrock, consisting of basalt or conglomerate, ranged from about 48 to 106 feet below ground surface (bgs) in the reviewed logs. Groundwater depths were typically recorded at depths between about 13.5 and 32 feet bgs, and occasionally as deep as 64.5 feet bgs.

A summary of reviewed borings and groundwater well data in the project vicinity is provided in Table 2 below.

TABLE 2. SUMMARY OF NEARBY BORINGS AND WELL DATA

Approximate Location (Distance Relative to Site)	Well ID	Water Level Reading (ft bgs)	Date of Boring / Well Reading(s)	Typical Generalized Soil Description (ft bgs)
700 State Street (500 feet SE)	MARI 20705	13.5	11/9/57	0 to 12 Soil 12 to 26 Sand & gravel w/ clay 12 to 48 Sand & Gravel, occ. w/ Clay 48 to 72 Weathered Basalt
350 Winter Street NE (1,000 feet NE)	MARI 8108	16.5 to 32 (typ.) 64.5 (one reading)	2/28/63 to 10/13/81 (intermittent)	0 to 9 Unknown 9 to 27 Gravel & boulders 27 to 106 Sand & Gravel, occ. w/ Clay 106 to 130 Rock & conglomerate
155 Waverly Street NE (1,700 feet ESE)	MARI 8091	18.3	11/12/81	0 to 1 Top soil 1 to 12 Clay 12 to 66 Gravel w/ sand & boulders 66 to 98 Conglomerate

3.4. Subsurface Conditions

3.4.1. Methodology

We completed field explorations at the site by advancing four drilled borings (B-1 through B-4) on March 4, 2022. Borings were advanced to depths between about 8 and 21.5 feet bgs. Representative soil samples from the borings were returned to our laboratory for examination and testing. Detailed descriptions of our site exploration and laboratory-testing programs, along with exploration logs and laboratory test results, are presented in Appendix A. Approximate locations of the explorations are shown on the Site Plan, Figure 2.

3.4.2. Soil Conditions

Two borings were advanced through asphalt pavement: Boring B-2 near the central area of the site in the north loading dock area and Boring B-3 along State Street SE near the southeast portion of the site. Boring B-1, originally planned for the Court Street SE right-of-way, was relocated at the time of drilling to the lawn area northwest of the building to avoid underground utilities. Boring B-4 was advanced in the southwest lawn area of the site in order to perform an infiltration test as discussed in later sections of this report.

The State Street pavement section consisted of approximately 5 inches asphalt pavement underlain by 13 inches aggregate base course, while the loading dock approach section consisted of 2 inches of asphalt over 6 to 8 inches of aggregate base.

Sod thickness in the lawn areas explored by B-1 and B-4 was observed to be about 3 inches thick. We anticipate sod/duff depths vary across the site and may be greater than those encountered at the exploration locations. Larger diameter roots and stumps are expected in treed areas of the site.

Shallow soil conditions vary across the project site. To the south (B-3 and B-4), very loose to medium dense silty sand with variable gravel and occasional debris was encountered to depths of about 5 to 7.5 feet bgs. Based on variable soil layering and debris observed, we interpret this material to be fill. To the north (B-1 and B-2), soft to stiff clay was encountered to depths of about 5 to 7.5 feet bgs. We interpret this clay material to be fine-grained alluvial deposits.

Below depths of about 5 to 7.5 feet bgs, borings encountered medium dense to very dense gravel with variable silt and sand. We interpret these soils to be coarse-grained alluvial deposits.

Boring B-4 was terminated within fill deposits at a depth of about 7.5 feet bgs. Remaining borings were terminated within coarse-grained alluvial deposits at depths of about 21.5 feet bgs

3.4.3. Groundwater

Wet soils and groundwater were encountered in all four borings at depths ranging from about 8 to 10 feet bgs during drilling (corresponding to approximate EL 149 to 148 feet). We also observed occasional soil coloring, as noted in the logs, which may indicate presence and fluctuations of groundwater at various times of the year. We interpret this to be perched groundwater atop relatively impermeable clay soils.

In general, we anticipate that groundwater levels will fluctuate throughout the year and in response to precipitation events. We expect groundwater levels will typically be at the highest during the winter and spring months.

Based on our review and experience in the area we anticipate perched groundwater could be encountered in the upper 5 to 10 feet, which consists of relatively fine-grained silts and silty sands. Dewatering of trenches and excavations will be required when perched groundwater is encountered. We anticipate typical static groundwater levels fluctuate between about 15 to 30 feet bgs, corresponding to approximate EL 142 to 127 feet. We do not expect that seasonal high static groundwater levels will be above about 15 feet bgs (EL 142).

4.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

4.1. Seismic Design Considerations

4.1.1. Site-Specific Seismic Hazard Evaluation

GeoEngineers completed a site-specific seismic hazard evaluation for the site. Based on the results of the site-specific evaluation, we recommend that code level seismic design parameters be used for this project. Complete results of the study are presented in Appendix B of this report. The sections below provide a summary of recommended seismic design considerations.

4.1.2. Seismic Design Parameters

Seismic design of the proposed improvements will be completed using procedures outlined in American Society of Civil Engineers (ASCE) 41-17. In accordance with ASCE 41-17, performance objectives are determined based on equivalent to new building standards or for existing buildings. For the designated performance objective, seismic design shall consider two earthquake levels:

4.1.2.1. BPON (Basic Performance Objective Equivalent to New Building Standards)

- BSE-1N: Basic Safety Earthquake-1, taken as two-thirds of the BSE-2N at a site.
- BSE-2N: Basic Safety Earthquake-2, taken as the ground shaking based on the Risk-Targeted Maximum Considered Earthquake (MCER) at a site.

4.1.2.2. BPOE (Basic Performance Objective for Existing Buildings)

- **BSE-1E**: Basic Safety Earthquake-1, taken as a seismic hazard with a 20% probability of exceedance in 50 years (225-year return period)
- **BSE-2E**: Basic Safety Earthquake-2, taken as a seismic hazard with a 5% probability of exceedance in 50 years (975-year return period)

Per ASCE 41-17, seismic ground motion parameters and response spectra records are determined in accordance with ASCE 7-16. We understand proposed improvements will include new foundation elements near existing footing grades, anticipated to be on the order of 10 feet bgs relative to surrounding street level. Based on soils encountered in the borings, reviewed boring logs in the vicinity and our experience we anticipate soils extending below 10 feet bgs typically consist of dense to very dense sand and gravel deposits. Therefore, we recommend the site is classified Site Class C in accordance with ASCE 7-16.

Recommended seismic design parameters for code level seismic design in accordance with ASCE 41-17 (and ASCE 7-16) are presented in Table 3 below and consider Risk Category II and Site Class C.

TABLE 3. RECOMMENDED ASCE 41-17 SEISMIC DESIGN PARAMETERS[1]

ASCE 41-17 (ASCE 7-16) Seismic Design Parameter[2]	Recommended Value[1]			
	BSE-1N (2/3*BSE-2N)	BSE-2N (MCER) (2,475-year)	BSE-1E (225-year)	BSE-2E (975-year)
Mapped Spectral Response Acceleration at Short Period (0.2 second) (S_S)	n/a	0.826 g	0.195 g	0.584 g
Mapped Spectral Response Acceleration at 1 second Period (S_1)	n/a	0.414 g	0.075 g	0.288 g
Site Amplification Factor at 0.2 second period (F_a)	n/a	1.2	1.3	1.266
Site Amplification Factor at 1.0 second period (F_v)	n/a	1.5	1.5	1.5
Site Adjusted Spectral Response Acceleration at Short Period (0.2 second) (S_{XS})	0.660 g	0.991 g	0.254 g	0.740 g
Site Adjusted Spectral Response Acceleration at 1 second Period (S_{X1})	0.414 g	0.621 g	0.113 g	0.432 g
Design Spectral Acceleration at 0.2 second period (S_{DS})	0.440 g	0.661 g	0.169 g	0.493 g
Design Spectral Acceleration at 1.0 second period (S_{D1})	0.276 g	0.414 g	0.075 g	0.288 g
Site Modified Peak Ground Acceleration (PGA_M)[3]	0.317 g	0.476 g	0.132 g	0.280 g

Notes:

[1] Parameters developed based on Latitude 44.9393356° and Longitude -123.0341537°

[2] Parameters developed based on Risk Category II and Site Class C

[3] Non-modified peak ground acceleration (PGA) is calculated as 0.4 times the site adjusted short-period response acceleration (S_{XS})

Recommended response spectra for BPON and BPOE earthquake events are provided in Figures 5 and 6.

4.1.3. Liquefaction Potential

4.1.3.1. General

Liquefaction refers to a condition where vibration or shaking of the ground, usually from earthquake forces, disturbs the soil structure (i.e., the arrangement of individual soil particles) within saturated and unconsolidated soils. Water in the pore spaces between soil particles will resist the natural tendency of the soils to re-arrange into a denser and more stable state during shaking, resulting in the development of excess porewater pressures. As porewater pressures increase, soil particles may lose contact with each other, and the affected soil deposit may lose much of its stiffness and strength. Liquefaction susceptibility is difficult to predict and not all soils are susceptible to liquefaction. The degree of susceptibility depends in part on the soil grain size. In general, soils most susceptible to liquefaction include loose to medium dense "clean" to silty sands below the water table. However, research and case histories indicate other loose granular soils, such as low-plasticity silts and gravels may also be susceptible to liquefaction.

Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures, such as buildings, supported on or within liquefied soils may suffer loss of bearing capacity, foundation settlement and/or lateral movement that can be damaging to the buildings.

4.1.3.2. Estimated Liquefaction Potential

We evaluated the potential for liquefaction using semi-empirical liquefaction triggering correlations based on standard penetration tests (SPTs) completed in the borings per Idriss and Boulanger (2014). Our liquefaction analyses considered a groundwater depth of 8 feet, site modified peak ground acceleration (PGA_M) equal to 0.461 g and earthquake magnitude of 8.3 (consistent with BSE-2N).

Based on our analyses, it is our opinion the potential for liquefaction and liquefaction-induced settlement at the site is very low.

4.2. Shallow Foundation Support

4.2.1. General

Existing buildings at the site (executive office building and parking garage) extend one-level below surrounding street level. We understand proposed renovations will include new foundation elements near existing footing grade, anticipated to be on the order of 10 feet bgs relative to surrounding street level. Based on soils encountered in the borings, reviewed boring logs in the vicinity and our experience, we anticipate soils extending below 10 feet bgs typically consist of dense to very dense sand and gravel deposits.

Based on our understanding of the proposed improvements and anticipated soil conditions, proposed structures can be satisfactorily founded on continuous strip or isolated column footings supported on stiff/dense on-site soils, or on structural fill placed over firm and unyielding soils. Detailed recommendations for shallow foundation support are provided in the sections below.

4.2.2. Footing Bearing Surface Preparation and Overexcavation

It is our understanding that new foundation elements will not bear on the upper fine-grained soils and will bear on the underlying dense gravels. If new foundations or pads are anticipated to bear at depths shallower than about 8 feet bgs, we recommend foundations not bear directly on relatively soft/loose soils (including fill and/or alluvial deposits); some removal and replacement could be required. Foundation bearing surfaces must be confirmed to be dense in place or compacted, as necessary, to a uniformly firm and unyielding condition prior to foundation construction. For footings founded at depths where soft soils are present, we recommend removing the unsuitable material to firm native soils and backfilling with structural fill to footing subgrade elevation. Overexcavation should extend 1 foot laterally from the outside edge of the footing for every 2 feet in depth.

Excavations should be performed using a smooth-edged bucket to limit bearing surface disturbance. Loose or disturbed materials present at the base of footing excavations must be removed, re-compacted or otherwise repaired. If soft or otherwise unsuitable areas are revealed during evaluation that cannot be compacted to a stable and uniformly firm condition, the following options may be considered: (1) unsuitable soils be moisture-conditioned and recompacted; (2) unsuitable soils be overexcavated and replaced with compacted structural fill, as needed; or (3) it may be possible to push, seat and compact quarry spalls into soft soils to stabilize the surface. These options will have to be evaluated on a case-by-case basis.

Prepared bearing surfaces and excavations should be evaluated by GeoEngineers during construction (prior to placement of structural fill or formwork and reinforcement) to confirm bearing surfaces have been prepared in accordance with our recommendations, subsurface conditions are as assumed, and to provide recommendations to stabilize or repair bearing surfaces.

4.2.3. Foundation Design Parameters

4.2.3.1. Minimum Footing Dimensions

Exterior footings should be established at least 18 inches below the lowest adjacent finished grade. Interior footings should be founded a minimum of 12 inches below the top of the floor slab. These minimum foundation depths are recommended, in part, to be greater than the anticipated frost depth.

We recommend continuous footings have a minimum width of 18 inches and isolated column footings have a minimum width of 24 inches, or as needed, to meet allowable design bearing pressures for the design loads.

4.2.3.2. Allowable Soil Bearing Pressure

Based on conditions encountered in the borings and our experience in the project vicinity, near-surface soils (fine-grained alluvium and fill) are typically less dense than underlying dense to very dense coarse-grained alluvium deposits. Given this potential for variability, we provide recommended allowable bearing capacities recommendations for the typical soil profiles encountered. Allowable bearing pressures below are used in conventional foundation design in order to limit settlement of supported structures. Ultimate bearing capacities are also considered in evaluation of existing structures per ASCE 41-17 as discussed below.

- **For footings supported on proof compacted near surface medium stiff to stiff clay, medium dense to dense fill, compacted structural fill overlying firm on-site soils, or spanning between relatively loose and dense soils:** For footings prepared as described above, we recommend an allowable soil bearing pressure of 2,500 pounds per square foot (psf) be used for design.
- **Footings supported on dense native gravel deposits (or on compacted structural fill extending to these deposits):** For footings prepared as described above, we recommend an allowable soil bearing pressure of 4,000 psf be used for design. For reference, these dense gravel deposits were encountered in the borings at depths below about 5 to 7.5 feet bgs, corresponding to approximate EL 150 to 151 feet.

These recommended allowable bearing pressures apply to the total of dead and long-term live loads. When considering total loads (e.g., earthquake or wind loads), the allowable bearing pressures above may be increased by one-third in the upper fine-grained soils and by one-half in the underlying dense gravel. These are net bearing pressures. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

ASCE 41-17 design procedures consider ultimate bearing resistances. The recommended allowable bearing pressures provided above consider a factor of safety (FS) of about 2 compared to the ultimate resistance. Ultimate bearing capacity can be calculated from the above allowable bearing capacities in each of the soil profiles discussed by multiplying by 2.

4.2.3.3. Foundation Settlement Estimates

We estimate long-term settlements of footings designed and constructed as recommended will be on the order of 1 inch or less, with differential settlements of about half this amount between comparably loaded isolated column footings or along 50 to 100 feet of continuous footing. Settlement is expected to occur rapidly, essentially as loads are applied. Settlements could be greater than estimated if loose or disturbed soil is present beneath footings.

To minimize the risk of unpredictable foundation settlement in the more variable near surface soils, footing bearing elevations should extend to the underlying dense native gravel deposits (or on compacted structural fill extending to these deposits). This will have the added advantage of a higher recommended bearing pressure to offset additional excavation costs.

4.2.4. Lateral Resistance

The ability of the soil to resist lateral loads is a function of frictional resistance, which can develop on the base of footings and slabs, and the passive resistance, which can develop on the face of below-grade elements of the structure as these elements tend to move into the soil.

We recommend using an allowable friction coefficient of 0.45 for foundations placed on native dense gravels, or 0.50 for foundations over compacted crushed rock as recommended in this report.

We recommend that passive earth pressures on the face of the footing or other embedded foundation elements be calculated using an equivalent fluid unit weight of 300 pounds per cubic foot (pcf) foundations confined by proof-compacted native soils consisting of medium stiff or stiffer silt. Equivalent fluid unit weight can be increased to 310 pcf if footings are confined by imported granular fill extending out from the face of the foundation element a horizontal distance at least equal to 3 times the depth of the footing.

These values include a factor of safety of about 1.5. The passive earth pressure and friction components may be combined, provided the passive earth pressure component does not exceed two-thirds of the total. The passive earth pressure value is based on the assumptions that the adjacent grade is level and groundwater levels remain below the base of the footing throughout the year. The top foot of soil should be neglected when calculating passive lateral earth pressure unless the area adjacent to the foundation is covered with pavement or slab-on-grade.

4.2.5. Foundation Drains

In general, we recommend the ground surface be sloped away from the buildings at least 2 percent. All downspouts should be tightlined away from the building foundation areas and should also be discharged into a stormwater disposal system. Downspouts should not be connected to footing drains.

If grades or landscape needs require grading toward the building as is the case in some areas of the site as presently developed, drain paths for surface water in the form of grading and catch basins with appropriately routed discharge should be incorporated into the site civil design plan such that water is not directed toward the building and foundations. This may include surface drainage routes and catch points, long catch basins or v-ditches at the base of slopes that route surface water away from structures.

Based on our interpretation of the regional groundwater table and groundwater conditions observed in our explorations, it is our opinion footing drains are not necessary to maintain bearing support for footings at the anticipated depths (about 10 feet bgs). If deeper elements (such as elevator pits, utility vaults, etc.) are installed below the expected level of groundwater, we recommend foundation drains be installed. Active dewatering or tightline routing of draining water will be required during wet times of the year at these locations in order to provide a removal pathway.

Although not necessary to maintain bearing support, we typically recommend footing drains be incorporated into project plans to maintain drier conditions around the structures and for long-term maintenance and management of near-surface water seeping in around the structures. This may also reduce the risk of potential impact of perched groundwater on moisture sensitive flooring by some flooring manufacturers or specific floor types. Our recommendation is based on the potential for near-surface seepage during wetter times of the year or from irrigation/landscaping activities and observed soil conditions.

We can provide specific recommendations for design of foundation drains, if requested, once final structure locations and elevations have been determined.

4.3. Slab-On-Grade Floors

4.3.1. General

In our opinion, satisfactory subgrade support for floor slab supporting up to 100 psf or less can be obtained, provided the floor slabs are prepared as recommended in this report.

4.3.2. Slab-on-Grade Subgrade Preparation

Exposed slab subgrades should be evaluated after site grading is complete. Depending on final building elevations, subgrade soils could be variable and range from soft/loose to stiff/dense. Disturbed areas beneath slabs-on-grade should be compacted, if possible, or removed and replaced with compacted structural fill. Sod, organic-rich soils, existing pavements, hardscaping or other structural elements must be removed from within building footprint(s). Existing fill may be considered for slab subgrade, provided the bearing surface is confirmed or compacted as necessary.

In all cases, the exposed soil should be proof compacted to a dense, firm and unyielding condition. Areas of isolated loose or organic-rich areas may be encountered and require some overexcavation and replacement.

4.3.3. Slab-on-Grade Design Parameters

A modulus of subgrade reaction of 200 pounds per cubic inch (pci) can be used for designing building floor slabs, provided the slab subgrade consists of firm and unyielding existing soils or compacted structural fill and has been prepared in accordance with recommendations presented in this report. This value is for a 1-foot by 1-foot square plate. The coefficient of subgrade reaction for a foundation varies based on its minimum width according to the following equation:

$$k_s = k_{s1}[(B+1)/2B]^2$$

Where k_s is the coefficient of subgrade reaction, k_{s1} is the coefficient of subgrade reaction for a 1-foot by 1-foot plate, and B is the minimum width or lateral dimension of the mat.

We recommend slab-on-grade floors be underlain by a minimum 6-inch-thick compacted crushed rock section to provide a capillary break, uniform bearing support and an adequate modulus of subgrade reaction as noted above. The base section material should conform to recommendations provided in Section 9.0 Fill Materials of this report and should be placed as recommended in Section 9.100 Fill Placement and Compaction.

Long-term static (non-seismic) settlement for floor slabs designed and constructed as recommended is estimated to be less than 0.5 inch for a floor load of 100 psf. We estimate differential settlement of floor slabs will be ½ inch or less over a span of 50 feet.

If dry slabs are required (e.g., where adhesives are used to anchor carpet or tile to the slab or for other moisture-sensitive situations), a waterproof liner may be placed as a vapor barrier below the slab. The vapor barrier should be selected by the structural engineer and should be accounted for in the design floor section and mix design selection for the concrete, to accommodate the effect of the vapor barrier on concrete slab curing.

Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations.

4.4. Conventional Retaining Walls

4.4.1. General

Proposed renovations include opening up a garden level in the existing office building with additional windows. Depending on final site grading, retaining walls could be required to maintain site grades.

We recommend GeoEngineers be retained to review the retaining wall design provided by others to confirm design meets the requirements provided in this report. The retaining wall designer should provide a total wall design including global stability analysis of the proposed wall, and not include global stability analysis as a requirement to be performed by the project geotechnical engineer.

4.4.2. Assumptions

Retaining wall design recommendations provided below are provided for the site based on soils encountered on site during our explorations and available sources of structural fill in the area and include the following assumptions. If these assumptions are not appropriate, we should be contacted to provide revised recommendations, including determination if a more detailed global stability analysis is required.

- Walls are less than about 8 feet tall in total wall support height. Total wall heights are determined based on a level front slope from the base of the wall.
- Walls will not be tiered.
- Walls will not be located above slopes steeper than about 4H:1V.
- Walls will not support building loads.
- The backfill within 2 feet of the wall consists of free-draining granular materials.
- Grades above the top of the walls are no steeper than a 2H:1V slope.
- Drainage will be provided behind the wall to prevent buildup of hydrostatic pressures. Recommended lateral earth pressures do not include hydrostatic forces.

4.4.3. Recommended Lateral Earth Pressures

We recommend subsurface structures be designed using the lateral earth pressures below. Structures that are free to deflect at least 0.001H, where H is the height of the structure, can be designed for active earth pressures. If the subsurface structures are restrained against deflection (not allowed to rotate), we recommend they be designed for an at-rest earth pressure. These values do not include hydrostatic forces.

Lateral earth pressures presented in this section for retaining walls assume conventional cantilevered retention of wall loads, that backfill placed within 2 feet of the wall is compacted by hand-operated equipment to a density of 92 percent of the maximum dry density (MDD), and that wall drainage measures are included as noted below. For walls constructed as described above and with a maximum retaining height of 8 feet (for practical purposes – taller walls should use braced or other support for efficiency in design/cost of construction), we recommend using an active lateral earth pressure corresponding to an equivalent fluid density of 35 pcf for the level backfill condition. For walls with backfill sloping upward behind the wall at 2H:1V, an equivalent fluid density of 55 pcf should be used. If the slope is shallower than 2H:1V, the active lateral earth pressures can be linearly interpolated between the two values above. This assumes that the tops of the walls are not structurally restrained and are free to rotate.

For the at-rest condition (walls restrained from movement at the top), an equivalent fluid density of 60 pcf for level conditions and 75 pcf for a 2H:1V slope behind the wall, should be used for design. For seismic conditions, we recommend a uniform lateral pressure of 9H (where H is the height of the wall) psf be added to these lateral pressures. If the retaining system is designed as a braced system but is expected to yield a small amount during a seismic event, an active earth pressure condition may be assumed and combined with the uniform seismic surcharge pressure.

The recommended pressures do not include the effects of surcharges from surface loads. If vehicles will be operated within one-half the height of the wall, a traffic surcharge should be added to the wall pressure. The traffic surcharge can be approximated by the equivalent weight of an additional 2 feet of backfill behind the wall. Additional surcharge loading conditions should also be considered on a case-by-case basis.

The above soil pressures assume that wall drains will be installed to prevent the buildup of hydrostatic pressure behind the walls, as discussed in the paragraphs below.

4.4.4. Drainage

Positive drainage is imperative behind retaining walls unless they are designed to resist hydrostatic forces. We recommend a zone of free-draining material behind the retaining structure with perforated pipes to collect seepage water. Most of the site soils encountered in our explorations contain a significant percentage of fines (material passing the U.S. No. 200 sieve). Fine soils are susceptible to particle migration, potentially clogging the drainage.

We recommend a zone of free-draining granular material be included behind the retaining structure. A filter fabric designed for separation should be placed between the gravel backfill and native site soils to prevent soil migration. Waffle boards or drainage mats specifically designed for this application are also appropriate.

A perforated, smooth-walled, rigid polyvinyl chloride (PVC) pipe with a minimum diameter of 4 inches should be placed at the bottom of the drainage zone along the entire length of the retaining structure with the pipe invert at or below the elevation of the base of the footing. The drainpipes should collect water and direct it to a tightline leading to an appropriate disposal system. Cleanouts should be incorporated into the design of the drains in order to provide access for regular maintenance. Roof downspouts, perimeter drains, or other types of drainage systems must not be connected to drain systems for retaining walls or below-grade structures.

We can provide additional drainage zone recommendations upon request.

4.4.5. Other Design Parameters

Footings for retaining structures should be designed in accordance with Section 4.2 Shallow Foundation Support of this report. Frictional wall resistance, including coefficient of friction along the base of the wall and allowable passive resistance on the face of the wall, should be computed using recommendations provided in Subsection 4.2.4 Lateral Resistance. We estimate settlement of retaining structures will be similar to the values previously presented for building foundations.

Fill material placed behind retaining walls should be placed and compacted as described in Sections 9.0 and 9.10 of this report.

5.0 INFILTRATION

5.1. General Feasibility Assessment

In general, we recommend on-site stormwater infiltration be considered provided adequate separation can be provided between the bottom of facility and high groundwater or impermeable layers.

- Near surface fine-grained alluvial soils (clays) were observed in borings B-1 and B-2 in the northern site area to depths of about 5 to 7 feet bgs. Reviewed well logs in the project vicinity note “top soil” and clay within the upper 9 to 12 feet bgs. We recommend clay be considered impermeable for infiltration purposes.

- Near surface existing fill (variable silty sand) was observed in borings B-3 and B-4 to depths of about 5 to 7 feet bgs. Infiltration test IT-1 was completed in boring B-4 at a depth of 3 feet bgs; results are presented in the sections below.
- Below depths of about 5 to 7.5 feet bgs, we interpret soil conditions at the site to consist of coarse-grained alluvial deposits (gravel with variable silt and sand). We anticipate similar or increased infiltration performance within these sand and gravel layers (compared to testing completed in IT-1).
- We anticipate typical static groundwater levels at the site fluctuate between about 15 to 30 feet bgs, corresponding to approximate EL 142 to 127 feet. Depending on the depth of proposed facility, maintaining required separation below the bottom of facility to high groundwater levels could limit design.

5.2. Infiltration Testing

We completed one infiltration test at the site to assist in the evaluation of stormwater infiltration design at the site. One test (IT-1) was completed within boring B-4 at an approximate depth of 3 feet bgs, corresponding to Elevation 154 feet. Approximate location of the test is shown on Figure 2.

Infiltration testing was completed using the encased falling head procedures in general accordance with "Division 004" of the City of Salem Department of Public Works Administrative Rules Design Standards (COSDS). A layer of pea gravel was placed in the pipe prior to adding water to diminish disturbance from flowing water at the base of the pipe interior. The test area was pre-soaked over a 4-hour period by addition of water into the pipe when necessary. In our opinion, a good seal was present between the base of the pipe and the underlying. After the saturation period, the pipe was filled with clean water to at least 12 inches above the bottom of the pipe placed in the boring. The drop-in water level was measured over a period of 1 hour after the soak period. Water in the pipe was refilled to repeat the test a minimum of three times. In the case where the water level falls during the time-measured testing, infiltration rates diminish as a result of less head from the water column in the test.

We measured average infiltration rates of about 14 to 15 inches per hour (in/hr) during testing. Field test results are summarized in Table 4.

TABLE 4. INFILTRATION TEST RESULTS

Infiltration Test No.	Location	Depth (feet)	USCS[1] Material Type	Field Measured Infiltration Rate[2] (inches/hour)
IT-1	SW corner of site (see Site Plan)	3	SM (Silty Fine to Medium Sand)	15

Notes:

[1] USCS = Unified Soil Classification System

[2] Appropriate factors should be applied to the field measured infiltration rate, based on the design methodology and specific system used.

The infiltration rate shown in the table above is a field-measured infiltration rate, in accordance with Division 004 of the COSDS. These reported rates represent a relatively short-term measured rate and factors of safety have not been applied for the type of infiltration system being considered, or for variability that may be present in the on-site soil. In our opinion, and consistent with the state of the practice, correction factors should be applied to this measured rate to reflect the small area of testing and the number of tests conducted.

Appropriate correction factors should be applied by the project civil engineer to account for long-term infiltration parameters. From a geotechnical perspective, we recommend a factor of safety (correction factor) of at least 2 be applied to this type of field infiltration testing result to account for potential soil variability with depth and location within the area tested. In addition, the stormwater system design engineer should determine and apply appropriate remaining correction factor values, or factors of safety, to account for repeated wetting and drying that occur in this area, degree of in-system filtration, frequency and type of system maintenance, vegetation, potential for siltation and bio-fouling, etc., as well as system design correction factors for overflow or redundancy, and base and facility size.

The actual depths, lateral extent and estimated infiltration rates can vary from the values presented above. Field testing/confirmation during construction is often required in large or long systems or other situations where soil conditions may vary within the area where the system is constructed. The results of this field testing might necessitate that the infiltration locations be modified to achieve the design infiltration rate.

The infiltration flow rate of a focused stormwater system typically diminishes over time as suspended solids and precipitates in the stormwater slowly clogging the void spaces between the soil particles or cake on the infiltration surface. The serviceable life of a stormwater system can be extended by pre-filtering or with on-going accessible maintenance. Eventually, most systems will fail and will need to be replaced or have media regenerated or replaced. We recommend that infiltration systems include an overflow that is connected to a suitable discharge point. Also, infiltration systems can cause localized high groundwater levels and should not be located near basement walls, retaining walls or other embedded structures unless these are specifically designed to account for the resulting hydrostatic pressure. Infiltration locations should not be located on sloping ground, unless it is approved by a geotechnical engineer, and should not be infiltrated at a location that allows for flow to travel laterally toward a slope face, such as a mounded water condition or too close to a slope face.

5.3. Suitability of Infiltration System

Successful design and implementation of stormwater infiltration systems and whether a system is suitable for a development depend on several site-specific factors. Stormwater infiltration systems are generally best suited for sites having sandy or gravelly soil with saturated hydraulic conductivities greater than 2 inches per hour. Sites with silty or clayey soil are generally not well-suited for long-term stormwater infiltration or as a sole method of stormwater infiltration. Soils that have fine-grained matrices are susceptible to volumetric change and softening during wetting and drying cycles. Fine-grained soils also have large variations in the magnitude of infiltration rates because of bedding and stratification that occurs during alluvial deposition, and often have thin layers of less permeable or impermeable soil within a larger layer.

Local groundwater conditions also significantly affect the capacity to infiltrate from a stormwater system. Sites with shallow groundwater, such as this site, can result in groundwater mounding. A hydraulic gradient that reaches the level of water in the soil immediately drops to zero and local groundwater will rise, and mound and the infiltration rate slows dramatically, resulting in overflows or system flooding (failure). Groundwater mounding can also negatively impact structures, slopes, or other areas adjacent to the stormwater infiltration facility. Typically, we do not recommend using infiltration systems where groundwater is less than 10 feet below the bottom of the proposed system unless the host soil is very permeable and consistently graded and will not cause mounding.

Most Oregon jurisdictions typically require a minimum of 5 to 10 feet between high groundwater conditions and the bottom of proposed facilities. Also, on-site soil infiltration capacities may be limited. These factors may be conditions where even a 5- to 10-foot separation between the level of groundwater and the base of the infiltration system may not be sufficient.

6.0 PAVEMENT RECOMMENDATIONS

6.1. General

Pavement recommendations provided below are appropriate for on-site pavements only and not for public roadways. If repaving areas of the site is required, we recommend pavement subgrades be prepared in accordance with the Site Development and Earthwork Recommendations Section 8.0 of this report.

Specific information on the frequency and type of vehicles that will use proposed pavements was not provided. We have based our design analysis on traffic consisting of 1,000 cars and up to five two- or three-axle delivery type trucks (UPS or FedEx type or garbage truck type vehicles) per day, and 10 five-axle vehicle transport delivery type trucks per week for heavy-duty pavement sections. Light duty pavement areas are considered those accessed only by auto traffic (i.e., parking areas) and delivery van type vehicles. Heavy-duty pavement areas include those within the drive path of heavy trucks and transport delivery vehicles such as loading docks. Both heavy and light duty pavement sections are capable of supporting fire engine and ladder type vehicles during emergency access situations not considered for daily use.

Heavy construction traffic has not been considered in our pavement design; therefore, we assume that pavement subgrade preparation and pavement construction will be completed at the end of the project after heavy construction vehicles, such as concrete trucks and construction material delivery trucks will no longer access the site. Construction traffic should not be allowed on new pavements. If this is not the case and areas are to be paved as part of site construction with heavy traffic, we will have to re-design the pavements for those heavier loading conditions.

6.2. Drainage

Long-term performance of pavements is influenced significantly by drainage conditions beneath the pavement section. Positive drainage can be accomplished by crowning the subgrade and establishing grades to promote drainage.

6.3. Pavement Sections

Our pavement recommendations are based on the following assumptions:

- Soil subgrade below the pavement section consists of recompacted native subgrade or structural fill over native subgrade, prepared as recommended in this report, including observations that indicate the native material is in a firm and unyielding condition.
- A resilient modulus of 3,800 pounds per square inch (psi) estimated for subgrade prepared as recommended.

- A resilient modulus of 20,000 psi was estimated for base rock.
- Initial and terminal serviceability indices of 4.2 and 2.0, respectively.
- Reliability and standard deviations of 75 percent and 0.45, respectively.
- Structural coefficients of 0.42 and 0.14 for the asphalt and base rock, respectively.
- Structural coefficient of 0.08 for cement-amended subgrade.
- A 20-year design life for asphalt concrete (AC) sections. A 40-year design life for PCC sections.
- Truck traffic consists of an even distribution of three- to five-axle trucks.

Based on the assumptions and estimated traffic noted above and our analyses, our recommended pavement sections for asphalt concrete (AC) and portland cement concrete (PCC) are presented in Table 5.

TABLE 5. RECOMMENDED PAVEMENT SECTIONS[1]

Section	Minimum PCC Thickness (inches)	Minimum AC Thickness (inches)	Minimum Aggregate Base Thickness (inches)
Light Duty (general automobile parking areas)	-	3.5	8.0
Heavy Duty (drive aisles and heavy delivery areas)	-	4.5	10.0
	5.0	-	6.0

Notes:

[1] See report text for assumed traffic loading considered.

The recommended pavement thickness in the light duty (car traffic only) areas may be modified to 3.5 inches of AC over 6.0 inches of base rock if pavement is to be placed in two lifts during construction. This allows for a 2-inch AC thickness to be placed prior to final placement of a 1.5-inch lift as a finished surface near the end of project construction. The base lift should be prepared with an appropriate tack coat and repaired where damaged during construction prior to placement of the finish lift.

Sidewalks and other concrete hardscapes should be provided with a minimum 4-inch-thick aggregate base section.

The crushed rock base course layer and subbase layer (the upper portion of in place soils) should be moisture-conditioned near the optimum moisture content and compacted to at least 95 percent of the MDD determined in accordance with American Association of State Highway and Transportation Officials (AASHTO) T-180 or ASTM International (ASTM) D 1557 test procedures. The subbase layer should be prepared during extended periods of dry weather or adequately amended as recommended in Section 8.4 Wet Weather Construction of this report. An appropriate number of in-place density tests should be conducted on the compacted subbase and base courses to check that adequate compaction has been obtained.

The AC pavement should conform to Section 00745 of the most current edition of the Oregon Department of Transportation (ODOT) Standard Specifications for Highway Construction. The Job Mix Formula should meet the requirements for a ½-inch Dense Graded Level 2 Mix. The AC binder should be PG 64-22 grade meeting the ODOT Standard Specifications for Asphalt Materials. AC pavement should be compacted to 91.0 percent of the Maximum Theoretical Unit Weight (Rice Gravity) as determined by AASHTO T-209.

For PCC pavements, paving concrete should be Class 4000 ¾-inch-minus with minimum 28-day flexural strength of 650 psi. A jointing plan should be provided in the project plans showing construction joints and transverse and longitudinal joints to control cracking consistent with American Concrete Pavement Association (ACPA) recommendations. Longitudinal joints typically coincide with PCC panel widths, and transverse joints are generally relatively equally spaced and close to the same spacing as longitudinal joints so that panels are relatively square. If limited panel width results in no longitudinal joints, the jointing plan should maintain a relatively square panel section based on the panel width.

The recommended pavement sections assume that final improvements surrounding the pavements will be designed and constructed such that stormwater or excess irrigation water from landscape areas does not infiltrate below the pavement section into the crushed base.

7.0 OTHER GEOTECHNICAL DESIGN CONSIDERATIONS

7.1. Frost Penetration

In our opinion, near-surface soils observed in the explorations are slightly to moderately susceptible to frost heave. Based on our review of local mapping, the depth of frost penetration in the project area is estimated to be less than 12 inches.

Overall, it is our opinion that recommendations provided in this report should allow adequate frost protection. We anticipate floor slab elements will bear on compacted granular fill as described in Section 4.3 Slab-On-Grade Floors. Recommended minimum footing embedment depths provided in Section 4.2 Shallow Foundation Support are greater than the anticipated frost depth. Frost susceptibility in pavement areas is also expected to be low if they are constructed and supported as recommended in this report.

7.2. Expansive Soils

Based on our laboratory test results and experience with similar soils in the area, we do not consider the soils encountered in our borings to be expansive.

8.0 SITE DEVELOPMENT AND EARTHWORK RECOMMENDATIONS

8.1. Earthwork Equipment and Demolition

We anticipate initial site preparation and earthwork activities on site will include clearing and stripping vegetated areas; demolition of existing hardscaping and foundations (if encountered); and site grading, including removal of unsuitable soils within proposed improvement areas. Based on the material encountered in our subsurface explorations, it is our opinion that conventional earthmoving equipment in proper working condition should be capable of completing expected site grading and excavations. The earthwork contractor should be responsible for reviewing this report, including the boring logs, providing their own assessments, and providing equipment and methods needed to excavate the site soils while protecting subgrades.

8.2. Site Preparation, Clearing and Demolition

Existing surfaces within proposed improvement areas should be cleared and stripped of all vegetation and organics prior to site development. We anticipate greater stripping depths could be required to remove localized zones of loose or organic-rich soil. During clearing and stripping, stumps and primary root systems of shrubs and trees should be completely removed. Voids caused by removal of stumps and/or root systems should be backfilled with compacted structural fill. Stripped material should be transported off site or processed and used as fill in landscaping areas.

In general, we recommend structural elements of previous buildings and pavements be demolished and removed from within the footprint of the new improvements. Abandoned utility lines larger than about 4 inches in diameter that are located beneath proposed structural areas should be completely removed or filled with grout if abandoned and left in place in order to reduce potential for settlement or caving in the future. Materials generated during demolition of existing improvements should be transported off site for disposal.

Existing voids and new depressions created during site preparation and/or resulting from demolition should be cleaned of loose soil or debris down to firm soil and backfilled with compacted structural fill. Disturbance to a greater depth should be expected if site preparation and earthwork are conducted during periods of wet weather.

8.3. Surface Features and Debris

Occasional debris was observed within near surface fill in portions of the site. Additional debris (e.g., metal, asphalt and/or concrete) may be present or buried in areas of the site not explored. The earthwork contractor should be prepared to encounter obstacles and debris in areas of the site to be regraded or excavated. If encountered, debris should be removed from within the footprint of the new improvements.

Although not encountered in our explorations, cobbles and/or boulders could be present in alluvial deposits. The contractor should be prepared for the presence of cobbles or boulders in areas to be excavated or re-graded. For uniform and adequate compaction, the contractor may be required to separate larger cobbles and boulders from soils generated during excavation if excavated soils are to be reused as structural fill or structural backfill. Boulders should be removed from the site or used in landscape areas. Voids caused by boulder removal should be backfilled with structural fill.

8.4. Wet Weather Construction

The upper fine-grained soils at the site surface are highly susceptible to disturbance from traffic when wet. Wet weather construction practices will be necessary if work is performed during periods of wet weather. If site grading will occur during wet weather conditions, it will be necessary to use track-mounted equipment, load removed material into trucks supported on existing pavements, use gravel working pads and employ other methods to reduce ground disturbance. The contractor should be responsible to protect the subgrade during construction.

During wet weather, some of the exposed soils could become muddy and unstable. If so affected, we recommend that:

- The ground surface in and around the work area should be sloped so that surface water is directed to a sump or discharge location. The ground surface should be graded such that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work areas.

- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.
- During periods of wet weather, concrete should be placed as soon as practical after preparing foundation excavations. Foundation bearing surfaces should not be exposed to standing water. Should water infiltrate and pool in the excavation, the water should be removed, and the foundation subgrade should be re-evaluated before placing reinforcing steel or concrete. Foundation subgrade protection, such as a 3- to 4-inch-thickness of aggregate base or lean concrete, may be necessary if footing excavations are exposed to extended wet weather conditions.

8.5. Subgrade Preparation and Evaluation

Specific subgrade preparation recommendations for shallow foundations and slabs-on-grade are presented in Sections 4.2 Shallow Foundation Support and 4.3 Slab-On-Grade Floors of this report, respectively.

Subgrades should be thoroughly compacted to a uniformly firm and unyielding condition upon completion of stripping and demolition, prior to placing fills or structures. We recommend prepared subgrades be evaluated to identify areas of yielding or soft soil. Probing with a steel probe rod or proof-rolling with a heavy piece of wheeled construction equipment are appropriate methods of evaluation.

If soft or otherwise unsuitable subgrade areas are revealed during evaluation that cannot be compacted to a stable and uniformly firm condition, we recommend: (1) the unsuitable soils be scarified (e.g., with a ripper or farmer's disc), aerated and recompacted, if practical; or (2) the unsuitable soils be removed and replaced with compacted structural fill, as needed.

8.6. Temporary Groundwater Handling

As discussed in Section 3.4.3 Groundwater of this report, we encountered what we interpret to be perched groundwater at about 8 to 10 feet bgs in the borings (EL 148 to 149 feet). We anticipate perched groundwater could be encountered in the upper 5 to 10 feet, which consists of relatively fine-grained silts and silty sands. Dewatering of trenches and excavations will be required when perched groundwater is encountered. We anticipate typical static groundwater levels fluctuate between about 15 to 30 feet bgs, corresponding to approximate EL 142 to 127 feet. Moderate to rapid groundwater seepage should be expected in excavations extending below the groundwater table.

In addition to groundwater seepage and upward groundwater flow into excavations, surface water inflow to the excavations during the wet season can be problematic. Provisions for surface water control and temporary dewatering during earthwork and excavations should be included in the project plans and should be installed prior to commencing earthwork.

The level of effort required for dewatering will depend to a great extent on the time of year during which construction is accomplished. We anticipate groundwater presence will vary throughout the year and will generally be highest during the wet season, typically October through May. Accordingly, groundwater handling needs will typically be lower during the late summer and early fall months. We recommend construction be completed in the late summer or early autumn months when the groundwater level is typically at its lowest elevation. Proactive handling of surface water (e.g., grading to reduce ponding, etc.) can reduce groundwater handling needs.

- Perched groundwater at relatively shallow depths is typically due to surface water that has recently infiltrated the ground surface. Controlling groundwater with sumps, pumps and/or diversion ditches may be adequate for relatively small shallow excavations that are only open for a short amount of time.
- Deeper excavations extending near static groundwater levels will likely require a more vigorous removal of water, possibly with additional pumps, sumps and/or storage. For deeper excavations extending below groundwater or large excavations required to be open for an extended period of time, dewatering using well points or other methods should be anticipated. Dewatering efforts such as well points or extraction wells and should be approached with caution as removal of large amounts of water over a short period of time can potentially induce settlement in the soils surrounding a dewatered excavation.

If sandy soil is present at the base of the excavation, dewatering should be extended deeper than the base to prevent upward inflow of water that will cause upward heaving from the base. This may also require a section of stabilization material (4-inch- or 2-inch-minus crushed rock) up to 2 feet thick to be placed at the base of the excavation as a working and firm foundation surface.

Some on-site soils grade with higher amounts of sand with depth and include interbedded lenses of sand. Based on the sandy nature of some site soils, inflowing water may cause a “running sand” condition where the water flows into the excavation with a sandy soil slurry that is difficult to pump with internal non-filtered pumps as well as causing significant sloughing of excavated sidewalls. Carefully planned and executed excavation will be required to maintain a manageable sized excavation when groundwater is present in sandy conditions.

We recommend the contractor performing the work be made responsible for developing a dewatering plan and controlling and collecting groundwater encountered. If dewatering plans are proposed, we recommend GeoEngineers review dewatering plans developed by the contractor.

8.7. Trench Excavations and Shoring

Based on soil types and densities encountered in our explorations, excavations at the site could experience caving, especially if excavations extend near or below areas of groundwater seepage. Excavations made to construct footings or other structural elements should be laid back or shored at the surface as necessary to provide safe working conditions and prevent soil from falling into excavations.

All excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. Site soils within expected excavation depths typically range from medium stiff to stiff silt to medium dense sand. Medium stiff to stiff silt soils should be classified as OSHA Soil Type B while loose to medium dense sand soils should be considered OSHA Soil Type C, provided there is no seepage and excavations occur during periods of dry weather.

Trench excavations deeper than 4 feet should be shored or laid back at an inclination of 1H:1V for Type B soils and 1.5H:1V for Type C soils. Flatter slopes may be necessary if workers are required to enter. These guidelines assume all surface loads are kept a minimum distance of at least one-half the depth of the cut away from the top of the slope and seepage is not present on the slope face. Flatter cut slopes will be necessary where seepage occurs or if surface surcharge loads are anticipated. Temporary covering with heavy plastic sheeting should be used to protect these slopes during periods of wet weather.

Shoring for trenches less than 6 feet deep that are above the effects of groundwater should be possible with a conventional box system. Moderate sloughing should be expected outside the box. Shoring deeper than 6 feet or below the groundwater table should be designed by a registered engineer before installation. Furthermore, the shoring design engineer should be provided with a copy of this report.

In our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to the soil and groundwater conditions. Construction site safety is generally the sole responsibility of the contractor, who also is solely responsible for the means, methods and sequencing of the construction operations, and choices regarding excavations and shoring. Under no circumstances should the information provided by GeoEngineers be interpreted to mean that GeoEngineers is assuming responsibility for construction site safety or the contractor’s activities; such responsibility is not being implied and should not be inferred.

8.8. Temporary Cut Slopes

In general, to maintain site grading and provide safe working conditions, we recommend temporary cut slopes be inclined no steeper than about 1.5H:1V. This guideline assumes all surface loads are kept a minimum distance of at least one-half the depth of the cut away from the top of the slope and seepage is not present on the slope face. Flatter cut slopes will be necessary where seepage occurs or if surface surcharge loads are anticipated. Temporary covering with heavy plastic sheeting should be used to protect these slopes during periods of wet weather.

For open cuts at the site, we recommend that:

- No construction traffic, construction equipment, stockpiles or building supplies be allowed within a distance of at least one-half the depth of the cut away from the top of the slope
- Exposed soil along the slope be protected from surface erosion using waterproof tarps or plastic sheeting, or flash coating with shotcrete.
- Construction activities be scheduled so that the length of time the temporary cut is left open is reduced to the extent practicable.
- Erosion control measures be implemented as appropriate such that runoff from the site is reduced to the extent practicable.
- Surface water be diverted away from the excavation.
- The general condition of the slopes be observed periodically by GeoEngineers.

Because the contractor has control of the construction operations, the contractor should be made responsible for the stability of cut slopes, as well as the safety of the excavations. All shoring and temporary slopes must conform to applicable local, state and federal safety regulations.

8.9. Permanent Cut and Fill Slopes

The proposed project consists of a complete interior renovation of the existing office building, including opening up a garden level with additional windows. Final grading could include permanent slopes from existing grade to the lower level of the office building or parking garage.

We recommend permanent slopes be constructed at a maximum inclination of 2H:1V. Where 2H:1V permanent slopes are not feasible, protective facings and/or retaining structures should be considered. Where access for landscape maintenance is desired, we recommend a maximum gradient of 3H:1V.

These guidelines assume all surface loads are kept at a minimum distance of at least one-half the height of the slope away from the top of the slope and seepage is not present on the slope face. Flatter cut slopes or additional drainage measures could be necessary where seepage occurs or if surface surcharge loads are anticipated.

Fill placement on existing slopes steeper than 5H:1V should be benched into the slope face. The configuration of benches depends on the equipment being used and the inclination of the existing slope. Bench excavations should be level and extend into the existing slope face at least half the width of the compaction equipment used. To achieve uniform compaction, we recommend fill slopes be overbuilt by at least 12 inches and subsequently cut back to expose well-compacted fill.

Exposed areas should be re-vegetated as soon as practical to reduce the surface erosion and sloughing. Temporary protection should be used until permanent protection is established. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope. We recommend GeoEngineers review proposed grading plans when they become available to confirm our recommendations are appropriate and have been incorporated into project design.

9.0 FILL MATERIALS

9.1. General

Our recommendations for fill materials are presented below. We recommend GeoEngineers review contractor submittals for earthwork materials to be used on site.

Material used for fill must be free of clays, debris, organic contaminants and rock fragments larger than 3 inches. The workability of material for use as fill will depend on the gradation and moisture content of the soil. As the percentage of fines increases, fill materials become increasingly sensitive to changes in moisture. Typically, soil containing more than about 5 percent fines is more sensitive to changes in moisture and will become difficult to compact when just a few percent above the optimum moisture content.

If earthwork occurs during a typical wet season, or if the soils are persistently wet and cannot be dried back due to prevailing wet weather conditions, we recommend the use of imported structural fill or select granular fill. Other fill materials such as crushed rock, permeable ballast or quarry spalls, may also be used during wet weather. Budgets should include provisions for import granular fill, especially if construction is planned during the wet weather season. We can provide specific recommendations for imported material specific for its intended use and based on the time of year of construction, once site development and planning is scheduled.

9.2. Structural Fill

For imported material, we recommend crushed rock or select granular fill (described below) be used for structural fill during the wet season. If prolonged dry weather prevails during the earthwork phase of construction, materials with a somewhat higher fines content (up to about 12 percent passing the U.S. No. 200 sieve) may be acceptable.

9.3. Select Granular Fill

Imported select granular material may be used as structural fill. Imported Select Structural Fill should consist of pit or quarry run rock, crushed rock, or crushed gravel and sand that is fairly well-graded between coarse and fine sizes, with approximately 25 to 65 percent passing the U.S. No. 4 sieve. It should have less than 5 percent passing the U.S. No. 200 sieve and have a minimum of two mechanically fractured faces. During dry weather, the fines content can be increased to a maximum of 12 percent.

9.4. On-Site Soils as Structural Fill

An experienced geotechnical engineer from GeoEngineers should determine the suitability of on-site soil encountered during earthwork activities for reuse as structural fill.

In general, on-site soils can be considered for use as structural fill provided material is adequately moisture conditioned and compacted as recommended in this report. On-site material must also be removed of particles greater than 3 inches in size, debris and organics. Based on our experience, near surface silts and silty sands encountered in our explorations are moisture sensitive and will be very difficult or impossible to properly compact when wet.

It is possible existing soils will be excavated at moisture contents above optimum moisture content and/or within groundwater seepage areas. For reference, moisture content test results are presented on the test pit logs for selected samples from our explorations. We recommend on-site soils only be considered for use as structural fill and backfill provided the material:

- Has a maximum particle size of 3 inches,
- Is used during only extended periods of dry weather,
- Can be adequately moisture-conditioned and placed and compacted as recommended, which may include limiting use to dry times of the year,
- Does not contain debris, organic matter or other deleterious material, and
- Meets any special requirements related to its end use.

9.5. Topsoil Strippings

Topsoil strippings may be placed on site, provided they are placed in non-structural areas that can tolerate some long-term total and differential settlements. Settlements of organic-rich soils are highly variable and difficult to quantify. Settlement could continue for several years after construction is completed as the organics break down and decompose. Alternatively, topsoil strippings can be hauled off site.

9.6. Aggregate Base

Aggregate base material located under floor slabs and pavements, and crushed rock used in footing overexcavations, should consist of imported clean, durable, crushed angular rock. Aggregate base material should be crushed and processed rock material that is well-graded, have a maximum particle size of 1 inch and have less than 5 percent passing the U.S. No. 200 sieve. In addition, aggregate base shall have a minimum of 75 percent fractured particles according to AASHTO TP-61 and a sand equivalent of not less than 30 percent based on AASHTO T-176.

Processed, recycled material from the demolition of concrete or AC may be used as general site fill but may not be used as a one-to-one substitute for crushed rock as an aggregate base.

9.7. Trench Backfill

Backfill for pipe bedding and in the pipe zone should consist of well-graded granular material with a maximum particle size of ¾ inch and less than 5 percent passing the U.S. No. 200 sieve. Trench backfill material should be free of organic matter and other deleterious materials. Further, the backfill should meet the pipe manufacturer's recommendations. Above the pipe zone, Imported Select Granular Fill may be used as described above.

Other materials may be required depending on pipe manufacturer specifications and/or jurisdictional requirements where utilities extend off the property and into the public right-of-way.

9.8. Retaining Wall Backfill

Fill placed to provide a drainage zone behind retaining walls should consist of free-draining sand and gravel or crushed rock with a maximum particle size of ¾ inch and less than 3 percent passing the U.S. No. 200 sieve.

9.9. Recycled Materials

Recycled materials such as AC, cement concrete and over-sized materials may be used as structural fill, provided the recommendations below are followed.

- **Processed Recycled Materials:** AC, cement concrete or over-sized rock can be considered for use as structural fill, provided the material is processed by crushing and screening, grinding in place or other methods to meet the structural fill recommendations in this report. Recycled asphalt may not be used at any depth within the building footprint. Other materials such as cement concrete or over-sized rock may be used as structural fill in all areas except within 3 feet beneath foundations.
- **Unprocessed Recycled Materials:** AC, cement concrete or over-sized rock fragments that have a maximum particle size of 4 inches in nominal diameter may be mixed with on-site soil or imported fill to create a generally uniform, well-graded material and used in pavement and landscape areas. If used beneath pavements, we recommend at least 2 feet of processed structural fill overlie the unprocessed recycled fill material.

9.10. Fill Placement and Compaction

Structural fill should be compacted at moisture contents that are within 3 percent of the optimum moisture content as determined by ASTM Standard Practices Test Method D 1557 (Modified Proctor). The optimum moisture content varies with gradation and should be evaluated during construction. Fill material that is not near the optimum moisture content should be moisture-conditioned prior to compaction.

Fill and backfill material should be placed in uniform, horizontal lifts and compacted with appropriate equipment. The appropriate lift thickness will vary depending on the material and compaction equipment used. Fill material should be compacted in accordance with Table 6 below. It is the contractor's responsibility to select appropriate compaction equipment and place the material in lifts that are thin enough to meet these compaction criteria. However, in no case should the loose lift thickness exceed 12 inches.

TABLE 6. RECOMMENDED FILL COMPACTION CRITERIA

Fill Type	Compaction Requirements Percent MDD Determined by ASTM Test Method D 1557 at ± 3% of Optimum Moisture		
	0 to 2 Feet Below Subgrade	> 2 Feet Below Subgrade	Pipe Zone
Fine-grained site soils (non-expansive)	92	92	----
Imported Granular, maximum particle size < 1¼ inch	95	95	----
Imported Granular, maximum particle size 1¼ inch to 3 inches	n/a (proof-roll)	n/a (proof-roll)	----
Retaining Wall Backfill*	92	92	-----
Nonstructural Zones	90	90	90
Trench Backfill	95	90	90

Notes:

*Measures should be taken to prevent overcompaction of the backfill behind retaining walls. We recommend placing the zone of backfill located within 5 feet of the wall in lifts not exceeding about 6 inches in loose thickness and compacting this zone with hand-operated equipment such as a vibrating plate compactor and a jumping jack.

During fill and backfill placement, material should be regularly evaluated to verify adequate compaction is being achieved. A representative from GeoEngineers should evaluate compaction of each lift of fill. Compaction should be evaluated by compaction testing, unless other methods are proposed for oversized materials and are approved by GeoEngineers during construction. These other methods typically involve procedural placement and compaction specifications together with verifying requirements such as proof-rolling.

10.0 DESIGN REVIEW AND CONSTRUCTION SERVICES

Recommendations provided in this report are based on the assumptions and design information stated herein. We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GeoEngineers should be retained to review the geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in this report.

Satisfactory foundation and earthwork performance depend to a large degree on quality of construction. Sufficient monitoring 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 explorations. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

We recommend that GeoEngineers be retained to observe construction at the site to confirm that subsurface conditions are consistent with the site explorations and to confirm that the intent of project plans and specifications relating to earthwork, pavement and foundation construction are being met.

11.0 LIMITATIONS

We have prepared this report for the exclusive use of the Oregon Department of Administrative Services (DAS), Otak, Inc., and their authorized agents and/or regulatory agencies for the DAS Executive Building Renovation project in Salem, Oregon. This report is not intended for use by others and the information contained herein is not applicable to other sites. No other party may rely on the product of our services unless we agree in advance and in writing to such reliance.

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

Please refer to Appendix C “Report Limitations and Guidelines for Use” for additional information pertaining to the use of this report.

12.0 REFERENCES

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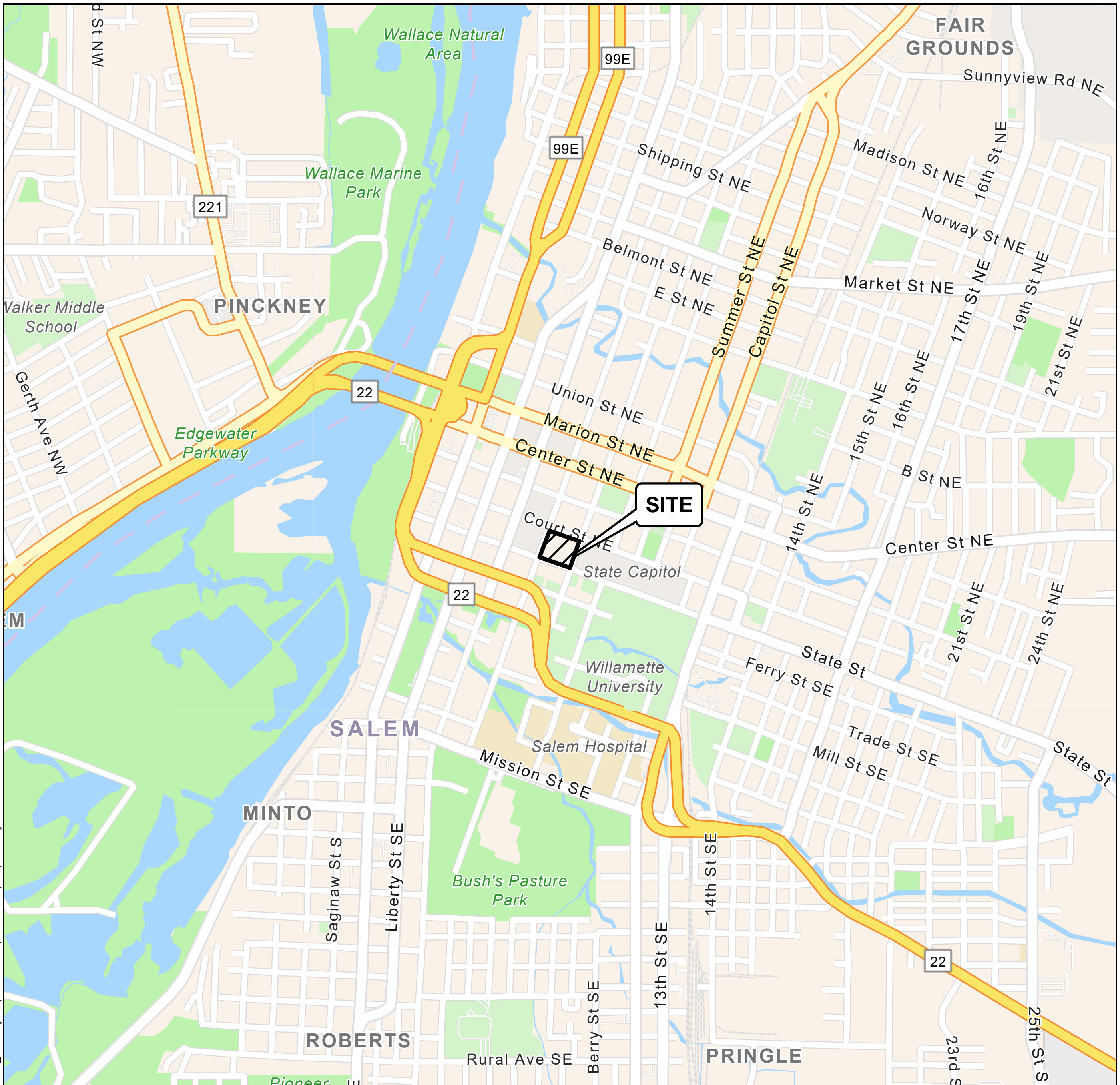
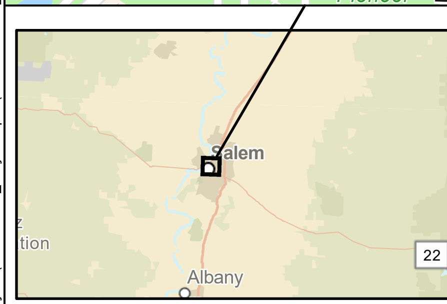
FIGURES





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Feet

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: ESRI

Projection: NAD 1983 UTM Zone 10N

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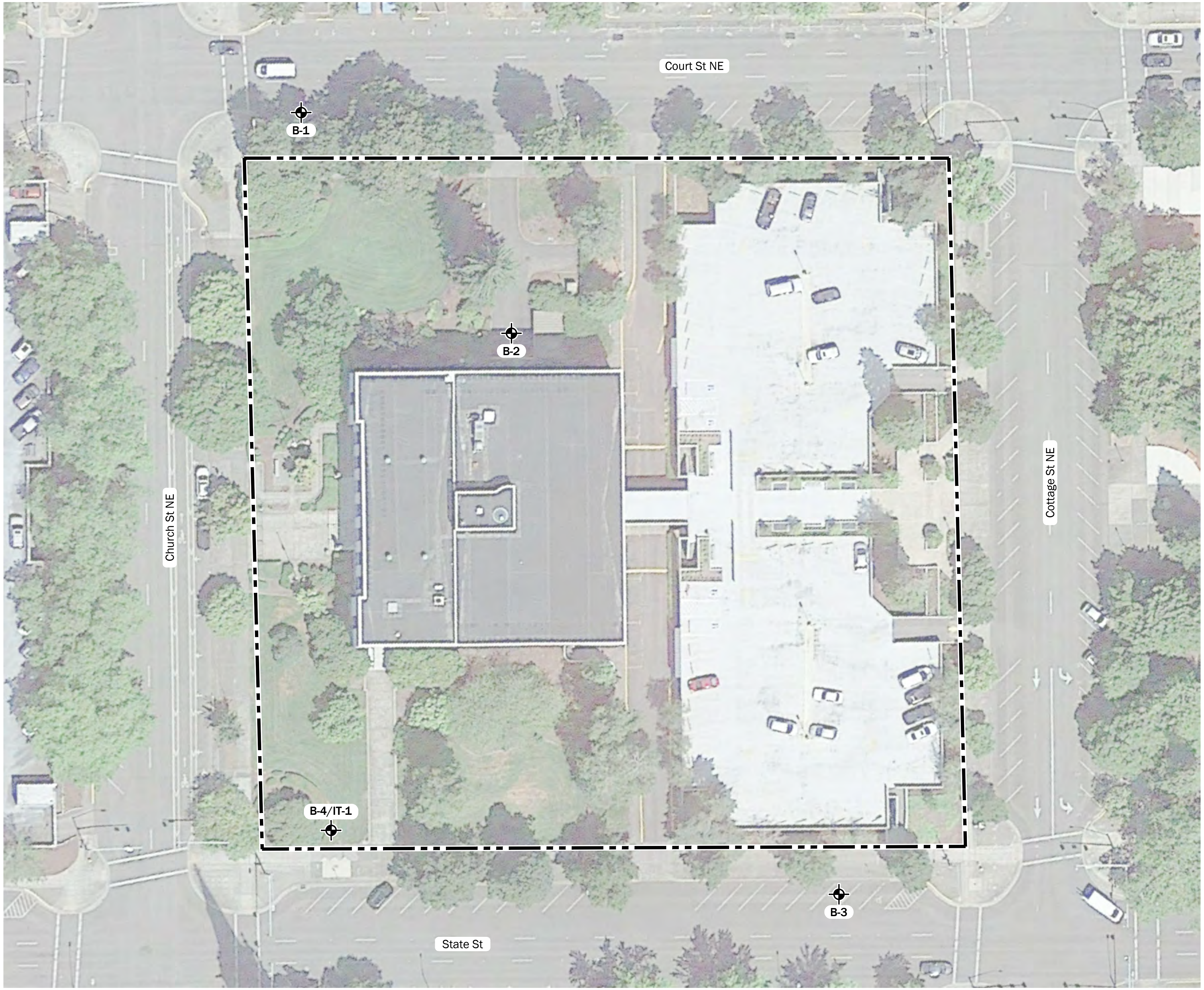
Vicinity Map	
DAS Executive Building Renovation Salem, Oregon	
GEOENGINEERS	Figure 1



Legend

Site Boundary

B-1 Boring by GeoEngineers, Inc., 2022

Notes:
1. The locations of all features shown are approximate.
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Data Source: Aerial from Google Earth Pro dated 08/12/2020.

Projection: Oregon State Plane, North Zone, NAD83, US Foot

N E S W

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Feet

Site Plan

DAS Executive Building Renovation
Salem, Oregon

GeoEngineers

Figure 2

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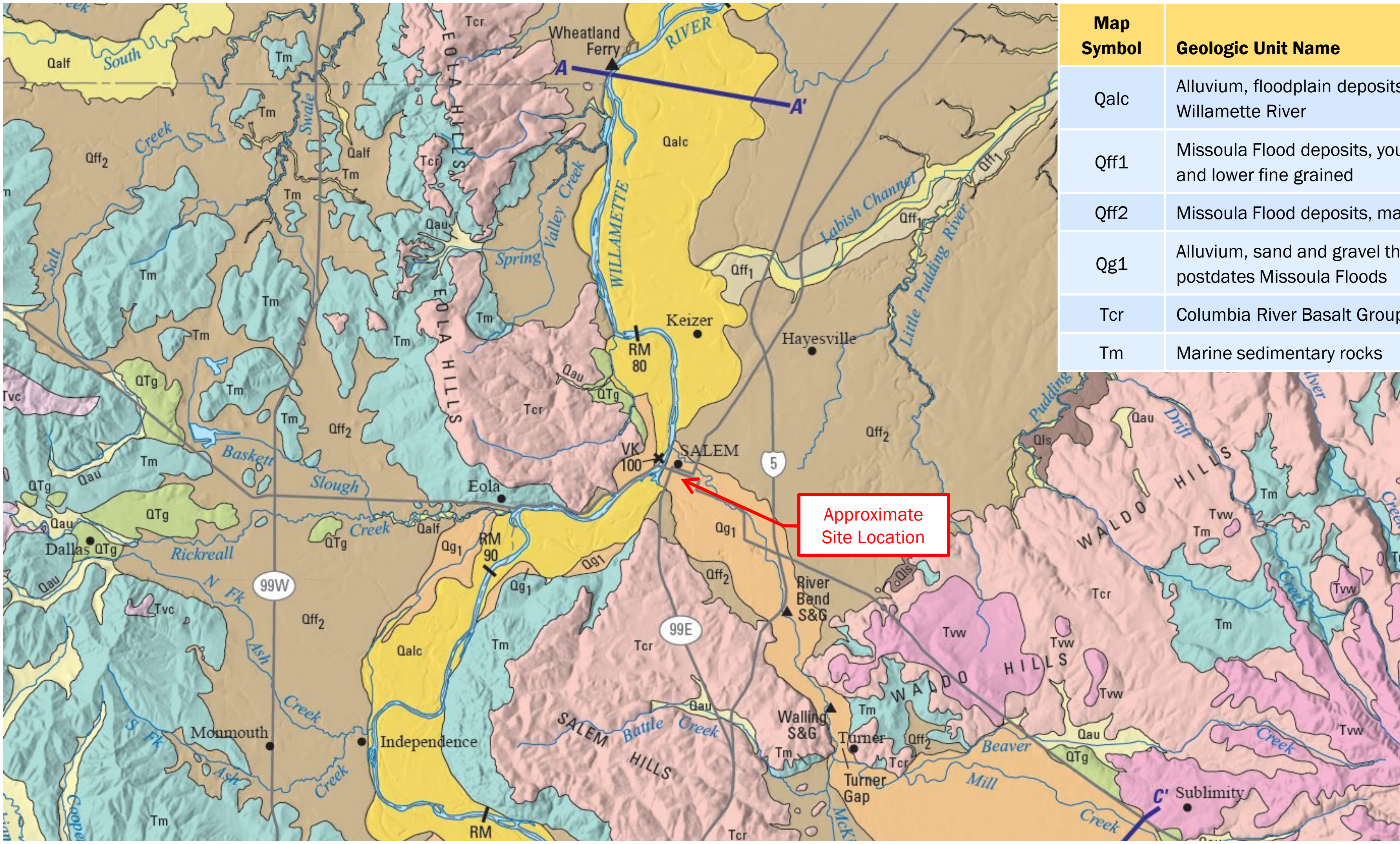

Map Symbol	Geologic Unit Name
Qalc	Alluvium, floodplain deposits of the Willamette River
Qff1	Missoula Flood deposits, younger and lower fine grained
Qff2	Missoula Flood deposits, main body
Qg1	Alluvium, sand and gravel that postdates Missoula Floods
Tcr	Columbia River Basalt Group
Tm	Marine sedimentary rocks

Geologic Map

DAS Executive Building Renovation
Salem, Oregon

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Figure 3

Notes:

1. This drawing has been reproduced from geologic mapping available online from the US Geological Survey. GeoEngineers cannot warrant or guarantee the accuracy or completeness of information provided or compiled by others.
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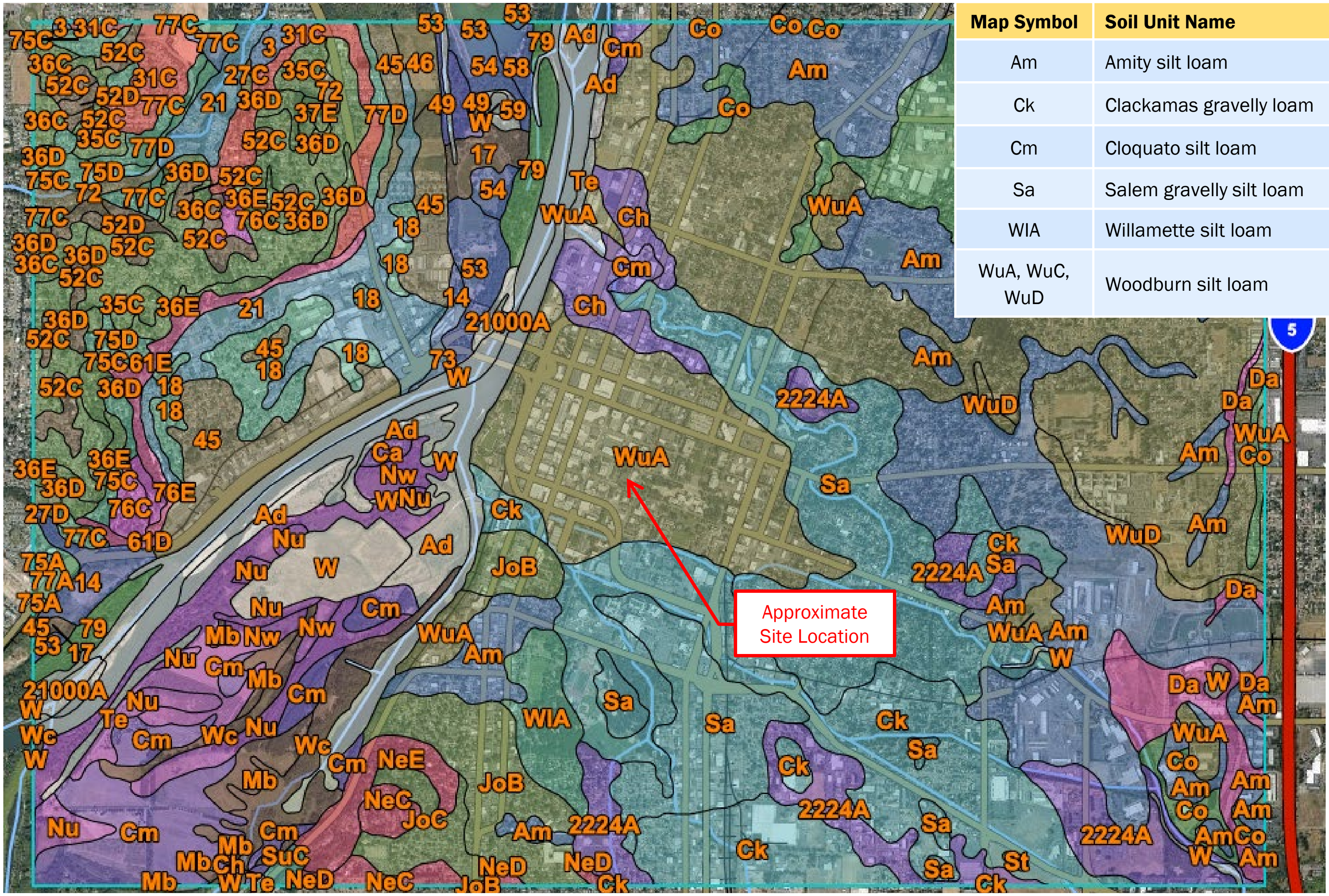
Data Source: O'Connor, J. E., Sarna-Wojcicki, A., Wozniak, K. C., Polette, D. J., & Fleck, R. J. (2001). Geologic Map of Quarternary Units in the Willamette Valley, Oregon. Professional Paper 1620, United States Geological Survey, Plate 1.

11681-015-00 Date Exported: 03/22/2022



Map Symbol	Soil Unit Name	Parent Material
Am	Amity silt loam	Mixed silty alluvium
Ck	Clackamas gravelly loam	Gravelly mixed alluvium
Cm	Cloquato silt loam	Alluvium
Sa	Salem gravelly silt loam	Gravelly mixed alluvium
WlA	Willamette silt loam	Silty alluvium
WuA, WuC, WuD	Woodburn silt loam	Silty alluvium

* Soil Units are color-coded based on Parent Material

Notes:

1. This drawing has been reproduced from the United States Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Web Soil Survey (WSS). GeoEngineers cannot warrant or guarantee the accuracy or completeness of information provided or compiled by others.
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Data Source: USDA NRCS Web Soil Survey, accessed online March 22, 2022

Web Soil Survey - Soil Type

DAS Executive Building Renovation
Salem, Oregon

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Figure 4

BPON (Basic Performance Objective Equivalent to New Building Standards)

BSE-1N (2/3*BSE-2N)	
Period, T (sec)	Sa (g)
0.01	0.317
0.13	0.660
0.63	0.660
0.70	0.591
0.80	0.518
0.90	0.460
1.00	0.414
1.20	0.345
1.40	0.296
1.70	0.244
2.00	0.207
2.50	0.166
3.00	0.138
4.00	0.104
5.00	0.083
6.00	0.069
7.00	0.060
8.00	0.052
9.00	0.047
10.00	0.041

BSE-2N (MCER) (2,475-year)	
Period, T (sec)	Sa (g)
0.01	0.476
0.13	0.991
0.63	0.991
0.70	0.887
0.80	0.776
0.90	0.690
1.00	0.621
1.20	0.518
1.40	0.444
1.70	0.365
2.00	0.311
2.50	0.248
3.00	0.207
4.00	0.155
5.00	0.124
6.00	0.104
7.00	0.089
8.00	0.078
9.00	0.069
10.00	0.062

BPOE (Basic Performance Objective for Existing Buildings)

BSE-1E (225-year)	
Period, T (sec)	Sa (g)
0.01	0.132
0.09	0.254
0.44	0.254
0.50	0.225
0.60	0.188
0.70	0.161
0.80	0.141
1.00	0.113
1.20	0.094
1.40	0.080
1.70	0.066
2.00	0.056
2.50	0.045
3.00	0.038
4.00	0.028
5.00	0.023
6.00	0.019
7.00	0.016
8.00	0.014
10.00	0.011

BSE-1N (2/3*BSE-2N)	
Period, T (sec)	Sa (g)
0.01	0.355
0.12	0.740
0.58	0.740
0.64	0.675
0.70	0.617
0.80	0.540
0.90	0.480
1.00	0.432
1.20	0.360
1.40	0.309
1.70	0.254
2.00	0.216
2.50	0.173
3.00	0.144
4.00	0.108
5.00	0.086
6.00	0.072
7.00	0.062
8.00	0.054
10.00	0.043

Notes:

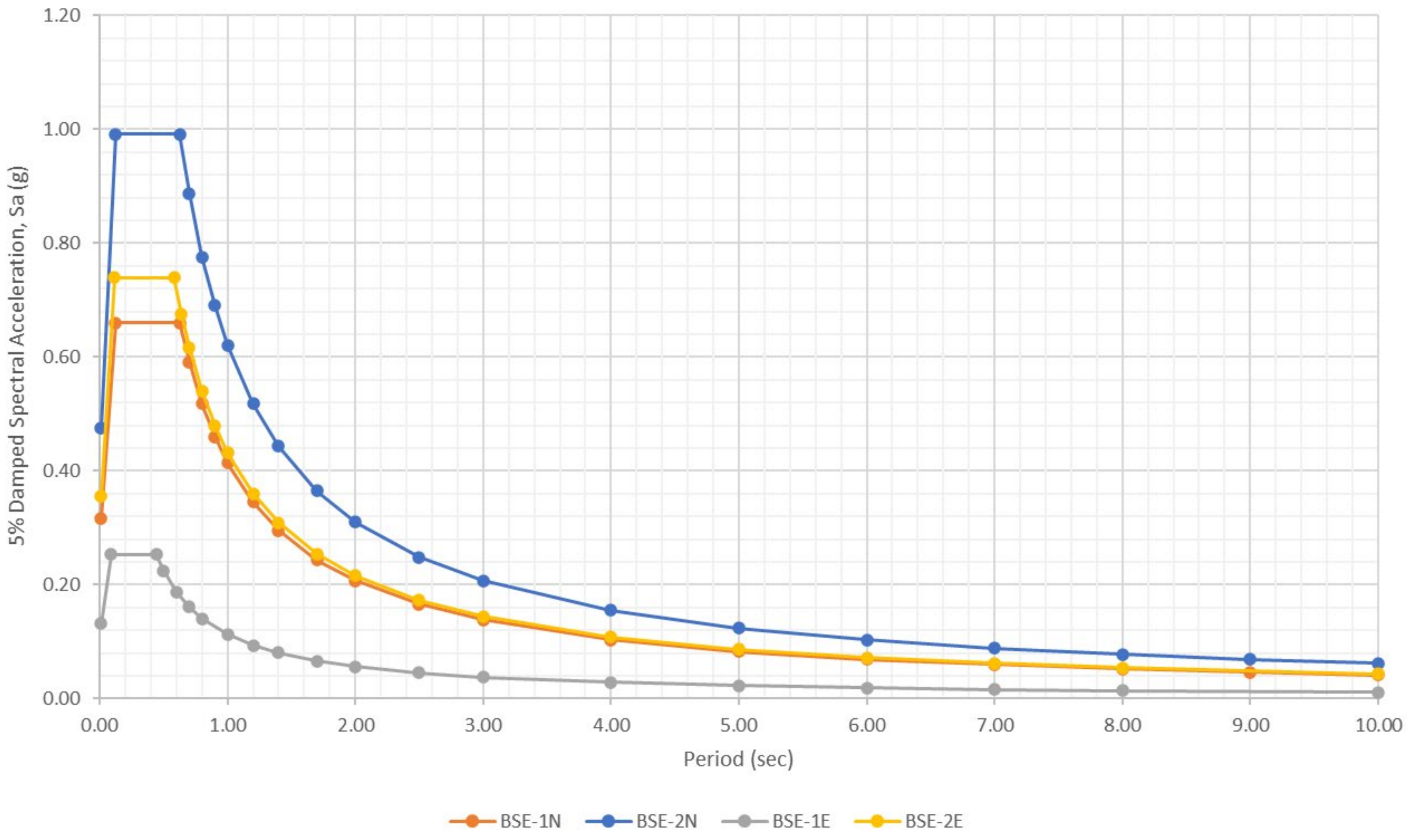
1. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Recommended ASCE 41-17 Response Spectra

DAS Executive Building Renovation
Salem, Oregon

GeoEngineers

Figure 5



Notes:

1. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Recommended ASCE 41-17 Response Spectra	
DAS Executive Building Renovation Salem, Oregon	
GeoEngineers	Figure 6

APPENDICES

APPENDIX A
Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Exploration Program

We completed field explorations at the site by advancing four drilled borings (B-1 through B-4) on March 4, 2022. Borings were advanced to depths between about 8 and 21.5 feet below ground surface (bgs). Representative soil samples from the borings were returned to our laboratory for examination and testing. Detailed descriptions of our site exploration and laboratory-testing programs, along with exploration logs and laboratory test results, are presented in Appendix A. The approximate locations of the explorations are shown on the Site Plan, Figure 2.

Soil and groundwater conditions at the site were explored on March 4, 2022 by completing four borings at the approximate locations shown on Figure 2. Borings were advanced using hollow stem drilling methods to depths ranging from about 8 to 21.5 feet bgs using a truck-mounted drill rig owned and operated by PLi Systems, Inc.

The drilling was continuously monitored by an engineering geologist from our office who maintained a detailed log of subsurface explorations, visually classified the soil encountered and obtained representative soil samples from the borings. Samples were collected using a 1-inch, inside-diameter, standard split spoon sampler or a 3-inch, inside-diameter, Dames and Moore split-spoon sampler. Samplers were driven into the soil using an automatic 140-pound hammer, free-falling 30 inches on each blow. The number of blows required to drive the sampler each of three, 6-inch increments of penetration were recorded in the field. The sum of the blow counts for the last two, 6-inch increments of penetration was reported on the boring logs as the ASTM International (ASTM) Standard Practices Test Method D 1556 SPT N-value.

The approximate N-values for D&M samples were converted to standard penetration test (SPT) N values using the Lacroix-Horn Conversion [N(SPT) = (2*N1*W1*H1)/(175*D1*D1*L1), where N1 is the non-standard blow count, W1 is the hammer weight in pounds (140), H1 is the hammer drop height in inches (30), D1 is the non-standard sampler outside diameter in inches (3.23) and L1 is the length of penetration in inches (12)].

Recovered soil samples were visually classified in the field in general accordance with ASTM D 2488 and the classification chart listed in Key to Exploration Logs, Figure A-1. The logs of the borings are presented in Figures A-2 through A-5. The log is based on interpretation of the field and laboratory data and indicate the depth at which subsurface materials, or their characteristics, change although these changes might actually be gradual.

Laboratory Testing

Soil samples obtained from the explorations were retained in sealed plastic bags and transported to the GeoEngineers laboratory. Representative soil samples were selected for laboratory tests to evaluate pertinent geotechnical engineering characteristics of the soils and refine our field classification, as necessary. The following paragraphs provide a description of the tests performed.

Moisture Content (MC)

Selected samples were oven dried to estimate the percentage of water (on a mass basis) in the soil. The moisture content was determined in general accordance with ASTM Test Method D 2216. Test results are presented on the exploration logs, as indicated for the sample tested.

Percent Fines (%F)

Selected samples were "washed" through the U.S. No. 200 sieve to estimate the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve (fines). Tests were conducted in general accordance with ASTM D 1140. Test results are presented on the exploration logs at the respective sample depths.

Atterberg Limits Testing

Atterberg Limits were performed on selected samples in general accordance with ASTM Test Method D4318. This test method determines the liquid limit, plastic limit and plasticity index of soil particles passing the No. 40 sieve. Results for plastic soils are presented in Figure A-6. The liquid limit and plasticity index are also presented on the exploration logs at the respective sample depths.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	SOD	Sod/Forest Duff
	TS	Topsoil

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel / Dames & Moore (D&M)

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PL	Point lead test
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
UU	Unconsolidated undrained triaxial compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

Key to Exploration Logs

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Figure A-1

Drilled	Start 3/4/2022	End 3/4/2022	Total Depth (ft) 21.5	Logged By JLL / Checked By ST	Driller PLI Systems	Drilling Method Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	156 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mobile Drill B-58 Truck-mounted
Latitude / Longitude	44.9402 / -123.0348	System / Datum	Decimal Degrees / WGS84	See "Remarks" section for groundwater observed	

Notes: D&M N-values reduced using Lacroix-Horn Equation to approximate SPT N-values

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SOD	Approximately 3 inches sod			
155								CL	Dark brown lean clay with trace sand (soft, moist) (fine-grained alluvium)			
			10	5		1 / AL			Becomes dark brown with gravel, medium stiff	25		AL (LL = 40, PL = 22, PI = 18)
150	5		10	50/6"		2 / MD		GP-GM	Brown poorly-graded gravel with sand and silt (medium dense to very dense, wet) (coarse-grained alluvium)	6		
			12	31		3			Becomes wet			Groundwater observed at approximately 8 feet during drilling
145	10		10	20		4						
140	15		10	59		5						
135	20		10	35		6						

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Google Earth. Vertical approximated based on Google Earth.

Log of Boring B-1

GEOENGINEERS

Project: DAS Executive Building Renovation
Project Location: Salem, Oregon
Project Number: 11681-015-00

Figure A-2
Sheet 1 of 1

Drilled	Start 3/4/2022	End 3/4/2022	Total Depth (ft) 21.5	Logged By JLL Checked By ST	Driller PLI Systems	Drilling Method Hollow-stem Auger

Surface Elevation (ft) Vertical Datum	158 NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mobile Drill B-58 Truck-mounted
Latitude Longitude	44.9398 -123.0345	System Datum	Decimal Degrees WGS84	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							AC	Approximately 2 inches asphalt concrete			
								GP	Approximately 6 inches aggregate base			
155		X	12	13		1		CL	Dark brown lean clay with sand and gravel (stiff, moist) (fine-grained alluvium)			
	5	X	6	18		2			Becomes gravelly silt, very stiff			
150		X	12	37		3		GM	Gray-brown gravel with sand with silt (medium dense to very dense, moist) (coarse-grained alluvium)			
	10	X	12	50		4			Becomes wet			Groundwater observed at approximately 10 feet during drilling
145												
	15	X	10	22		5						
140												
	20	X	10	58		6						

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Google Earth. Vertical approximated based on Google Earth.

Log of Boring B-2

GEOENGINEERS

Project: DAS Executive Building Renovation
Project Location: Salem, Oregon
Project Number: 11681-015-00

Drilled	Start 3/4/2022	End 3/4/2022	Total Depth (ft)	21.5	Logged By JLL / Checked By ST	Driller PLI Systems	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	157 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mobile Drill B-58 Truck-mounted
Latitude / Longitude	44.9389 / -123.0343	System / Datum	Decimal Degrees / WGS84	See "Remarks" section for groundwater observed	

Notes: D&M N-values reduced using Lacroix-Horn Equation to approximate SPT N-values

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
157	0							AC	Approximately 5 inches asphalt concrete pavement			
								GP	Approximately 13 inches aggregate base			
155								SM	Dark brown silty fine to coarse sand with gravel and occasional debris (medium dense, moist) (fill)			
			10	16		1						
	5		12	50/6"		2 MD		GP-GM	Dark gray-brown poorly-graded gravel with sand and silt (medium dense to very dense, moist) (Linn gravel)	9		
150												
			10	47		3 MD			Becomes wet	9		Groundwater observed at approximately 8 feet during drilling
	10		5	20		4			Becomes yellow-brown			
145												
	15		12	60		5						
140												
	20		12	40		6						

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Google Earth. Vertical approximated based on Google Earth.

Log of Boring B-3

GeoEngineers

Project: DAS Executive Building Renovation
Project Location: Salem, Oregon
Project Number: 11681-015-00

Figure A-4
Sheet 1 of 1

Drilled	Start 3/4/2022	End 3/4/2022	Total Depth (ft)	8	Logged By JLL	Checked By ST	Driller PLI Systems	Drilling Method Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	157 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Mobile Drill B-58 Truck-mounted
Latitude / Longitude	44.9392 / -123.0352	System / Datum	Decimal Degrees / WGS84	See "Remarks" section for groundwater observed	

Notes:

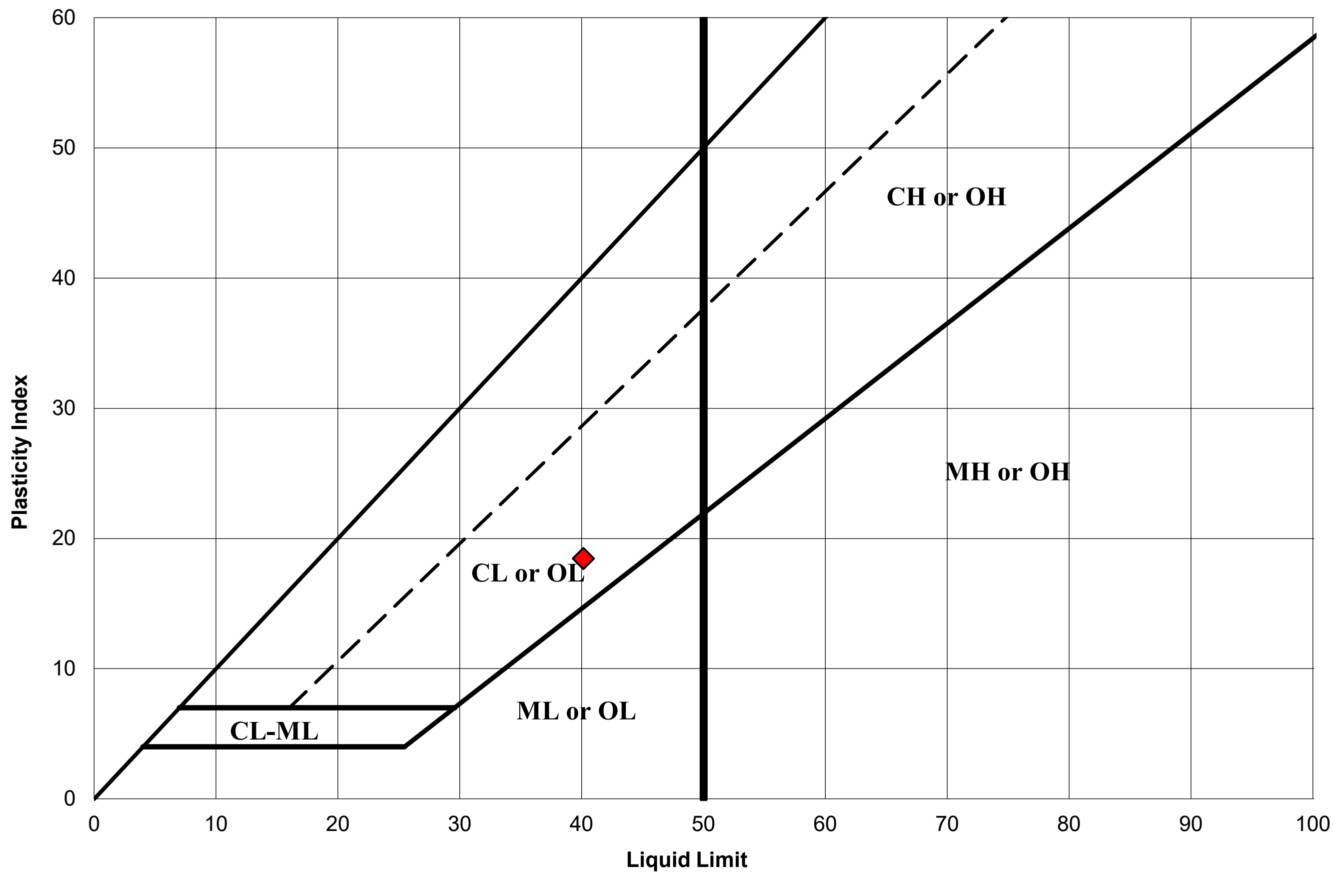
Elevation (feet)	Depth (feet)	FIELD DATA: Interval	FIELD DATA: Recovered (in)	FIELD DATA: Blows/foot	FIELD DATA: Collected Sample	FIELD DATA: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							SM	Dark brown silty fine sand with fine gravel (very loose, moist) (fill)			
155									Becomes brown, without gravel			
		X	6	3		1 / %F				21	31	%F = 31 Infiltration test IT-1 completed at 3 feet, refer to report text
	5	X	10	3		2			Becomes with organic matter			
150												
									Becomes wet			Groundwater observed at approximately 7¾ feet during drilling

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on Google Earth. Vertical approximated based on Google Earth.

Log of Boring B-4/IT-1

GEOENGINEERS

Project: DAS Executive Building Renovation
Project Location: Salem, Oregon
Project Number: 11681-015-00



Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆	B-1	2.5	25	40	18	Lean Clay (CL)

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AASHTO ACCREDITED

Atterberg Limits Test Results

DAS Executive Building Renovation
Salem, OR

GEOENGINEERS

Figure A-6

11681-015-00 Date Exported: 3/23/22

APPENDIX B
Site-Specific Seismic Hazard Report

APPENDIX B
SITE-SPECIFIC SEISMIC HAZARD REPORT

Introduction

This appendix presents the results of GeoEngineers' site-specific seismic hazard evaluation for the proposed Oregon Department of Administrative Services (DAS) Executive Building Renovation project in Salem, Oregon. This hazard report was prepared to meet the requirements of the State of Oregon Structural Specialty Code (OSSC) Section 1803 and the International Building Code (IBC).

Subsurface and Groundwater Conditions

The details of our field exploration program, subsurface conditions, and measured groundwater levels are provided in Section 3.0 Site Conditions in the main body of this report. Approximate field exploration locations are shown in the Site Plan, Figure 2.

Regional Seismicity

Earthquake Source Zones

Tectonic conditions in the Salem area are similar to those typical of western Oregon; potential seismic sources include the convergent continental boundary known as the Cascadia Subduction Zone (CSZ), an approximately 720-mile-long thrust fault that extends along the Pacific Coast from mid-Vancouver Island to Northern California. The CSZ is the zone where the westward advancing North American Plate is overriding the subducting Juan de Fuca Plate. The interaction of these two plates results in two potential seismic source zones: (1) the subducting slab/Benioff (or "intraslab") source zone; and (2) the CSZ subduction-zone (or "interplate") source zone. A third seismic source zone, referred to as the shallow crustal source zone, is associated with the regional convergent tectonic stresses. A discussion of these potential sources is provided below.

Cascadia Subduction Zone/Interplate Sources

The closest portion of the CSZ to Salem are several strands of the "Cascadia Fold and Thrust Belt" mapped approximately 50 miles west of the site (Personius 2002a). This structure is a forearc-portion of the roughly 720-mile-long zone of active tectonic convergence where oceanic crust of the Juan de Fuca Plate is subducting beneath the North American continent at a rate of 5 cm/year (DeMets et al. 2010). Most of these folds and faults are mapped as active in the Holocene or late Pleistocene, but some are mapped as active in the middle and late Quaternary (<700-780 ka) or Plio-Pleistocene (Personius 2002a). The hazard to the Salem region from a great Cascadia/interplate event, while present, is not significantly greater than any other portion of the western Oregon region.

Intraslab Earthquake Sources

Typically, the region of maximum curvature of the subducting Juan de Fuca slab is where large intraslab earthquakes are expected to occur; in northern and central Oregon this is assumed to be roughly 30 miles below the Oregon Coast Range. This area is located roughly 30 miles west of the site. However, Wong (2005) notes that "…no moderate-to-large intraslab earthquakes of moment magnitude (M) 5.5 or greater have occurred within the subducting Juan de Fuca plate of the central CSZ from south of the Puget Sound in northwestern Washington to the Oregon–California border. Also, very few intraslab earthquakes as small as M3 have been instrumentally located within the central CSZ since 1960, and a Wadati–Benioff zone is not apparent." Therefore, we conclude that the hazard to the site from intraslab earthquakes is low.

Crustal Earthquake Sources

The nearest mapped fault to the site is the Salem-Eola Hills homocline, an approximate 20-mile-long northwest-striking fold mapped to within about 4.4 miles southwest of the site (Personius 2002b). This fault is listed as having a low probability to be active. The nearest mapped fault with documented historical activity is the Mount Angel Fault, mapped to within about 14 miles northeast of the site (Black 1996). The Mount Angel Fault is considered to be the dominant crustal seismic source for the site vicinity.

The historical seismicity in relation to the project site is shown on the Historical Seismicity Map, Figure B-1. The location and relative distance of Quaternary faults within a 50-mile radius from the site have been summarized from geologic publications and are indicated on the Quaternary Fault Map, Figure B-2. The estimated distance, displacement, and relative age of faults within a 40-mile radius from the site is indicated in Table B-1 (Personius 2002b-o).

TABLE B-1. MAPPED FAULTS WITHIN 40-MILE RADIUS OF SITE

Fault Name	Proximity to the Site, Surface Projection in Miles (Direction)	Estimated Displacement Description	Estimated Age
Salem-Eola Hills homocline	4.4 (SW)	Offsets CRB flows and Miocene age deposits.	Quaternary (<1.6 million years before present)
Waldo Hills	5.1 (SE)	Offsets Columbia River Basalt (CRB) flows. Does not offset surficial Quaternary deposits.	Quaternary (<1.6 million years before present)
Mill Creek	7.5 (SSE)	Offsets CRB flows and Miocene age deposits.	Quaternary (<1.6 million years before present)
Mount Angel	14 (NE)	Offsets late Pleistocene and Holocene deposits. Associated with earthquake swarms near Woodburn (1990) and ML 5.6 earthquake near Scotts Mills (1993).	Late Quaternary (<15,000 years before present)
Newberg	24.2 (NNE)	Controlled emplacement of CRB flows. No documented offset of overlying younger deposits.	Quaternary (<1.6 million years before present)
Corvallis	24.6 (SSW)	Possible offsets in late Pleistocene-age deposits. Does not offset Missoula flood deposits.	Quaternary (>15,000 years before present)
Owl Creek	24.8 (SSW)	Offsets late Pleistocene age deposits. Does not offset Missoula flood deposits.	Middle Quaternary (<750,000 years before present)
Canby-Molalla	26.6 (NE)	Probable offset of Missoula flood deposits.	Late Quaternary (<15,000 years before present)

Fault Name	Proximity to the Site, Surface Projection in Miles (Direction)	Estimated Displacement Description	Estimated Age
Gales Creek	29.6 (NNW)	Offsets CRB flows.	Quaternary (<1.6 million years before present)
Beaverton Fault	36.5 (N)	Offsets CRB flows and Miocene age deposits.	Late Quaternary (<1.6 million years before present)
Oatfield Fault	36.9 (NE)	Offsets CRB flows and Miocene age deposits.	Quaternary (<1.6 million years before present)
Portland Hills Fault	38.3 (NE)	Offsets CRB flows.	Quaternary (<1.6-1.8 million years before present)
Damascus-Tickle Creek Fault	39 (NE)	Offsets Pliocene Troutdale Formation, Pliocene-Pleistocene Springwater Formation, and Pleistocene Boring Lava.	Late to middle Quaternary (750,000 years before present)
Helvetia Fault	42.3 (NNE)	Probable offset of Miocene CRB Group.	Quaternary (<1.6 million years before present)

Seismic Hazards

Based on our subsurface exploration, literature review, and experience, a summary of the seismic hazards at the site are as follows:

Seismic Hazard Mapping

We reviewed hazard level mapping in the project vicinity (Wang and Leonard, 1996). The hazard level is associated with local geology, liquefaction, amplification, and landslide hazards. Mapped hazard levels range on a four-category scale from Zone A (highest hazard) to D (lowest).

The site is mapped as Hazard Zone C, low to intermediate hazard.

Liquefaction Potential

A detailed summary of estimated liquefaction potential at the site is provided in Section 4.1.3 Liquefaction Potential in the main body of this report. Soils encountered in the borings below the ground water table generally consist of dense to very dense gravel with varying amounts of silt and sand. Based on our analyses, it is our opinion the potential for liquefaction and liquefaction-induced settlement at the site is low.

Lateral Spreading Potential

Liquefaction-induced soil strength loss can also result in slope instability and lateral spreading. Lateral spreading related to seismic activity typically involves lateral displacement of large, surficial blocks of nonliquefied soil when an underlying soil layer loses strength during seismic shaking. Lateral spreading usually develops in areas where sloping ground or large grade changes (including retaining walls) are present.

Based on our understanding of the subsurface conditions, liquefaction risk and current site topography, it is our opinion the risk of lateral spreading at the site is low.

Surface Rupture Potential

We reviewed published geologic seismic feature maps, including maps available online from DOGAMI. According to the maps, the nearest mapped faults are the Salem-Eola Hills homocline (mapped about 4.4 miles southwest of the site) and the Waldo Hills fault (about 5.1 miles southeast). Other local crustal faults are listed in the sections and table above. No faults are mapped as crossing the site or in the immediate vicinity. Based on our understanding of local geology, bedrock in the project area is covered by alluvial soil deposits.

Based on this information, it is our opinion the risk for seismic surface rupture at the site is low.

Amplificaiton

Based on relative earthquake hazard mapping completed in the site vicinity (Wang and Leonard 1996), the site is mapped as having an amplification factor ranging 1.2 to 1.4.

Earthquake-Induced Landslides

The site vicinity currently is generally flat and is planned to be graded to have relatively flat slopes during development. In our opinion, site soils encountered in the borings are not susceptible to earthquake slope instability at these inclinations. We reviewed relative earthquake hazard mapping completed in the site vicinity (Wang and Leonard 1996). The site is mapped as low hazard potential for earthquake-induced landslides.

In our opinion, the project site has a low hazard potential for earthquake-induced landslides.

Tsunami Inundation/Subsidence/Seiche

Tsunami inundation zone is defined as the horizontal distance inland that water levels penetrate during a tsunami event. Subsidence is the downward vertical movement of the Earth's surface, such as due to tectonic movement or settlement/compaction of unconsolidated sediments from seismic shaking. The project site is in the Willamette Valley, located inland and elevated away from tsunami inundation and subsidence zones.

A seiche is an oscillating or standing wave that can occur on rivers, reservoirs, ponds and lakes. They can be triggered by strong winds, changes in atmospheric pressure, earthquakes, tsunami or tides. The Willamette River is located approximately 0.5 miles west of the site and could be a potential seiche source following a seismic event. In our opinion, the site is far enough away from the nearest body of water, is higher in elevation and has a low potential for seiche inundation at the site.

Design Earthquakes and Base Rock Motions

We have evaluated three potential earthquake sources that could impact building design at the site. These include CSZ interface earthquakes, CSZ intraplate earthquakes (also referred to as Benioff Zone or intraslab), and local crustal earthquakes.

Geologic evidence indicates that CSZ interface earthquakes occur approximately every 100 to 1,000 years with average recurrence intervals of approximately 400 years. Based on postulated rupture lengths, widths, and displacements (as well as historical Pacific tsunamis), coastal subsidence, and liquefaction evidence, magnitudes for such earthquakes are estimated to range from approximately Mw= 7.8 to 9.1. These earthquakes are expected to have epicenters ranging from about 50 to 100 miles west of the site.

CSZ intraplate earthquakes have occurred in the Puget Sound area and are also postulated to occur beneath Western Oregon. These earthquakes have a deep focus of 40 to 80 kilometers (km) in the subducted Juan de Fuca Plate and theoretical magnitudes of up to 7.8. These earthquakes are expected to have epicenters ranging from about 25 to 45 miles from the site. Base rock motions estimated for these earthquakes are less than those from either CSZ interface earthquakes or crustal earthquakes at the site and do not control design.

Ground Motion Estimation

Peak ground accelerations (PGAs) were estimated for each of the events discussed above. The PGA values for the CSZ events were computed with the BCHydro ground motion prediction equation (Abrahamson 2016). The PGAs for specific and random crustal events were computed using the attenuation relationships of the five NGA West2 horizontal ground motion prediction equations second (PEER 2014). We considered Site Class C (i.e., Vs30, mean shear wave velocity of the top 30 meters, equal to 537 meters per second) in our analyses.

Computed response spectra for the postulated ground motion events are shown in comparison to BSE-2N and BSE-2E on the Estimated Response Spectra Comparison, Figures B-3 and B-4. As shown on the figures, the BSE-2N and BSE-2E spectra as defined in ASCE 41-17 are higher than the estimated spectra for CSZ interface, CSZ intraplate and local crustal earthquakes. Therefore, we recommend seismic design parameters in accordance with the ASCE 41-17 be used for seismic design as described in the main body of this report. Table B-2 shows the postulated earthquakes and computed PGAs.

TABLE B-2. CALCULATED PGA VALUES

Earthquake Source	Magnitude	Focal Depth (miles)	Epicentral Distance (miles)	Anticipated PGA (g)
CSZ Interplate	9.1	20	38.5	0.267
CSZ Intraplate	7.8	31	25.5	0.353
Salem-Eola Hills Fault	6.0	4.8	4.4	0.372
Waldo Hills Fault	6.2	4.8	5.1	0.361

Note:

[1] Maximum Considered Earthquake (MCER) epicentral distance to center of fault.

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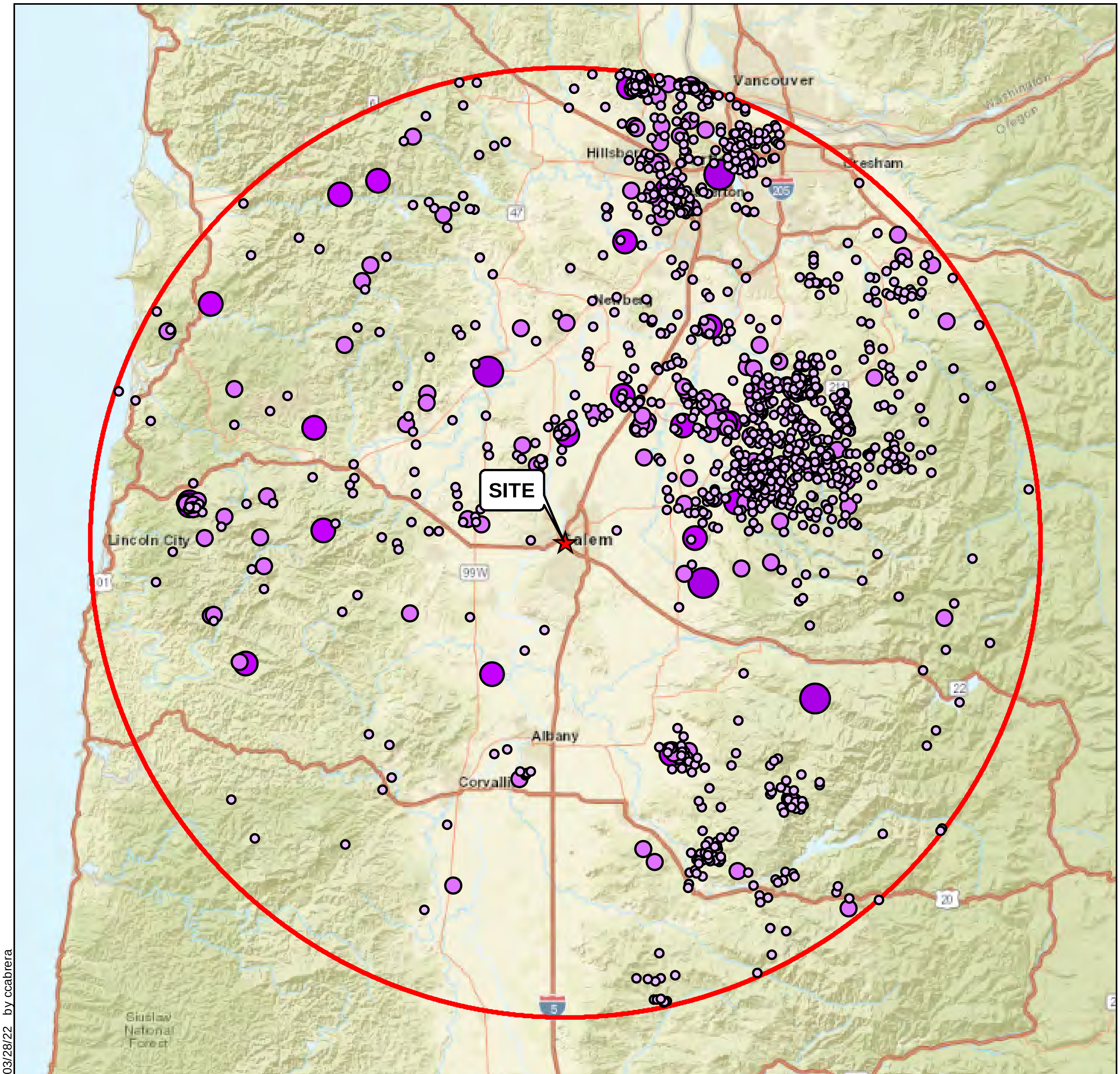

Legend

★ Project Site

50-mile Radius

USGS Earthquakes by Magnitude

- > 5
- 4 - 5
- 3 - 4
- 2 - 3
- < 2

N
W E
S

16 0 16
Miles

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2015
Seismic locations from USGS, http://earthquake.usgs.gov/earthquakes
Projection: NAD 1983 UTM Zone 10N

Historical Seismicity Map

DAS Executive Building Renovation
Salem, Oregon

GEOENGINEERS

Figure B-1

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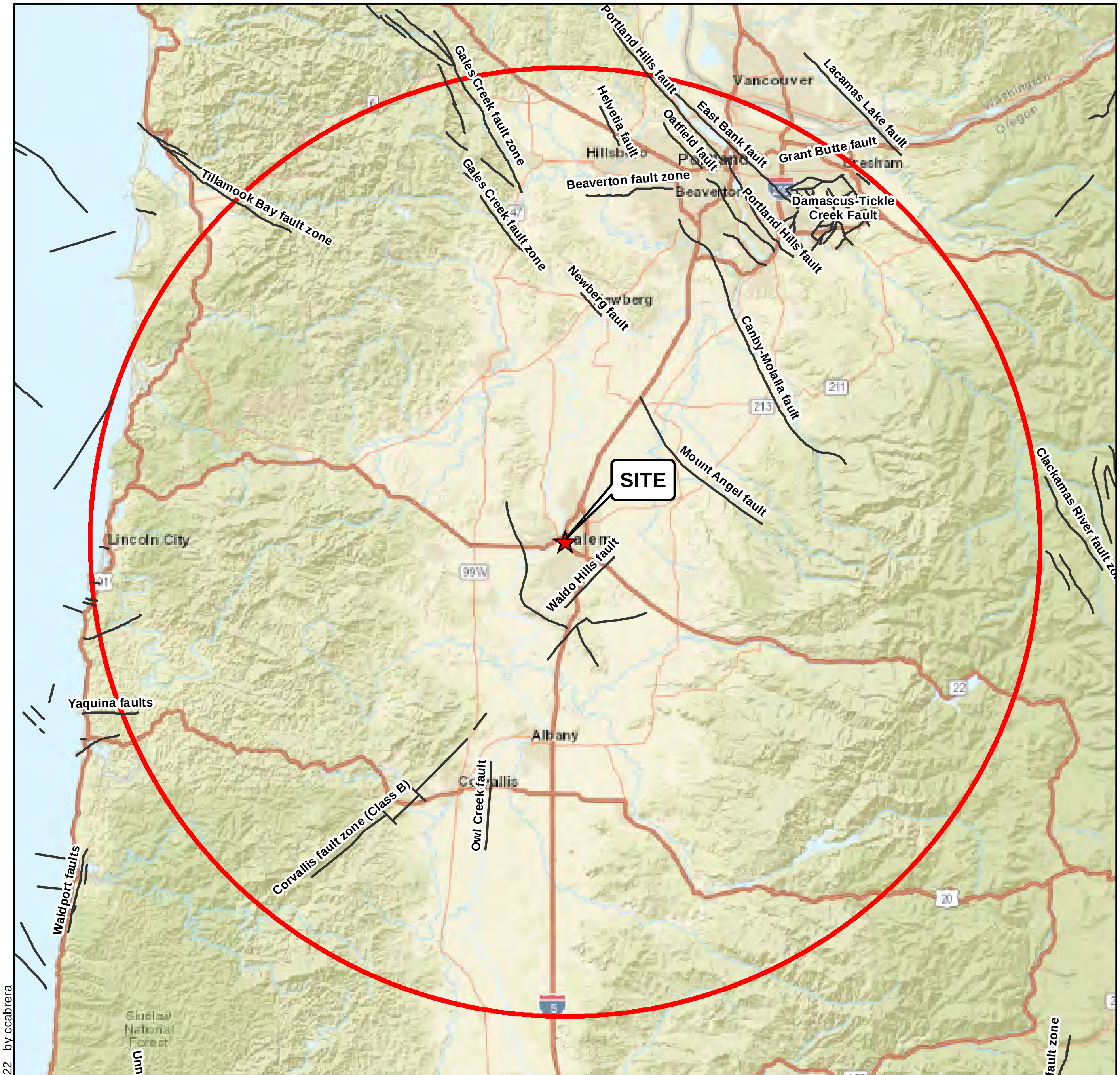

Legend

- Project Site
- USGS Quaternary Faults
- 50-mile Radius

N
W E
S

16 0 16
Miles

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2015
Quaternary faults from USGS, http://earthquake.usgs.gov/hazards/qfaults/
Projection: NAD 1983 UTM Zone 10N

Quaternary Fault Map

DAS Executive Building Renovation
Salem, Oregon

GeoEngineers

Figure B-2

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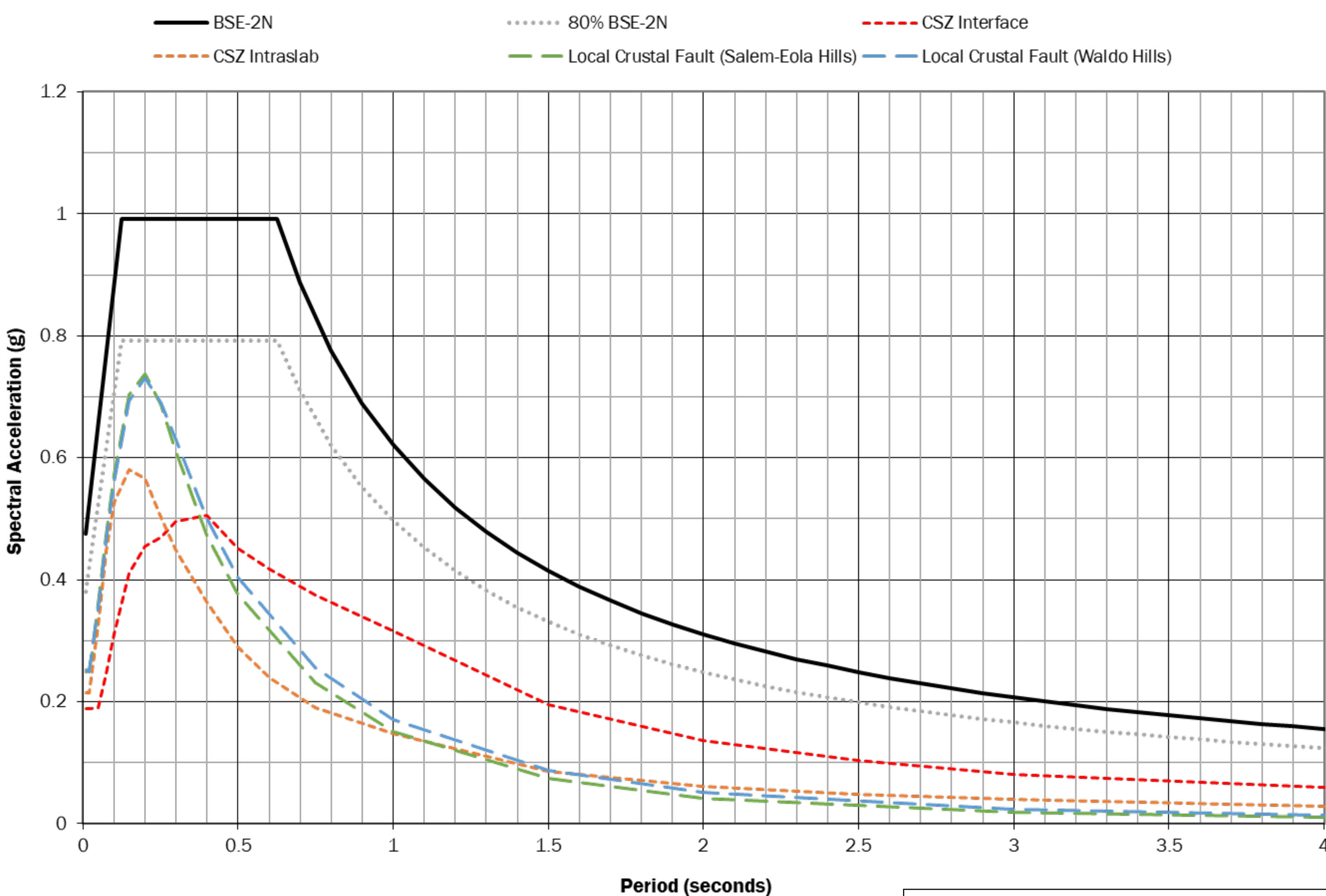


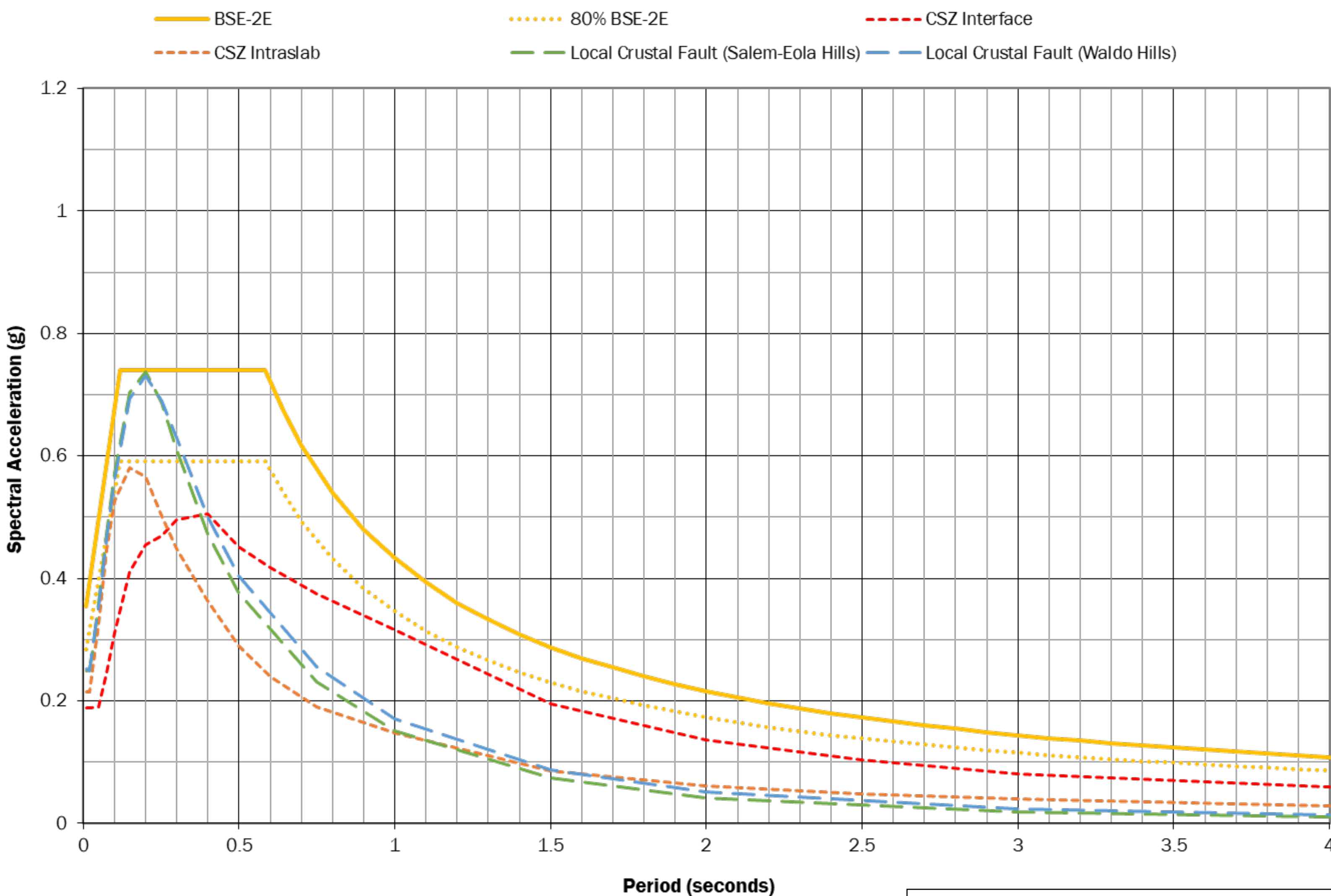
Response Spectra Comparison, BSE-2N

DAS Executive Building Renovation
Salem, Oregon

GeoEngineers

Figure B-3

Notes:
1. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.



Notes:

1. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Response Spectra Comparison, BSE-2E

DAS Executive Building Renovation
Salem, Oregon

GeoEngineers

Figure B-4

APPENDIX C
Report Limitations and Guidelines for Use

APPENDIX C
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory "limitations" provisions in its reports. Please confer with GeoEngineers if you need to know more about how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical Services are Performed for Specific Purposes, Persons and Projects

This report has been prepared for Otak, Inc. and the Oregon Department of Administrative Services (DAS) for the Project specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the Project, and its schedule and budget, our services have been executed in accordance with our Agreement with Otak, Inc. dated February 17, 2022 (authorized March 1, 2022), and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

A Geotechnical Engineering or Geologic Report is based on a Unique Set of Project-Specific Factors

This report has been prepared for the DAS Executive Building Renovation project in Salem, Oregon. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;

1 Developed based on material provided by GBA, GeoProfessional Business Association; www.geoprofessional.org.

- composition of the design team; or
- project ownership.

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

Environmental Concerns are Not Covered

Unless environmental services were specifically included in our scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

Subsurface Conditions Can Change

This geotechnical report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Geotechnical and Geologic Findings are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from one sampling location at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

Geotechnical Engineering Report Recommendations are Not Final

We have developed the following recommendations based on data gathered from subsurface investigation(s). These investigations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions. If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable but separating logs from the report can create a risk of misinterpretation.

Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- encourages contractors to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer.

Contractors are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field

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