GEOTECHNICAL INVESTIGATION
SITE-SPECIFIC SEISMIC HAZARD EVALUATION

WESTERN OREGON UNIVERSITY NEW SCIENCE CENTER

GEOCON
NORTHWEST, INC.
GEOTECHNICAL &
ENVIRONMENTAL
CONSULTANTS

PREPARED FOR

WESTERN OREGON UNIVERSITY
MONMOUTH, OREGON

OCTOBER 2011

GEOCON
NORTHWEST, INC.
GEOTECHNICAL/ENVIRONMENTAL CONSULTANTS

P1844-05-01
October 28, 2011

Mr. Brad Huggins
Western Oregon University
345 North Monmouth Avenue
Monmouth, Oregon 97361

Subject: WESTERN OREGON UNIVERSITY
NEW SCIENCE CENTER
MONMOUTH, OREGON
GEOTECHNICAL INVESTIGATION AND SITE
SPECIFIC SEISMIC HAZARD EVALUATION

Dear Mr. Huggins:

In accordance with our proposal number P11-05-92, dated September 6, 2011, and your authorization, we have performed a geotechnical investigation and site specific seismic hazard evaluation for the proposed New Science Center at the Western Oregon University Campus in Monmouth, Oregon. The accompanying report presents the findings of our study and our conclusions and recommendations regarding the geotechnical aspects of the proposed project. Based on the results of this investigation, it is our opinion that the New Science Center site can be developed as proposed, provided the recommendations of this report are followed. Important geotechnical issues addressed herein include the presence of existing improvements, undocumented fill, perched groundwater potential, and grading recommendations for the moisture sensitive underlying fine-grained soil. It is recommended that site grading be completed during the summer months to help reduce the potential for increased site preparation costs. Architectural and structural information for the proposed building was not available at the time of our report preparation. Geocon Northwest should be consulted as the project design progresses for additional geotechnical evaluation as required.

If you have questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Sincerely,

Geocon Northwest, Inc.

CERTIFIED ENGINEERING GEOLOGIST
SEAN M. DIXON
No. 2159

REGISTERED PROFESSIONAL ENGINEER
18281
JAN 16, 1996
WESLEY SPANG
EXPIRATION DATE: 6/30/ 2012

Wesley

Sean M. Dixon, C.E.G. Exp. 12/31/11
Project Geologist

Wesley Spang, Ph.D., P.E.
President

8283 SW Cirrus Drive ■ Beaverton, Oregon 97008 ■ Telephone (503) 626-9889 ■ Fax (503) 626-8611

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GEOTECHNICAL INVESTIGATION

1 PURPOSE AND SCOPE

This report presents the results of the geotechnical investigation for the proposed New Science Center at Western Oregon University in Monmouth, Oregon. The development site is located immediately north of the recently constructed Health and Wellness Center, as shown in Figure 1, Site Vicinity Map. The purpose of the geotechnical investigation was to evaluate subsurface soil and geologic conditions at the site and, based on the conditions encountered, provide conclusions and recommendations pertaining to the geotechnical aspects of the construction of the proposed New Science Center.

The scope of the field investigation consisted of a site reconnaissance, review of published geological literature, review of our geotechnical report completed for the Health and Wellness Center in April 2009, and the completion of two exploratory borings. A detailed discussion of the field investigation is presented in Section 4 of this report.

Laboratory tests were performed on selected soil samples obtained during the investigation to evaluate pertinent physical properties. The results of laboratory moisture content tests are presented on the exploratory boring logs. Appendix B presents a summary of the remaining laboratory test results.

The recommendations presented herein are based on analyses of the data obtained during the field investigation, laboratory test results, and our experience with similar soil and geologic conditions. This report has been prepared for the exclusive use of Western Oregon University and its agents, for specific application to this project, in accordance with generally accepted geotechnical engineering practice. This report may not contain sufficient information for purposes of other parties or other uses.

2 SITE AND PROJECT DESCRIPTION

We understand Western Oregon University is proposing to construct a new New Science Center in the southern portion of the existing Parking Lot F and north of the recently constructed Health and Wellness Center. The new building is proposed to be approximately two to three stories and have an approximate 10,000 square-foot footprint. Our firm performed the geotechnical services for the construction of the new Health and Wellness Center and we have reviewed our previous work for the preparation of this report. Numerous known underground utilities traverse the area of proposed construction.

The site topography exhibits relatively minor elevation change; therefore, site grading is expected to consist of cuts and fills of less than 5 feet, unless below-grade construction is planned. Building loads are not known at this time. A maximum column load of 250 kips was used for purposes of this report. The new construction adjacent to existing structures may require new footings in the vicinity of existing foundation elements. Geocon Northwest should be consulted as the project design and structural information progresses to provide additional recommendations as required.

Architectural drawings, grading plans, and structural loads were not available to Geocon Northwest at the time of the preparation of this report. Therefore, several assumptions regarding the geotechnical design have been presented herein. Geocon Northwest should be consulted as the project design progresses to either confirm or modify the geotechnical design provided in this report.

3 REGIONAL GEOLOGY

Based on the U.S. Geological Survey *Geologic Map of Oregon* the site, located west of the Willamette River, is mapped as Pleistocene age lacustrine deposits of clay, silt, sand, and gravel. Review of readily available well logs completed near the project site indicates alluvial clay, silt, or sand to depths of at least 30 feet in the project area. Groundwater has been reported at 30 to 35 feet below the ground surface in the project vicinity.

4 SUBSURFACE EXPLORATION AND CONDITIONS

4.1 Site Exploration

The subsurface soil conditions at the site were determined based on the literature review, review of our geotechnical report for the Health and Wellness Center (dated April 2009), field exploration, and laboratory investigation. In particular, boring B3 from the April 2009 investigation, located near the southeast corner of the new proposed building, was included for this investigation. The field exploration was completed on October 14, 2011, and consisted of the excavation of two solid stem auger soil borings (B4 & B5). The soil borings extended to a maximum depth of approximately 21.5-feet below the ground surface (bgs) and were completed with the use of a trailer-mounted solid stem auger drill rig. A member of Geocon Northwest's geotechnical engineering staff logged the subsurface conditions encountered within the borings. Standard penetration tests (SPT) were performed by driving a 2-inch outside diameter split spoon sampler 18 inches at selected depths into the bottom of the hollow stem borings, in general accordance with ASTM D 1586. The number of blows to drive the sampler the last 12 of the 18 inches are reported on the boring logs located in Appendix A at the end of this report. The rope and cathead SPT method was used. Disturbed bag samples were obtained from SPT testing. Service providers subcontracted by Geocon Northwest completed the drilling. The approximate soil boring locations are depicted on the Site Plan, Figure 2.

4.2 Subsurface Conditions

The subsurface exploration is assumed to be representative of the subsurface conditions across the site; however, it is possible that some local variations and possible unanticipated subsurface conditions may exist. Based on the conditions observed during the reconnaissance and field exploration, the subsurface conditions, in general, consisted of the following:

ASPHALT PAVEMENT- Each exploration was completed within the asphalt parking area north of the newly constructed Health and Wellness Center. The asphalt pavement section consists of a surface layer of approximately 3.5-inches of asphalt that is underlain by 8-inches to 1.5 -feet of base rock. Underground utilities traverse through the parking area and have been backfilled with unknown material.

SILTY CLAY TO CLAYEY SILT – Underlying the asphalt pavement section, an alluvial deposit of moist to wet, mottled brown and light gray, silty clay to clayey silt was encountered. The formation was medium stiff to stiff in the upper 20 feet and increased in stiffness to very stiff consistency at a depth of 20 feet. Soil coloration changed to gray at a depth of 20 feet in boring B-3 which is indicative of continuous saturation.

GROUNDWATER –. Perched groundwater was encountered at depths ranging from 5 to 10-feet below the existing ground surface. It is estimated that groundwater levels may fluctuate seasonally and perched water may be present within the near-surface clayey soils during wet periods of the year.

5 SITE SPECIFIC SEISMIC HAZARD EVALUATION

5.1 Seismic Setting

5.1.1 Earthquake Sources

The seismicity of the general project area is controlled by three separate fault mechanisms. These include the Cascadia Subduction Zone (CSZ), the mid-depth intraplate zone, and relatively shallow crustal faults. The Cascadia Subduction Zone is located offshore and extends from Northern California to British Columbia. Within this zone the oceanic Juan De Fuca Plate is being subducted beneath the continental North American Plate to the east. The interface between these two plates is located at a depth of approximately 15 to 20 kilometers. The seismicity of the CSZ is subject to several uncertainties, including the maximum earthquake magnitude and the recurrence intervals associated with various magnitude earthquakes. Anecdotal evidence of previous CSZ earthquakes has been observed within coastal marshes along the Oregon coast (Peterson et al. 1993). Sequences of interlayered peat and sands have been interpreted to be the result of large subduction zone earthquakes occurring at intervals on the order of 300 to 500 years, with the most recent event taking place approximately 300 years ago. A study completed by Geomatrix (1995) for the Oregon Department of Transporation suggests that the maximum earthquake associated with the CSZ is moment magnitude (M_W) 8 to 9, and is supported by recent studies published by the United States Geologic Survey (USGS, 2008). This is based on an empirical expression relating moment magnitude to the area of fault rupture derived from earthquakes which have occurred within subduction zones in other parts of the world.

The intraplate or intraslab zone encompasses the portion of the subducting Juan De Fuca Plate located at a depth of approximately 20 to 40 km below Western Oregon. Very low levels of seismicity have been observed within the intraplate zone in Oregon. However, much higher levels of seismicity within this zone have been recorded in Washington and California. Several reasons for this seismic quiescence in Oregon were suggested in the Geomatrix (1995) study and include changes in the direction of subduction between Oregon and British Columbia as well as the effects of volcanic activity along the Cascade Range. Historical activity associated with the intraplate zone includes the 1949 Olympia (magnitude 7.1), the 1965 Puget Sound (magnitude 6.5), and the 2001 Nisqually (magnitude 6.8) earthquakes. Both Geomatrix (1995) and Wong et al (2000) present estimates of M_W 7.0 to 7.2 for the maximum moment magnitude of the intraslab zone.

The third source of seismicity that can result in ground shaking at the subject property is near-surface crustal earthquakes occurring within the North American Plate. The historical seismicity of crustal earthquakes in western Oregon is higher than the seismicity associated with the CSZ and the intraplate zone. The 1993 Scotts Mills (magnitude 5.6) and Klamath Falls (magnitude 6.0) were crustal earthquakes. Individual faults or fault zones, which have been mapped by the Oregon Department of Geology and Mineral Industries (1991), Geomatrix (1995) and Wong et al (2000) within the near-vicinity of the site, are indicated below in Table 2: Crustal Faults. As discussed within Wong et al (2000), the estimated maximum moment magnitude for each crustal fault was determined using empirical relationships developed by Wells and Coppersmith (1994) between rupture length, rupture area, and earthquake magnitude.

Seismic and geologic parameters such as slip rate, horizontal and vertical offset, rupture length, and geologic age have not been determined for the majority of the faults in Table 1. This is primarily due to the lack of surface expressions or exposures of faulting because of urban development and the presence of late Quaternary soil deposits that overlie the faults. The low level of historical seismicity (particularly for earthquakes greater than magnitude 5) and lack of paleo-seismic data results in large uncertainties when evaluating individual crustal fault maximum magnitude earthquakes and recurrence intervals, and limits the available knowledge of characteristic motions of estimated maximum moment earthquakes.

5.1.2 Historical Seismicity

The historical seismicity of the site and the vicinity was determined based upon the review of the September 1993 and November 1995 issues of Oregon Geology and on the analysis of the 150 year Oregon earthquake catalog, DOGAMI Open-File Report O-94-4. OFR O-94-4 is a database of 15,000 Oregon earthquakes that occurred between 1833 and October 25, 1993. In order to establish

Table 1: Crustal Faults

Mapped Fault or Fault Zone	*Probability of Activity (Wong, 2000)*	*Fault Type (Geomatrix, 1995)*	*Maximum Moment Magnitude (Wong, 2000)*	*Approx. Horizontal Distance From Site to Surface Fault Trace (miles)*
Corvallis Fault	0.3	Reverse	6.5	11
Owl Creek Fault	0.3	Reverse	6.4	18
Mount Angel Fault	0.9	Strike-slip	6.6	27
Mill Creek Fault	0.3	Reverse	6.6	11
Waldo Hills Fault	0.3	Reverse	6.5	11

an estimated Richter Magnitude for those seismic events that do not have such a recording, the Gutenberg and Richter, 1965 relationship, $M = (2/3)\,MMI + 1$, was applied to those earthquakes that only had a reported Modified Mercalli Intensity (MMI). The MMI scale is a means of estimating the size of an earthquake using human observations and reactions to the earthquake. The MMI scale ranges from I to XII, with XII representing the highest intensity. A search of the database was conducted to determine the number and estimated magnitude of earthquakes that have taken place within 50 kilometers of the site. The information derived from the Oregon earthquake catalog indicates that four M5.0 or greater earthquakes occurred within the search zone. M5.0 was the largest recorded magnitude within the 50-km search area.

5.2 Crustal Faults

Based on the literature review, there are no identified tectonic faults mapped within the boundaries of the site or within adjacent properties. Evidence was not encountered during the field investigation to suggest the presence of faults within the property. The potential for fault displacement and associated ground subsidence at the site is considered remote.

5.3 Ground Shaking Characteristics

The peak rock acceleration for the 2% chance of exceedance in 50-year event (2475-year return period) was evaluated using the 2008 United States Geological Survey's National Seismic Hazard Project. This probabilistic uniform hazard study incorporates the relative contributions of the Cascadia Subduction Zone, intraplate earthquakes within the Juan de Fuca plate, and crustal earthquakes from the North American Plate. Deaggregation of the uniform hazard data provides the relative contributions of the individual earthquake sources. The results of the probabilistic analysis indicate an estimated peak rock acceleration of 0.45g for a 2475-year return period. The deaggregation of the probabilistic uniform hazard indicates approximately 75% of the overall hazard is attributed to the Cascadia Subduction Zone with the remaining hazard contributed by shallow crustal faults. There is a negligible contribution to the seismic hazard at the site from the intraslab source.

5.4 Soil Site Class Evaluation

According to the procedures for construction of the general design response spectrum outlined in the 2010 OSSC, a soil characteristic called "Soil Site Class" is used to account for the effect of the underlying soil conditions on bedrock motion. The soil site class was determined using the subsurface information obtained during the geotechnical investigation at the subject site, and through review of published geologic literature.

Soil explorations were completed at the site to a maximum depth of 30 feet during the geotechnical field investigation. Based on the subsurface conditions encountered during the field investigation and the geological literature reviewed for the site, it is estimated that the material in the upper 100 feet below foundation level, determined in accordance with the procedures outlined in OSSC Section 1613 "Site Categorization Procedure", has an average blowcount (N) value between 15 and 50 and an average shear wave velocity between 600 and 1,200 feet per second. The preceding criteria characterize the site as Site Class D.

5.5 General Procedure for Design Response Spectrum

The 2010 OSSC contains probabilistic maps for determination of the general design acceleration response parameters. The maps have both peak ground acceleration values and spectral acceleration values at 5% structural damping. Map parameters are for a generic rock site corresponding to site class B which is defined as rock with shear wave velocity ranging from 2,500 feet per second to 5,000 feet per second. As discussed in the previous section, the site is classified as Site Class D. The mapped values were adjusted for the site class D soil conditions using the site coefficients F_A and F_V.

The adjusted ground motion parameters, designated S_{MS} for the short spectral period (0.2 second) and S_{M1} for the one-second spectral period, were used to construct the probabilistic maximum considered earthquake (MCE) response spectrum. The ground motion parameters S_{MS} and S_{M1} were multiplied by two-thirds to obtain the design ground motion parameters S_{DS} and S_{D1}. Ground motion parameters used for development of the General Horizontal Design Response Spectrum shown in Figure 3 are provided in Table 2.

5.6 Response Spectra Associated with Site-Specific Seismic Hazards

The site-specific seismic hazard evaluation procedure as required by the 2010 OSSC is included within ASCE document 7-05, *Minimum Design Loads for Buildings and Other Structures,* Chapter 21, Section 21.2. The probabilistic MCE was calculated according to Section 21.2.1 which is the same procedure used to calculate S_{MS} and S_{M1} as discussed in the previous section of this report.

Table 2: 2010 OSSC Seismic Design Recommendations

Seismic Variable	*Recommended Value*
Site Class	D
MCE short period spectral response acceleration, S_{MS}	0.98
MCE 1-second period spectral response acceleration, S_{M1}	0.64
5% damped short period spectral response acceleration, S_{DS}	0.65
5% damped 1-second period spectral response acceleration, S_{D1}	0.43

Per Section 21.2.2, the deterministic response spectra were calculated using published attenuation relationships for stiff soil sites to evaluate the specific response spectra independently associated with the Cascadia Subduction Zone and the random gridded crustal faults. The hazard associated with the crustal source was determined to be represented by a M6 earthquake at a distance of approximately 11 kilometers. The interslab source associated with the CSZ is expected to be represented by a M9 earthquake at a distance of approximately 65 kilometers. Both of these earthquake scenarios were determined through deaggregation of the seismic hazard having a 2% chance of exceedence in 50-years according to the USGS.

The Next Generation Attenuation (NGA) equations developed by Abrahamson and Silva, Atkinson and Boore, Campbell and Bozorgnia, and Chiou and Youngs were used for the crustal source while the subduction zone was evaluated by attenuation equations from Youngs et. al. and Atkinson and Boore. Each attenuation equation includes a soil input variable to account for the potential amplification or damping effect of the soil column subjected to seismic shaking. The deterministic response spectra were multiplied by 150%, compared with the deterministic lower limit (Figure 21.2-1, ASCE 7-05), and are shown in Figure 4 of this report. The results indicate that the deterministic lower limit curve exceeds the 150 percent deterministic spectra at all periods and is therefore the deterministic MCE.

According to Section 21.2.3, the site specific MCE is the lesser of the probabilistic MCE and deterministic MCE. The results of this comparison indicate that the probabilistic MCE for a Site Class D is less than the deterministic MCE up to a period of 4 seconds (maximum period calculated as part of this study). This relationship is illustrated in Figure 5. The site specific design response spectrum is two thirds of the site specific MCE which is the same response spectrum as presented in Figure 3. Therefore, it is Geocon Northwest's opinion that the General Horizontal Design Response Spectrum for site class D shown in Figure 3 be used for seismic design.

5.7 Soil Liquefaction Potential

Liquefaction can cause aerial and differential settlement, lateral spreading, loss of bearing capacity, and sudden loss in soil strength. Soils prone to liquefaction are typically loose, saturated sands and, to a lesser degree, silt. Cyclic failure can result in similar hazards to those of liquefaction, but is a phenomena related to low-strength, fine-grained silt and clay soils. When ground shaking commences, the low-strength saturated soils tend to generate excess pore water pressures. The degree of excess pore water pressure generation is largely a function of the magnitude and duration of the ground shaking, as well as the density or consistency of the soil.

The cyclic failure potential of the fine grained deposits was assessed using procedures outlined by Boulanger and Idriss, 2008. The undrained shear strength of the soils was evaluate using methods based on the SPT blow counts and grain size distribution data obtained during the geotechnical field and laboratory investigation. The seismically induced shear stresses at the site were assessed through the use of a standard-of-practice simplified empirical procedure. The analyses were conducted using the 2010 Oregon Structural Specialty Code design level earthquakes which consisted of a moment magnitude 6 for the crustal source and moment magnitude 8.5 for a subduction zone event. Peak ground surface acceleration values of 16% and 23% gravity (0.16g and 0.23g) were used for the crustal and subduction zone earthquakes, respectively. The peak ground acceleration values for each earthquake were estimated using attenuation relationships for stiff soil sites independently associated with individual crustal faults as well as the Cascadia Subduction Zone. Based on the results of our analyses, cyclic failure potential at the site is considered remote. There is not a liquefaction hazard at the site due to lack of sandy soil within the subsurface profile and depth to groundwater.

5.8 Landslide Hazard

The site, in the location of proposed development, is flat and not susceptible to slope instability. Therefore, it is Geocon Northwest's opinion the landslide hazard at the site is negligible.

5.9 Lateral Spreading

Lateral spreading is a liquefaction related seismic hazard that may adversely impact some sites. Areas subject to lateral spreading are underlain by liquefiable sediments and are sloping sites or flat sites adjacent to an open face. Due to the lack of liquefiable material and flat topography of the site, there is not a potential for lateral spreading.

5.10 Seiche and Tsunami Inundation

There is not a potential for seiche- and tsunami-related damage at the site due to the distance of the site from lakes and coastal areas.

5.11 Seismic Lateral Earth Pressures on Retaining Walls

Seismic activity can generate increased lateral earth pressures acting on retaining walls, including basement walls. The increase is influenced by the horizontal ground acceleration and the allowable movement of the wall. A horizontal acceleration of 0.15g and an allowable wall movement greater than 0.001 times the wall height to produce active pressures was used to determine seismically induced wall pressures. Based on Mononobe-Okabe procedures for a vertical wall with horizontal backfill, the *additional* lateral pressures due to earthquake motions should be based on an equivalent fluid weight of 15 pcf. The results of the additional dynamic earth pressure should be applied at a height of 0.6H above the wall base.

6 LABORATORY TESTING

Laboratory testing was performed on selected soil samples to evaluate moisture content, plasticity, and gradation. Visual soil classification was performed both in the field and laboratory, in general accordance with the Unified Soil Classification System. Moisture content determinations (ASTM D2216) were performed on soil samples to aid in classifying the soil. Grain size analyses were performed on selected samples using procedures ASTM D1140 and ASTM D422. The plasticity index was determined in general accordance with ASTM D4318. Moisture contents are indicated on the boring logs and are located in Appendix A of this report. Other laboratory test results for this project are summarized in Appendix B.

7 CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 It is our opinion that the proposed Western Oregon University New Science Center is geotechnically feasible, provided the recommendations of this report are followed. Architectural drawings, grading plans, and structural loads were not available to Geocon Northwest at the time of the preparation of this report. Therefore, several assumptions regarding the geotechnical design have been presented herein. Geocon Northwest should be consulted as the project design progresses to either confirm or modify the geotechnical design provided in this report.

7.1.2 The surface layer of pavement, topsoil, undocumented fill soil (if encountered), and associated above and below grade improvements (utilities) are unsuitable for structural foundation or pavement support. Recommendations for removal of existing improvements are provided in subsequent sections of this report.

7.1.3 Moisture contents of the near-surface soils were above optimum at the time of the investigation, therefore recommendations for both wet and dry weather construction are provided herein. **However, dry weather construction is highly recommended and extra costs should be expected if site grading is completed during wet weather.**

7.1.4 Perched water was encountered in the near-surface clayey silt to silty clay during the field investigation. Recommendations regarding drainage and vapor retarders are provided in subsequent sections of this report. An underslab drainage system is recommended if the elevation of the lower level floor will be one-foot or more below the existing ground surface.

7.1.5 The near-surface soils at the site could be used as structural fill if properly moisture conditioned and compacted. However, difficult placement operations should be expected due to the high moisture content of this material. Specifications for both native and granular import material for use as structural fill are provided. Near-surface soils may be efficiently used in non-structural areas (landscaping, field areas, etc.).

7.1.6 New foundation elements are anticipated in the near vicinity of the footings for the existing structures. Geocon Northwest should be consulted as project plans and structural loads have been determined to evaluate the potential surcharging or undermining effect of new footings on existing foundations.

7.2 Site Preparation

7.2.1 Prior to beginning construction, the areas of the site to receive fill, footings, structural improvements or pavement should be stripped of concrete, asphalt, vegetation, topsoil, non-engineered fill, previous subsurface improvements, debris, and otherwise unsuitable material, down to firm native soil. Localized areas of undocumented fill, particularly associated with existing underground utilities, may be encountered across the site and should be removed during site preparation and backfilled with crushed rock or controlled density fill (CDF). Excavations made during site preparation should be backfilled with structural fill per Section 7.4 of this report.

7.2.2 Recommendations for both dry weather and wet weather construction are provided in the following sections. However, due to the moisture sensitive near-surface soils, it is recommended that the site be prepared during dry weather. **Additional costs, above those that will be required during 'dry' weather, will likely be incurred if site grading occurs during periods of wet weather.**

7.2.3 The existing parking area is underlain by 1.5 to 2.5 feet of base rock. It is recommended that the existing pavement be left in place during construction, where practical, to provide a working surface in locations not planned for development.

7.2.4 Dry Weather Construction

Subgrades in pavement and structural areas that have been disturbed during stripping or cutting operations should be scarified to a depth of at least 8-inches. The scarified soil should be moisture conditioned as necessary to achieve the proper moisture content, then compacted to at least 92% of the maximum dry density as determined by ASTM D-1557. Minimum compaction for the 8 inches immediately underlying pavement sections should be 95%. Even during dry weather it is possible that some areas of the subgrade will become soft or may "pump," particularly in poorly drained areas. Soft or wet areas that cannot be effectively dried and compacted should be prepared in accordance with Section 7.2.5.

7.2.5 Wet Weather Construction

During wet weather, or when adequate moisture control is not possible, it may be necessary to install a granular working blanket to support construction equipment and to provide a firm base on which to place subsequent fills and pavements. Commonly, the working blanket consists of a bank run gravel or pit run quarry rock (6 to 8-inch maximum size with no more than 5% by weight passing a No. 200 sieve). A member of Geocon Northwest's engineering staff should be contacted to evaluate the suitability of the material before installation.

The working blanket should be installed on a stripped subgrade in a single lift with trucks end-dumping off an advancing pad of granular fill. It should be possible to strip most of the site with careful operation of track-mounted equipment. However, during prolonged wet weather, or in particularly wet locations, operation of this type of equipment may cause excessive subgrade disturbance. In some areas, final stripping and/or cutting may need to be accomplished with a smooth-bucket trackhoe, or similar equipment, working from an advancing pad of granular fill. After installation, the working blanket should be compacted by a minimum of four complete passes with a moderately heavy static steel drum or grid roller. It is recommended that Geocon Northwest be retained to observe granular working blanket installation and compaction.

The working blanket must provide a firm base for subsequent fill installation and compaction. Past experience indicates that about 18 inches of working pad is normally required. This assumes that the material is placed on a relatively undisturbed subgrade prepared in accordance with the preceding recommendations. Areas used as haul routes for heavy construction equipment may require a work pad thickness of 2 feet or more.

In particularly soft areas, a heavy-grade, non-woven, non-degradable filter fabric or biaxial geogrid installed on the subgrade may reduce the thickness of working blanket required.

Construction practices can affect the amount of work pad necessary. By using tracked equipment and special haul roads, the work pad area can be minimized. The routing of dump trucks and rubber tired equipment across the site can require extensive areas and thicknesses of work pad. Normally, the design, installation and maintenance of a work pad are the responsibility of the contractor.

7.3 Proof Rolling

7.3.1 Regardless of which method of subgrade preparation is used (i.e., wet weather or dry weather), it is recommended that, prior to on-grade slab or pavement construction, the subgrade or granular working blanket be proof-rolled with a fully-loaded 10- to 12-yard dump truck. Areas of the subgrade that pump, weave, or appear soft or muddy should be scarified, dried and compacted, or overexcavated and backfilled with structural granular fill per Section 7.4. If a significant length of time passes between fill placement and commencement of construction operations, or if significant traffic has been routed over these areas, the subgrade should be similarly proof-rolled before slab construction. It is recommended that a member of our geotechnical engineering staff observe the proof-roll operation.

7.4 Fills

7.4.1 Structural fills should be constructed on a subgrade that has been prepared in accordance with the recommendations in Section 7.2 of this report. Structural fills should be installed in horizontal lifts not exceeding approximately 8 inches in thickness and should be compacted to at least 92% of the maximum dry density for the native soils, and 95% for imported granular material. Imported granular material is recommended for structural fills in all but the driest extended periods. Compaction should be referenced to ASTM D-1557 (Modified Proctor). The compaction criteria may be reduced to 85% in landscape, planter, or other non-structural areas.

7.4.2 During dry weather when moisture control is possible, structural fills may consist of native material, free of topsoil, debris and organic matter, which can be compacted to the preceding specifications. The native, non-organic soil would be acceptable for structural fills during extended dry periods if properly moisture conditioned. Based on our experience with similar soil types, the optimum moisture content for the near-surface silty clay soil is approximately 17% to 20% at a maximum dry density of approximately 110 pcf (Modified Proctor). Due to the relatively high in situ moisture contents of the near surface soils, generally between 35% and 40%, it is estimated that proper moisture conditioning of the material would require significant aeration due to the high clay content of these soils. Therefore, Geocon Northwest recommends that, for the majority of the year, structural fills consist of imported, well-graded, angular, granular soils (sand or sand and gravel) that do not contain more than 5% material by weight passing the No. 200 sieve. It is usually desirable to limit this material to a maximum of six inches in diameter for future ease in the installation of utilities.

7.5 Surface and Subsurface Drainage

7.5.1 During site contouring, positive surface drainage should be maintained away from foundation and pavement areas. Additional drainage or dewatering provisions may be necessary if soft spots, springs, or seeps are encountered in subgrades. Where possible, surface runoff should be routed independently to a storm water collection system. Surface water should not be allowed to enter subsurface drainage systems.

7.5.2 Drainage and dewatering systems are typically designed and constructed by the contractor. Failure to install necessary subsurface drainage provisions may result in premature foundation or pavement failure.

7.6 Foundations

7.6.1 At this time no structural information is available for the proposed building loads. The depth of footings and location with respect to existing foundation elements are important factors that will have to be evaluated as the project design progresses.

7.6.2 New foundation elements are anticipated in the near vicinity of the footings for the existing structures. Geocon Northwest should be consulted as project plans and structural loads have been determined to evaluate the potential surcharging effect of new footings on existing foundations.

7.6.3 Preliminary foundation bearing pressure analyses were completed assuming a column load of 250 kips and continuous wall load of 4 kips/foot. Spread and perimeter foundation support for the proposed New Science Center may be obtained from the near-surface, non-organic native soil or from structural fill installed in accordance with the previous recommendations.

7.6.4 If unsuitable fill soils, or soft, saturated soil are encountered at footing elevation, the unsuitable soils should be overexcavated and backfilled with compacted crushed rock. Undocumented fill soils should be anticipated where existing underground utilities are encountered.

7.6.5 Spread and perimeter footings should be at least 2 feet wide and should extend at least 18 inches below the lowest adjacent pad grade. Foundations supported on firm native soils or engineered fill may be designed for an allowable soil bearing pressure of 2,500 pounds per square foot (psf).

7.6.6 Foundation subgrades that are anticipated to be exposed to inclement weather prior to concrete placement should be protected to prevent future overexcavation of unsuitable soil.

7.6.7 Gravel or lean concrete may need to be placed in the bottom of the footing excavation to reduce soil disturbance during foundation forming and construction.

7.6.8 Lateral loads may be resisted by sliding friction and passive pressures. A base friction of 40% of the vertical load may be used against sliding. An equivalent fluid weight of 325 pcf may be used to evaluate the passive resistance to lateral loads.

7.6.9 Foundation settlements for the loading conditions expected for this project are estimated to be less than one inch, with not more than one-half inch occurring as differential settlement.

7.6.10 Geocon Northwest recommends that foundation drains be installed at or below the elevation of perimeter footings to intercept potential subsurface water that may migrate under the building area.

7.7 Concrete Slabs-on-Grade

7.7.1 Subgrades in floor slab areas should be prepared in accordance with Section 7.2 of this report. Floor slab areas should be proof-rolled with a fully loaded 10- to 12- yard dump truck to detect areas that pump, weave, or appear soft or muddy. When detected these areas should be overexcavated and stabilized with compacted granular fill.

7.7.2 The elevation of the lower level of the proposed New Science Center was not known at the time of the preparation of this report. Perched groundwater was encountered within the near surface fine grained soil during the geotechnical investigation and, as such, an underslab drainage system is recommended if the finish grade of the lower level will be more than 12 inches below the existing ground surface. An underslab drainage system should be constructed with a minimum 12-inch thick layer of drain rock (less than 5% by weight passing the No. 200 Sieve). It is recommended that the underslab drainage system consist of 4-inch diameter PVC perforated pipe placed within the drain rock at 10- to 15-foot centers beneath the building footprint. The perforated PVC pipe should be wrapped with a geotextile filter fabric to protect against possible siltation. The underslab drainage system should be constructed to drain by gravity.

7.7.3 A minimum 6-inch thick layer of compacted ¾-inch minus material should be installed over the prepared subgrade to provide a capillary break and to minimize subgrade disturbance during construction. The crushed rock or gravel material should be poorly graded, angular, and contain no more than 5% by weight passing the No. 200 Sieve. This 6-inches of crushed rock should be place above the 12 inches of drain rock associated with the underslab drainage system recommended in the previous paragraph (where required).

7.7.4 A modulus of subgrade reaction of 125 pci is recommended for design.

7.7.5 The fine-grained near-surface soils at the site have high natural moisture contents and low permeability. These characteristics indicate that high ground moisture may develop under floor slabs during the life of the project. The difference in moisture content and temperature between the air in the subgrade soil and the air in the finished building may create a water vapor pressure differential between the two environments. This pressure differential can force migration of moisture through the slab. This migration of moisture can result in the loosening of flooring materials attached with mastic, the warping of wood flooring, stained concrete, and in extreme cases, mildewing of carpets and building contents. To retard the migration of moisture through the floor slab, Geocon Northwest recommends installing a 10-mil polyethylene vapor retarding membrane below the concrete slab. Installation of the membrane should be in conformance with product manufacturer's specifications. A minimum 6-inch under-slab section of crushed rock should be placed as a capillary break above the subgrade and below the vapor retarder. Any moisture that has accumulated on the vapor retarding membrane should be removed prior to the concrete pour.

7.8 Utility Excavations

7.8.1 Based on the subsurface explorations, difficult excavation of the near-surface silts and clay is not anticipated.

7.8.2 The contractor should comply with applicable local, state, and federal safety regulations, including current OHSA excavation and trench safety standards, during utility installation. Excavations deeper than four feet, or those that encounter groundwater, should be sloped or shored in conformance with OSHA regulations. Shoring systems are typically contractor designed.

7.8.3 Perched groundwater should be anticipated to approach the ground surface, particularly during prolong periods of wet weather. Therefore, excavation dewatering may be necessary if substantial flow of groundwater is encountered. Dewatering systems are typically designed and installed by the contractor.

7.8.4 Utilities should be bedded in sand within one conduit diameter in all directions, prior to the placement of coarser backfill. Trench backfill should be lightly compacted within two diameters or 18 inches, whichever is greater, above breakable conduits. The remaining backfill, to within 12 inches of finished grade, should be compacted to 92% of the maximum dry density as determined by ASTM D1557. In structural areas, the upper foot of backfill should be compacted to 95% of the maximum dry density.

7.9 Pavement Design

7.9.1 Near surface soil samples were evaluated to determine pavement design parameters. A CBR of 3 at 95% compaction and a resilient modulus of 4500 psi were used for pavement design based on our experience with similar soils.

7.9.2 Alternate pavement designs for both asphalt and portland cement concrete (pcc) are presented in Tables 4 and 5. Pavement designs have been prepared in accordance with accepted AASHTO design methods. A range of pavement designs for various traffic conditions is provided in the tables. The designs assume that the top eight inches of pavement subgrade will be compacted to 95% of ASTM D-1557. Specifications for pavement and base course should conform to current Oregon Department of Transportation specifications. Additionally, the base rock should contain no more than 5% by weight passing a No. 200 Sieve, and the asphaltic concrete should be compacted to a minimum of 91% of ASTM D2041. **A non-woven geotextile fabric should be placed between the subgrade soil and base rock.**

7.9.3 Pavement sections were designed using AASHTO design methods, with an assumed reliability level (R) of 90%. Terminal serviceability of 2.0 for asphaltic concrete, and 2.5 for portland cement concrete were assumed. The 18 kip design axle loads are estimated from the number of trucks per day using Federal Highway Administration typical axle distributions for truck traffic and AASHTO load equivalency factors, and assuming a 20 year design life. The concrete designs were based on a modulus of rupture equal to 550 psi, and a compressive strength of 4000 psi. The concrete sections assume plain jointed or jointed reinforced sections with no load transfer devices at the shoulder.

7.9.4 It is important to note that these pavement design recommendations do not include an allowance for construction traffic. If paving is planned prior to the completion of heavy construction, the construction traffic (i.e. concrete trucks) should be limited to unpaved and untreated roadways, or specially constructed haul roads. If this is not possible, the pavement design should include an allowance for construction traffic.

Table 4: Asphalt Concrete Pavement Design

Approximate Number of Trucks per Day (each way)	*Approximate Number of 18 Kip Design Axle Load (1000)*	*Asphalt Concrete Thickness (inches)*	*Crushed Rock Base Thickness (inches)*
Auto Parking	10	2.5	8
5	22	3.0	8
10	44	3.0	10
15	66	3.5	10
25	110	4.0	10
50	220	4.0	12
100	440	4.5	12
150	660	5.0	13

Table 5: Portland Cement Concrete Pavement Design

Approximate Number of Trucks per Day (each way)	*Approximate Number of 18 Kip Design Axle Load (1000)*	*P.C.C. Thickness (inches)*	*Crushed Rock Base Thickness (inches)*
25	110	6.0	6
50	220	7.0	6
100	440	8.0	6
150	660	8.5	6
200	880	8.5	6
250	1100	9.0	6

8 FUTURE GEOTECHNICAL SERVICES

The analyses, conclusions and recommendations contained in this report are based on site conditions as they presently exist, and on the assumption that the subsurface investigation locations are representative of the subsurface conditions throughout the site. It is the nature of geotechnical work for soil conditions to vary from the conditions encountered during a normally acceptable geotechnical investigation. While some variations may appear slight, their impact on the performance of the proposed improvements can be significant. Therefore, it is recommended that Geocon Northwest be retained to observe portions of this project relating to geotechnical engineering, including site preparation, grading, and compaction. This will allow correlation of observations and findings to actual soil conditions encountered during construction and evaluation of construction conformance to the recommendations put forth in this report.

A copy of the plans and specifications should be forwarded to Geocon Northwest so that they may be evaluated for specific conceptual, design, or construction details that may affect the validity of the recommendations of this report. The review of the plans and specifications will also provide the opportunity for Geocon Northwest to evaluate whether the recommendations of this report have been appropriately interpreted.

9 LIMITATIONS

Unanticipated soil conditions are commonly encountered during construction and cannot always be determined by a normally acceptable subsurface exploration program. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Northwest, Inc. should be notified so that supplemental recommendations can be given.

This report is issued with the understanding that the owner, or his agents, will ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, the conclusions and recommendations provided in this letter are subject to review should such changes occur.

N
NO SCALE

GEOCON
NORTHWEST INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS
8283 SW CIRRUS DRIVE - BEAVERTON, OREGON 97008 - 6443
PHONE 503 626-9889 - FAX 503 626-8611

VICINITY MAP

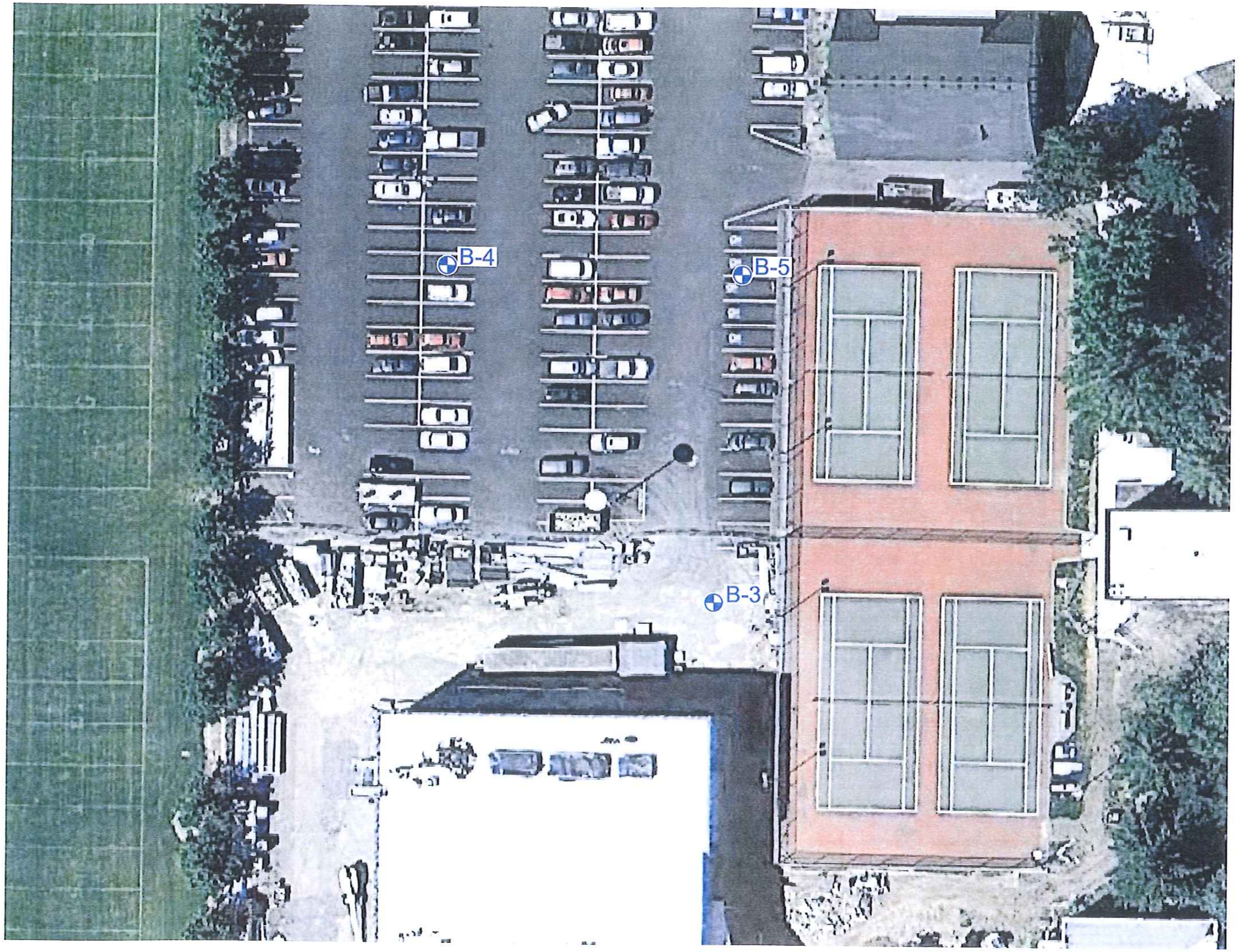
WESTERN OREGON UNIVERSITY
THE NEW SCIENCE CENTER
MONMOUTH, OREGON

SMD / AML		DSK/E0000

DATE October 2011	PROJECT NO. P1844 - 05 - 01	FIG. 1

Vicinity Map

WESTERN OREGON UNIVERSITY
THE NEW SCIENCE CENTER
MONMOUTH, OREGON



APPROX. SCALE: 1" = 40'

GEOCON LEGEND

B-5 ⊕........APPROX. LOCATION OF SOIL BORING

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SITE PLAN

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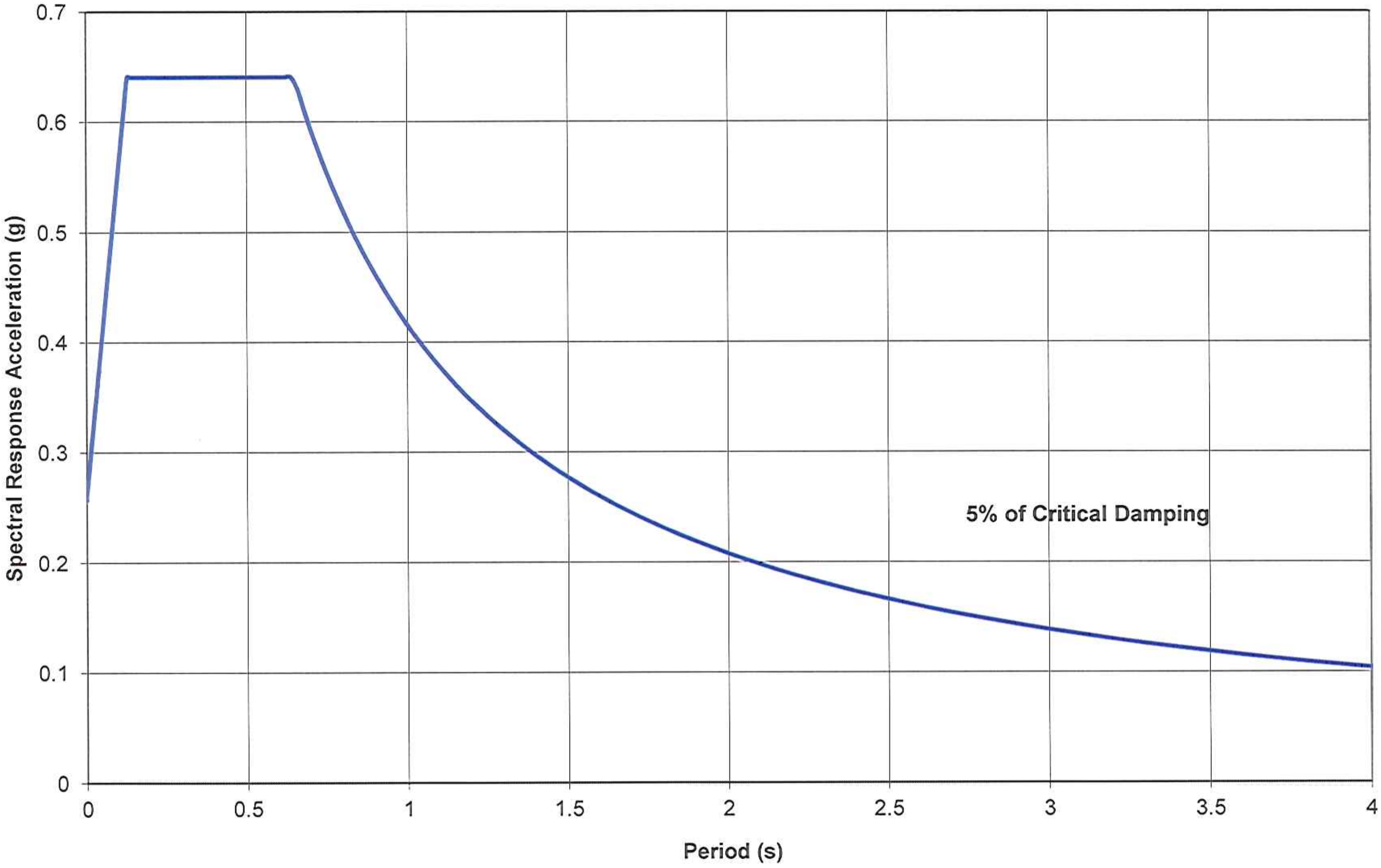
8283 SW CIRRUS DRIVE - BEAVERTON, OREGON 97008 - 6443
PHONE 503 626-9889 - FAX 503 626-8611

DATE October 2011 | PROJECT NO. P1844 - 05 - 01 | FIG. 2

Y:\PROJECTS-Portland\P1844-05-01\SHEETS\P1844-05-01 Map.dwg



2010 Oregon Structural Specialty Code Site Class D Design Response Spectrum
Western Oregon University New Science Center

Figure 3

Western Oregon University New Science Center
Deterministic Maximum Considered Earthquake (MCE) Response Spectra

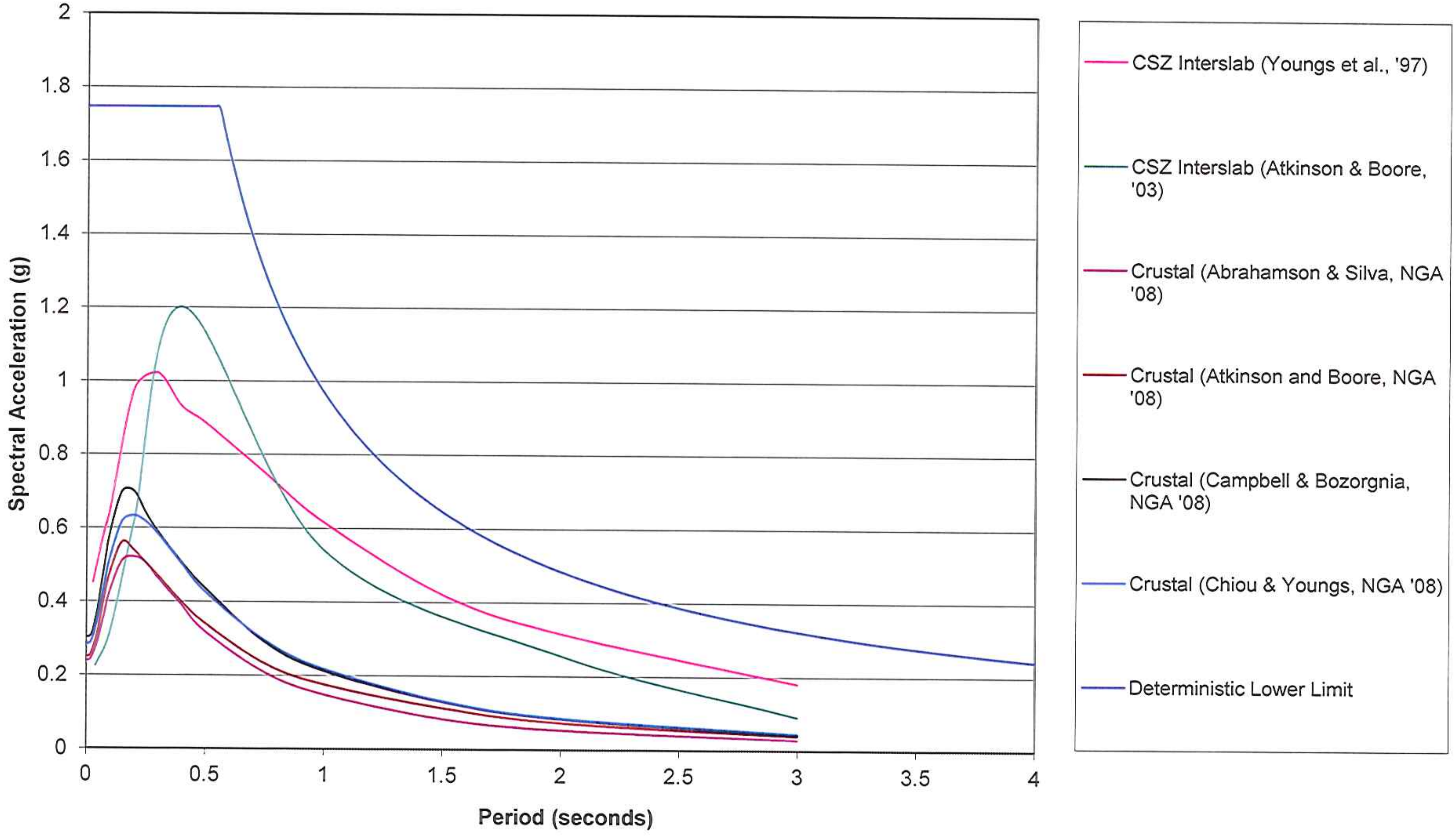


Figure 4

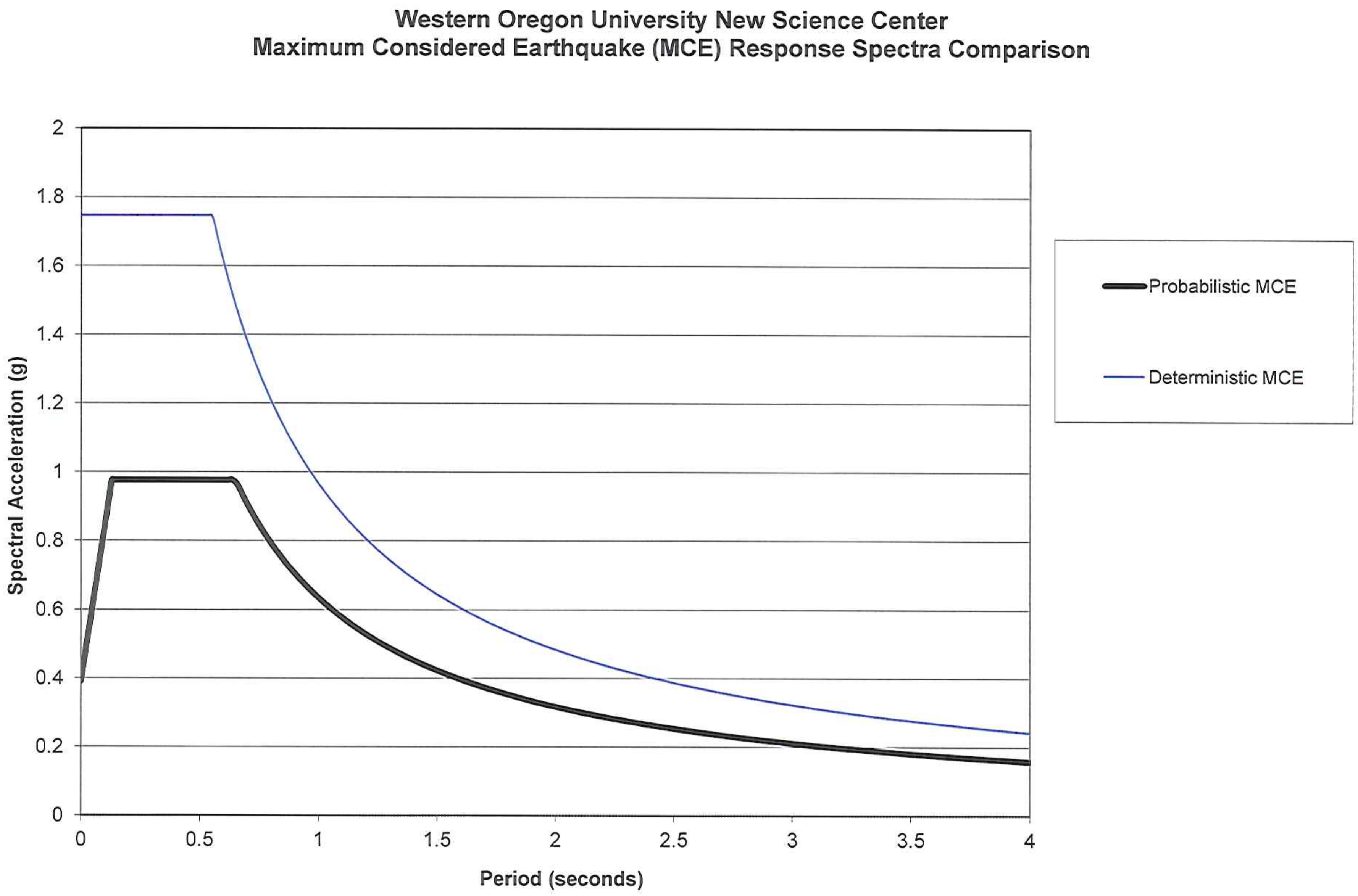

Western Oregon University New Science Center
Maximum Considered Earthquake (MCE) Response Spectra Comparison
Spectral Acceleration (g)
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
0.5
1
1.5
2
2.5
3
3.5
4
Period (seconds)
Probabilistic MCE
Deterministic MCE

Figure 5

APPENDIX A

FIELD INVESTIGATION

The field exploration was completed on October 14, 2011, and consisted of the excavation of two solid stem auger soil borings (B4 & B5). The soil borings extended to a maximum depth of approximately 21.5- feet below the ground surface (bgs) and were completed with the use of a trailer mounted solid stem auger drill rig. A member of Geocon Northwest's geotechnical engineering staff logged the subsurface conditions encountered within the borings. Standard penetration tests (SPT) were performed by driving a 2-inch outside diameter split spoon sampler 18 inches at selected depths into the bottom of the hollow stem borings, in general accordance with ASTM D 1586. The number of blows to drive the sampler the last 12 of the 18 inches are reported on the boring logs located in Appendix A at the end of this report. The rope and cathead SPT method was used. Disturbed bag samples were obtained from SPT testing. Service providers subcontracted by Geocon Northwest completed the drilling. The approximate soil boring locations are depicted on the Site Plan, Figure 2.

Subsurface logs of the conditions encountered are presented in the following pages. Also included is boring B3 from the April 2009 investigation for the Health and Wellness Center. Boring B3 was excavated near the southeast corner of the proposed new building. Both solid and dashed contact lines indicated on the logs are inferred from soil samples and drilling characteristics, and should be considered approximate.

PROJECT NO. P1667-05-01

BORING B 3

ELEV. (MSL.) _______ DATE COMPLETED **03-10-2009**

EQUIPMENT **B-57 MUD** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					2" ASPHALT over BASE ROCK			
2								
	B3-1			ML-CL	**NATIVE ALLUVIUM** Stiff, moist, brown, Clayey SILT to Silty CLAY	17		27.7
4								
	B3-2					8		42.7
6								
8								
10								
	B3-3					9		37.9
12								
				CL	Stiff, moist, brown and gray, CLAY			
14								
	B3-4					8		42.1
16								
18								
20								
	B3-5				-Becomes gray	15		43.7
					BORING TERMINATED AT 21.5 FEET Static groundwater not encountered			

Figure A-3,
Log of Boring B 3, Page 1 of 1

P1667-05-01.GPJ

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

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

GEOCON

PROJECT NO. P1844-05-01

BORING B 4

ELEV. (MSL.)________ DATE COMPLETED **10-14-2011**

EQUIPMENT **TRAILER HSA** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				**3.5" ASPHALT OVER 8"BASE ROCK**			
			CH	**WILLAMETTE SILT** Medium stiff, moist, light brown, Silty CLAY			
2							
	B1-1				7		37.8
4							
	B1-2				6		40.4
6							
8	B1-3			-Becomes moist to wet	4		40.0
10		∇					
	B1-5			-Becomes soft, wet to saturated	3		38.8
12							
	B1-4			-Becomes medium stiff	4		43.7
14							
	B1-6				4		41.8
16							
				Stiff to very stiff, wet to saturated, gray, CLAY			
18							
20							
	B1-7				17		38.0

BORING TERMINATED AT 21.5 FEET
Groundwater encountered at 10 feet

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

P1844-05-01.GPJ

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

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

PROJECT NO. P1844-05-01

BORING B 5

ELEV. (MSL.)_______ DATE COMPLETED **10-14-2011**

EQUIPMENT **TRAILER HSA** BY: **S. DIXON**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**3.5" ASPHALT OVER 1.5"BASE ROCK**			
				ML/CL	**WILLAMETTE SILT** Medium stiff, moist, light brown, SILT with clay			
2								
	B2-1					7		36.6
4								
	B2-3					6		41.7
6								
8	B2-3				-Becomes soft	3		39.7
10	B2-4				-Becomes medium stiff, moist to wet	4		37.8
12								
14								
	B2-5					7		40.3
16								
18					Very stiff, wet, lighy gray, CLAY			
20	B2-6					20		51.5

BORING TERMINATED AT 21.5 FEET
Groundwater not encountered

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

P1844-05-01.GPJ

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

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

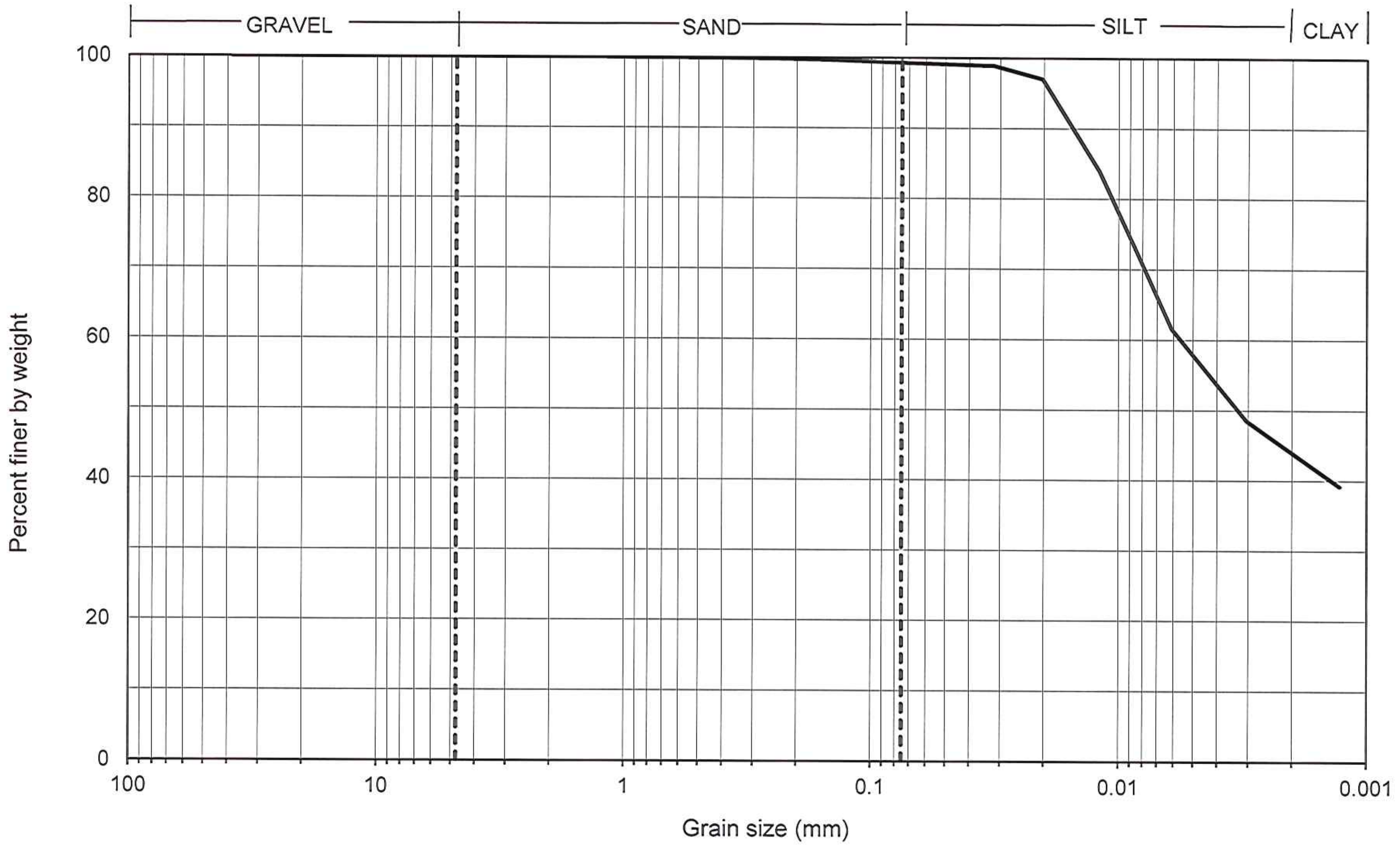
APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected soil samples were tested for their moisture content, plasticity, and gradation. Moisture contents are indicated on the boring logs in Appendix A. The results of the remaining laboratory tests performed are presented in following pages.

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

Sample Number	*Depth (ft)*	*Liquid Limit*	*Plastic Limit*	*Plasticity Index*	*USCS Classification*
B4-2	5	53	24	29	CH
B5-3	7.5	35	27	8	CL/ML



Grain Size Distribution (ASTM D1140 and D 422)
Western Oregon University - New Science Center
Sample B4-2 Depth = 5 feet

Grain Size Distribution (ASTM D1140 and D 422)
Western Oregon University - New Science Center
Sample B5-3 Depth = 7.5 feet