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GEOTECHNICAL REPORT
Cedarwood School Site Improvements
Portland, Oregon

Geotech Solutions Inc.

December 7, 2007

GSI Project: Cedarwood-07-01-gi

December 7, 2007 *Cedarwood-07-01-gi*

Cedarwood School
c/o Richard Brown Architects
239 NW 13th Avenue, Room 305
Portland, Oregon 97209
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Attention: Andrew Bernaus

GEOTECHNICAL ENGINEERING REPORT
Cedarwood School Site Improvements – Portland, Oregon

INTRODUCTION

We appreciate the opportunity to present this Geotechnical Engineering Report for the proposed site improvements to the Cedarwood School in Portland, Oregon. The existing school building, attached 'annex' building, and associated parking lot are located at 3030 SW Second Avenue in Portland, Oregon. Site improvements also include an existing building and parking lot at 3015 SW First Avenue. Our services were completed in accordance with our agreement dated October 3, 2007.

Based on preliminary project information provided by Richard Brown Architects (RBA) and TM Rippey Consulting Engineers (TMR), we understand that the 'annex' building will be demolished and the area will be redeveloped as a parking lot. The existing building at the northeast corner of the block (3015 SW First Avenue) will be added to the school campus and a new outdoor play area will be constructed immediately east of the new parking area and south of the house at 118 SW Porter Street.

Based on existing grades and proposed site improvements, we anticipate that site grading will generally be limited to cuts and fills less than 5 feet. The addition of an elevator to the existing school building may require an excavation up to 25 feet deep for construction of the elevator shaft. We understand that the 'annex' building houses an in-ground swimming pool that will be demolished and backfilled during construction of the parking lot. We anticipate that the pool and associated utilities could extend to depths greater than 12 feet below surrounding floor grades.

On-site infiltration of storm water is proposed through use of new infiltration swales, trenches, and/or dry wells to be constructed in the vicinity of the new parking area. We anticipate that the base of swales and soakage trenches will be between 5 and 10 feet deep and dry wells will extend to a depth of approximately 25 feet (relative to existing grades).

We understand that a seismic upgrade is planned for the building at 3015 SW First Avenue and the building is classified as a 'special occupancy structure' by the International Building Code (IBC). A site-specific hazard study is therefore required and is included as an appendix to this report.

PURPOSE AND SCOPE

The purpose of our services was to explore subsurface conditions at the site and provide geotechnical engineering recommendations for design and construction of the proposed site improvements. Our specific scope of services included the following:

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Geotechnical Investigation and Report

- Provide senior-level project management including management of field and subcontracted services, report writing, analyses, and invoicing.
- Review previous reports, geologic maps and vicinity geotechnical information available in our files as indicators of subsurface conditions.
- Complete a site reconnaissance to observe surface features relevant to geotechnical issues, such as topography, vegetation, presence and condition of springs, exposed soils and rock, and evidence of previous grading.
- Identify exploration locations and complete One-Call utility locates for location of public utilities. Subcontract a private locator for location of private on-site utilities.
- Explore subsurface conditions utilizing one cone penetrometer test probe (CPT) and one boring. The CPT was advanced to 66.9 feet with shear wave velocity testing at 2-meter intervals (for seismic recommendations and the site specific study) and a pore pressure dissipation test (to evaluate groundwater depth). The boring was advanced to a depth of 36.5 feet using hollow-stem auger drilling methods.
- Classify and sample materials encountered and maintain a detailed log of the explorations.
- Determine the moisture content of selected samples obtained from the explorations and complete soil classification testing as necessary.
- Provide recommendations for earthwork including site stripping and preparation, seasonal material usage, use of granular working pads, temporary and permanent cut and fill slope inclinations, and fill preparation and compaction.
- Provide recommendations for re-use of demolition materials (if planned) and backfill of the swimming pool excavation.
- Provide recommendations related to the seismic upgrade of the existing structure including seismic bearing pressures, sliding coefficient, and site class.
- Provide recommendations for retaining wall design including lateral earth pressures, drainage, and foundations as needed.
- Provide recommendations for parking area pavements including subgrade preparation and stabilization, and base rock and asphalt concrete thicknesses. Pavement design recommendations will be based on traffic information provided by others.
- Provide a written report summarizing the results of our geotechnical evaluation.

Infiltration Testing

- Complete falling-head infiltration testing at two depths in the proposed boring. Shallow infiltration rates were evaluated at 6.0 feet for design of swales and/or trenches and deeper infiltration rates were evaluated at 25.0 feet for design of dry wells.
- Provide geotechnical recommendations for infiltration system design, including estimated infiltration rates, embedment, infiltration strata, and backfill materials. Actual system design will be completed by others based on storm water volumes and site configurations.

Site-Specific Hazard Study

- Review geologic information available in our files regarding site geologic setting, nearby faults, seismic sources and related ground motions.
- Evaluate seismic hazards including potential for liquefaction, lateral spread, amplification, fault surface rupture, and seismic elements for hazard evaluation to the degree of complexity compatible with the project and required by the IBC code.
- Provide estimates of ground deformations due to liquefaction and lateral spread and qualitative methods of reducing impacts of deformations to the site structures if necessary.

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- Select earthquake models and scale them to represent general expected motions from relevant earthquakes.
- Using the preceding information, complete computer modeling of ground motions using the program PROSHAKE.
- Present the results of our analyses, including response spectra for the modeled ground motions.
- Provide comment on the appropriateness of the use of the calculated spectra relative to the possible variation in the input parameters.
- Provide the results of our site-specific hazard study in an appendix to our geotechnical report.

SITE OBSERVATIONS AND CONDITIONS

Surface Conditions

The site consists of the existing Cedarwood School building, an attached 'annex' building, two parking areas, and the building located at 3015 SW First Avenue as shown on the attached ***Site Plan***. With the exception of a house located at 118 SW Porter Street, the site occupies the southwest, northwest, and northeast quarters of a city block. The block is bordered by SW Porter Street to the north, SW First Avenue to the east, SW Woods Street to the south, and the SW Second Street right-of-way to the west.

The site improvement areas are currently occupied by buildings except for the asphalt concrete covered parking area at the northwest corner of the block and the portland cement concrete pavement driveway and parking area located west of the 3015 SW First Building. All buildings include basements or first floors partially embedded below adjacent grades. Concrete sidewalks are present on all sides of the block.

Ground surface elevations surrounding the site generally slope downward to the east from 170 to 171 feet along the west side of the block to approximately 156 feet at the northeast corner of the block and 162 feet at the southeast corner of the Cedarwood School building. The preceding ground elevations are based on topographic survey information provided RBA and a survey completed by ZTec Engineers, Inc.

Subsurface Conditions

General – The site was explored on November 20, 2007 by advancing one cone penetrometer (CPT) probe to a depth of 66.9 feet (P-1) and a drilled boring (B-1) to a depth of 36.5 feet at the approximate locations shown on the attached ***Site Plan***. The boring was completed using hollow-stem auger drilling methods. Standard penetration tests (SPT) and sampling were completed at 2.5 to 5.0 foot intervals using a split spoon sampler. Encountered subsurface conditions are described below and shown on the attached ***Boring Log***. Data from the CPT are shown on the attached ***Cone Penetrometer Log***.

Asphalt concrete and base rock thicknesses encountered in boring B-1 were 2 and 5 inches, respectively. Portland cement concrete and base rock thicknesses at the location of the CPT (P-1) were 8 and 12 inches, respectively. Beneath the pavement and base rock, subsurface conditions encountered in boring B-1 generally consisted of stiff, brown silt to a depth between 21.5 to 25.0 feet underlain by medium dense, brown, silty fine sand to the depths explored (36.5 feet). The silt layer included trace amounts of fine sand at 7.5 feet below the existing ground surface (bgs), some amounts of fine sand at 15.0 feet bgs, and became stiff to very stiff and sandy at 20.0 feet bgs. Similar subsurface conditions were encountered by the CPT at P-1 including refusal at 66.9 feet bgs.

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Laboratory Testing – Laboratory testing resulted in moisture contents of between 28 and 34 percent in the silt unit (6 samples) and between 22 and 25 percent (3 samples) in the underlying silty fine sand unit. Moisture contents at and below infiltration test depths may have been influenced by water added to the boring for infiltration testing. Fines content testing resulted in 77, 60, and 46 percent passing the #200 sieve from samples obtained at 15.0, 20.0, and 25.0 feet, respectively.

Groundwater – Groundwater levels were not measured directly in our boring due to the addition of water for infiltration testing but piezometric tests in P-1 indicated a groundwater level of 41.1 feet bgs. We anticipate that ground water levels will fluctuate with the seasons and shallow perched ground water conditions could exist during extended periods of wet weather.

CONCLUSIONS AND RECOMMENDATIONS

General

Based on the results of our explorations, infiltration and laboratory testing, and engineering analyses, the proposed site improvements can be completed as proposed following the recommendations contained herein. Specific geotechnical recommendations are provided in the following sections.

The near surface soils at the site consist of fine-grained silt which is easily disturbed when wet. If construction is planned for wet conditions, measures must be taken to minimize disturbance. Although not encountered in our explorations, areas of fill will likely be encountered during demolition and construction. Fill composition and consistency will likely vary with location and depth and removal of soft and/or unsuitable fill material may be required. We should be contacted to evaluate subgrades and subsurface conditions encountered during construction. We also recommend that the project budget and schedule include contingencies for possible over-excavation and replacement of unsuitable fill material.

Excavations adjacent to existing structures, pavements, sidewalks, and utilities will require minimum setbacks and temporary slopes. Temporary shoring may be required if minimum setbacks cannot be accommodated. Recommendations for setbacks and temporary slopes are provided in the ***Excavation Considerations*** section of this report. We can provide recommendations for design of temporary shoring if required.

Earthwork

Preparation - Prior to earthwork construction, the site should be prepared by removing any existing structures, foundation elements, utilities, and loose, surficial fill from site improvement areas. Any excavation resulting from the aforementioned preparation should be brought back to grade with structural fill. Site preparation for earthwork will also require the removal of the existing pavement, base rock, and any uncontrolled fill from all pavement, building, and fill areas, and a 5-foot perimeter around those areas. Existing asphalt concrete and base rock thicknesses at the boring location were 2 and 5 inches, respectively. Existing portland cement concrete and base rock thicknesses at the CPT location were 8 and 12 inches, respectively.

Existing concrete floor slabs, retaining walls, and stem walls associated with demolished structures and the swimming pool may remain in place beneath new pavement areas only provided the floor slabs are cracked and broken on 2-foot centers to allow drainage, any associated utilities are grouted to eliminate

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voids, and any adjacent soft and/or unsuitable fill or materials are removed and replaced with structural fill. The top of foundation elements such as retaining or stem walls should be demolished and removed to at least 6 inches below finished subgrade elevations.

Existing foundation elements, pool elements, and utilities that may conflict with the construction and operation of new underground utilities and other site improvements should be removed. Recommendations for use of demolition materials as structural fill are provided in the ***Fill*** section of this report.

Root balls from trees may extend several feet and grubbing operations can cause considerable subgrade disturbance. All disturbed material should be removed to undisturbed subgrade and backfilled with structural fill. In general, roots greater than one-inch in diameter should be removed as well as areas of concentrated smaller roots.

Stabilization and Soft Areas - After stripping, we should be contacted to evaluate the exposed subgrade. This evaluation can be done by proof rolling in dry conditions or probing during wet conditions. Soft areas will require overexcavation and backfilling with well graded, angular crushed rock compacted as structural fill. A geosynthetic may also be required. We recommend that a geosynthetic used for stabilization consist of a woven geosynthetic with an AOS of #70 to # 100 sieve, and a minimum puncture resistance of 120 pounds (such as an AMOCO ProPex 2019 or equivalent).

Working Blankets and Haul Roads - Construction equipment should not operate directly on the subgrade when wet, as it is susceptible to disturbance and softening. Rock working blankets and haul roads placed over a geosynthetic fabric in a thickened advancing pad can be used to protect subgrades. We recommend that sound, angular, pit run or crushed basalt with no more than 6 percent passing a #200 sieve be used to construct haul roads and working blankets. Working blankets should be at least 12 inches thick, and haul roads at least 18 inches thick. Some repair of working blankets and haul roads should be expected.

The above rock thicknesses are the minimum recommended. Subgrade protection is the responsibility of the contractor and thicker sections may be required based on subgrade conditions and type and frequency of construction equipment.

If the construction schedule allows, existing pavement areas can be used as staging areas. The existing asphalt concrete pavement section should not be expected to protect subgrades from concentrated heavy construction traffic.

Fill – The on-site fine grained soils can be used for structural fill if properly moisture conditioned. This will not be feasible during wet conditions. In dry summer conditions the soils will require drying by scarification and frequent mixing in thin lifts. Space constraints at this site may make moisture conditioning of on-site material impractical. If moisture conditioned to within 3 percent of the optimum moisture content, the material should be compacted to at least 92 percent relative to ASTM D-1557 (modified proctor) using a tamping foot or sheeps-foot type compactor.

Fine grained fill should be placed in lifts no greater than 10 inches in loose thickness. In addition to meeting density specifications, fill will also need to pass a proof roll using a loaded dump truck, water

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truck, or similar size equipment. In wet conditions, fill should be imported granular material with less than 6 percent fines, such as clean crushed or pit run rock. Granular material should be compacted to 95 percent relative to ASTM D-1557 and must also pass a proof roll.

Demolished pavements, excavated base rock, and demolition materials that are free of organic and other deleterious materials and crushed to no greater than 12 inches in any dimension may be suitable for fill depending on moisture and fines contents. Such material should be well graded and placed and compacted in a manner to prevent voids. Recycled fill materials should be placed in lifts no greater than 12 inches in loose thickness. In addition to meeting density specifications, fill will also need to pass a proof roll using a loaded dump truck. Excavation and utility construction through fill constructed with demolition materials and other large aggregate materials will be difficult and likely result in increased backfill volumes.

Trenches – Utility trenches may encounter groundwater seepage and caving should be expected where seepage is present. Shoring of utility trenches will be required for depths greater than 4 feet and where groundwater seepage or sloughing occurs. We recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation. All excavations must be completed in accordance with applicable OSHA safety standards.

Pipe bedding should be installed in accordance with the pipe manufacturers' recommendations. If groundwater is present in the base of the utility trench excavation, we recommend over-excavating the trench by 12 to 18 inches and placing trench stabilization material in the base. Trench stabilization material should consist of well-graded, crushed rock or crushed gravel with a maximum particle size of 4 inches and be free of deleterious materials. The percent passing the U.S. Standard No. 200 Sieve shall be less than 5 percent by weight when tested in accordance with ASTM C 117.

Trench backfill above the pipe zone should consist of well graded, angular crushed rock or sand fill with no more than 7 percent passing a #200 sieve. Trench backfill should be compacted to 92 percent relative to ASTM D-1557, and construction of hard surfaces, such as sidewalks or pavement, should not occur within one week of backfilling.

Slopes - All slopes should be excavated with a smooth excavator bucket with the surface repaired if disturbed and upslope surface runoff should be rerouted so that it does not run down the face of the slopes. Equipment should not be allowed to induce vibration or infiltrate water above the slopes. Erosion control is critical to maintaining all slopes and should be in place immediately after construction of all slopes. All slopes and excavations must be constructed in accordance with applicable OSHA safety standards. Temporary cut slopes should be constructed as recommended in the ***Excavation Considerations*** section of this report.

Permanent cut slopes up to 5 feet high can be inclined at 2H:1V in the medium stiff or better silt. The presence of slow seepage may require drainage in the form of a blanket of angular pit run rock or a suitably revegetated reinforced erosion control blanket (such as North American Green SC150 or equivalent). Faster seepage may require improved erosion control measures, including additional drainage elements, and/or flatter slopes, and we should be consulted. Exposed soils which are soft or loose may also require such measures.

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Permanent fill slopes should be inclined no steeper than 2H:1V for slopes up to 5 feet high. The face of the fill slope must be overbuilt and cut back into compacted materials with a smooth excavator bucket. If steeper fill slopes are desired, we should be consulted to evaluate use of amended soils or grid reinforcement.

Excavation Considerations

Temporary Slopes and Shoring – Demolition of the 'annex' building and removal and backfill of the pool will require excavations adjacent to existing structures (118 SW Porter Street), pavements, utilities, and sidewalks. At the time of this report, we did not have elevation information associated with the bottom of the pool and associated utilities or foundation elevations for the house at SW 118 SW Porter Street. We anticipate that the pool and associated utilities could extend to depths greater than 12 feet below surrounding slab grades.

Excavations may be completed using open cut methods if adjacent existing structures, pavements, utilities, and sidewalks are at least 6 feet behind (horizontally) a plane extending upward at 1H:1V from the base of the excavation. Open excavation techniques may be used to depths up to 12 feet using temporary 1H:1V slopes, provided groundwater seepage is not present and with the understanding that some sloughing may occur. If excessive sloughing or caving occurs, the open excavations must be flattened and/or buttressed and we should be contacted immediately.

Drainage must be routed away from slope faces and no surcharges or construction equipment are allowed within 10 feet of the slope crest. As with permanent slopes, erosion control is critical to maintaining temporary slopes. All temporary slopes must be covered with plastic or other impervious sheeting weighted in place during wet conditions.

Excavations where the sides cannot be sloped back as described above will require temporary shoring. Shoring may also be required for temporary excavations adjacent to settlement-sensitive elements such as existing buildings, pavements, sidewalks, and utilities. We can provide recommendations for design of temporary shoring if necessary.

Shallow Foundations

General – We assume that all existing buildings are supported on conventional shallow foundations. Geotechnical recommendations for design and construction of new shallow foundations as well as for seismic evaluation of existing footings are provided in the following paragraph.

Footings should be embedded at least 18 inches below the lowest adjacent, exterior grade. Footings can be designed for an allowable bearing pressure of 2,500 psf for medium stiff or stiffer native silt or properly constructed structural fill. The preceding bearing pressure can be increased to 5,000 psf for temporary wind and seismic loads. Continuous footings should be no less than 18 inches wide, and pad footings should be no less than 24 inches wide. Resistance to lateral loads can be obtained by a passive equivalent fluid pressure of 300 pcf against suitable footings, ignoring the top 12 inches of embedment, and by a footing base friction coefficient of 0.35. Properly founded new footings are expected to settle less than a total of 1 inch, and less than ½-inch differentially (relative to new footings).

Seismic Design

General - In accordance with the 2003 International Building Code (IBC) as adapted by the State of Oregon Structural Specialty Code (SOSSC) and based on our explorations and analyses, the subject

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project should be evaluated using the parameters associated with Site Class D with the exception of building periods between 0.12 to 0.24 seconds (refer to the Seismic Hazard Study in the Appendix). Liquefaction is discussed below and additional site-specific seismic hazard information is included in the attached ***Seismic Hazard Study***. Seismic hazards are generally low.

Liquefaction - Liquefaction occurs in loose, saturated, granular soils. Strong shaking, such as that experienced during earthquakes, causes the densification and the subsequent settlement of these soils. Given the site topography and the soil type and consistency encountered in our explorations, the risk of structurally damaging ground deformations is low.

Retaining Walls

General - The following recommendations are based on the assumptions that (1) Wall backfill consists of level, well-drained, angular, granular material, (2) Walls are less than 10 feet in height, and (3) No surcharges such as stockpiled soil or equipment are placed within 10 feet of the wall.

Walls restrained against rotation should be designed using an equivalent fluid pressure of 55 pcf. Walls not restrained against rotation should be design using an equivalent fluid pressure of 33 pcf. These forces can be resisted by passive pressure at the toe of the wall using an equivalent fluid pressure of 300 pcf (this should exclude the top 12 inches of embedment) and friction along the base using a friction coefficient of 0.35. Retaining wall footings should be designed as recommended previously for shallow foundations.

Backfill - Retaining walls should be backfilled with clean, imported, granular soil with less than 6 percent fines, such as clean sand or rock. This material should also be compacted to a minimum of 92 percent relative to ASTM D-1557 (modified proctor). Within 3 feet of the wall, backfill should be compacted to not more than 90 percent relative to ASTM D-1557 using hand-operated equipment.

Retaining structures typically rotate and displace up to 1 percent of the wall height during development of active pressures behind the wall. We therefore recommend that construction of improvements adjacent to the top of walls be delayed until approximately two weeks after wall construction and backfill.

Drainage

General - If footing construction is to occur in wet conditions, a few inches of crushed rock should be placed at the base of footings to reduce subgrade disturbance and softening during construction. The surface around the building perimeter should be sloped to drain away from the building. As stated previously, our retaining wall recommendations are based on drained conditions. All retaining walls must include a drain constructed as described in the following section.

Wall Drains – Retaining wall drains should consist of a two-foot wide zone of drain rock encompassing a 4-inch diameter perforated pipe, all enclosed with a non-woven filter fabric. The drain rock should have no more than 2 percent passing a #200 sieve and should extend to within one foot of the ground surface. The geosynthetic should have an AOS of a #70 sieve, a minimum permittivity of 1.0 sec^{-1}, and a minimum puncture resistance of 80 pounds (such as an AMOCO ProPex 4551 or equivalent). One foot of low permeability soil (such as the on-site silt) should be placed over the fabric at the top of the drain to isolate the drain from surface runoff.

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Pavement

Asphalt Concrete – At the time of this report we did not have specific information regarding the type and frequency of expected traffic. We therefore developed asphalt concrete pavement thicknesses for areas exposed to passenger vehicles only and areas exposed to up to 5 trucks per day based on a 20-year design life and a truck factor of 0.6. We assumed that the average truck will consist of a panel-type delivery truck. Traffic volumes can be revised if specific data is available.

Our pavement analyses is based on AASHTO methods and subgrade of structural fill or undisturbed medium stiff or stiffer native silt having a resilient modulus of 6,000 psi and prepared as recommended herein. We have also assumed that roadway construction will be completed during an extended period of dry weather. The results of our analyses based on these parameters are provided in the table below.

Traffic	ESAL's	AC (inches)	CR (inches)
Passenger Vehicle Only	-	2.5	6
Up to 5 Trucks	24,272	3	8

The thicknesses listed in the above table are intended to the minimum acceptable for construction during an extended period of dry weather. Increased rock thicknesses will be required for construction during wet conditions. Crushed rock should conform to ODOT base rock standards and have less than 6 percent passing the #200 sieve. Asphalt concrete should be compacted to a minimum of 91 percent of a Rice Density.

Subgrade Preparation - The pavement subgrade should be prepared in accordance with the ***Earthwork*** *and* ***Site Preparation*** recommendations presented in this report. All pavement subgrades need to pass a proof roll prior to paving. Soft areas should be repaired by over-excavating the areas and installing a stabilization geosynthetic. Well graded, angular crushed rock backfill compacted as structural fill should be used to bring the aforementioned areas to-grade. For a stabilization geosynthetic we recommend a woven geosynthetic with an AOS of #70 to #100 sieve, and a minimum puncture resistance of 120 pounds (such as an AMOCO ProPex 2019 or equivalent).

Stormwater Infiltration Systems

General – Swales, soakage trenches, and/or dry wells are proposed for on-site disposal of storm water and will be located in the vicinity of the proposed new parking area at the northwest corner of the site. We anticipate that shallow systems such as swales or soakage trenches will extend to depths between 5 to 10 feet below surrounding grades and dry wells will extend up to 25 feet below surrounding grades. Infiltration system design, including drywell and/or soakage trench dimensions, will be determined by the project civil engineer based on storm water volumes, detention capacity, and infiltration rates. The following paragraphs provide geotechnical recommendations for design of the proposed systems.

Infiltration systems must be designed in accordance with the 2004 City of Portland Stormwater Management Manual and other applicable codes. Systems must be set back from embedded building walls in accordance with the 2004 City of Portland Stormwater Management Manual (see diagram on page 2-13 and 'Private Soakage Trench' section). Minimum vertical offsets from groundwater elevations may also be required by the City of Portland and the Department of Environmental Quality and should be incorporated into the system design.

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Testing Procedures – Falling head infiltration testing was completed at 6.0 and 25.0 feet bgs in boring B-1 by adding water to the inside of the auger flights and recording the water level drop with time following an initial saturation period of approximately 1 hour. Average head conditions during testing were approximately 4.6 feet during the test at 6.0 feet bgs and approximately 17 feet for the test completed at 25.0 feet bgs.

Infiltration Rates and System Design Recommendations - Based on the results of our testing and analyses, infiltration rates in the native silt unit are very low to low and rates in the native silty fine sand unit are low but may be suitable depending on the proposed system(s) and storm water volumes. We recommend using a design infiltration rate of 0.17 cubic inches per hour per square inch (in^3/hour per in^2) for infiltration in the silt unit which was encountered from approximately 1 to 24 feet below existing grades. We recommend using a design infiltration rate of 2.0 cubic inches per hour per square inch (in^3/hour per in^2) for infiltration in the silty fine sand unit which was encountered from approximately 24 feet to the depth of the boring (36.5 feet). The above rates include a reduction factor of 3 applied to test results to account for variable subsurface conditions and long term siltation of the infiltrating surface.

The above infiltration rates should be applied to the sides of the swales, trenches, and drywells in the respective units only. We recommend neglecting infiltration at the base of drywells, trenches, and swales to account for long-term siltation. System dimensions may require adjustment based on field observations during system construction.

We must be contacted during infiltration system construction to confirm that exposed conditions are consistent with those observed during our infiltration testing. Systems should be sized by the civil engineer according to design storm water volumes and rates. Minimum embedment in the sand unit should also be specified by the civil engineer.

Soakage trenches and swales should be backfilled with clean drain rock with no more than 2% passing a #200 sieve. The drain rock should be covered with a geosynthetic filter fabric and capped with a minimum of 12 inches of low permeability material such as the on-site near surface silt. Dry well annuluses should be at least one foot wide and filled with clean drain rock with no more than 2% passing a #200 sieve.

Confirmation Testing and Maintenance - Testing of infiltration systems is required to confirm the design infiltration rate as actual subsurface conditions and infiltration rates can vary widely. Flexibility for adaptation and expansion of infiltration systems should be incorporated into the design and construction, with contingencies included in the project budget and schedule. Infiltration systems must be maintained free of debris and silt in order to function properly.

LIMITATIONS AND OBSERVATION DURING CONSTRUCTION

We have prepared this report for use by the Cedarwood School and the design and construction teams for this project only. The information herein could be used for bidding or estimating purposes but should not be construed as a warranty of subsurface conditions. We have made observations only at the aforementioned locations and only to the stated depths. These observations do not reflect soil types, strata thicknesses, water levels or seepage that may exist between observations. We should be

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consulted to observe all foundation bearing surfaces, subgrades, installation of structural fill, subsurface drainage, and construction of infiltration systems. We should be consulted to review final design and specifications in order to see that our recommendations are suitably followed. If any changes are made to the anticipated locations, loads, configurations, grading, or construction timing, our recommendations may not be applicable, and we should be consulted. The preceding recommendations should be considered preliminary, as actual soil conditions may vary. In order for our recommendations to be final, we must be retained to observe actual subsurface conditions encountered. Our observations will allow us to interpret actual conditions and adapt our recommendations if needed.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty, expressed or implied, is given.

We appreciate the opportunity to work with you on this project and look forward to our continued involvement. Please call if you have any questions.

Sincerely,

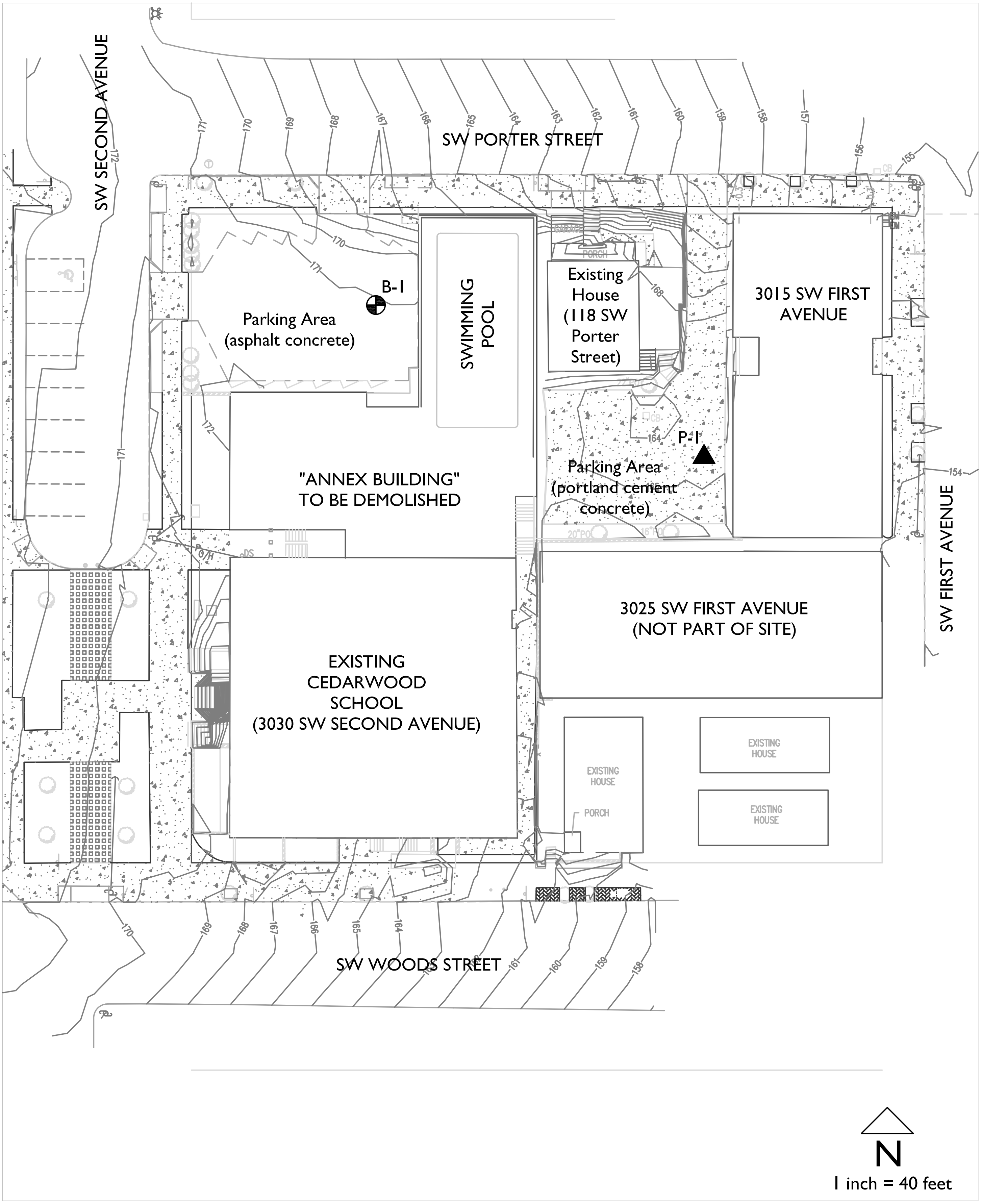
Christopher J. Palmer, MS, PE
Senior Project Engineer

REGISTERED PROFESSIONAL ENGINEER
72846PE
OREGON
July 15, 2003
CHRISTOPHER J. PALMER
EXPIRES: 12/31/08

Don Rondema, MS, PE, GE
Principal

Attachments – Site Plan, Guidelines for Classification of Soil, Boring Log, Cone Penetrometer Log, Fines Contents, Seismic Hazard Study, Ground Surface Spectral Response – Crustal, Ground Surface Spectral Response – CSZ, USGS fault map, USGS Partial Report for the Portland Hills Fault.

cc: Sam Galbreath, Sam Galbreath Associates - samg61@comcast.net
Karl Koroch, TM Rippey Consulting Engineers - kkoroch@tmrippey.com



BASE DRAWING PROVIDED BY ZTEC ENGINEERS INC.

Geotech Solutions Inc.

SITE PLAN

Cedarwood-07-01-gi

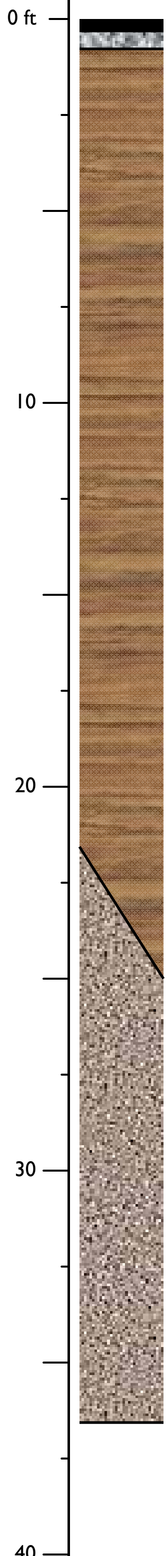
GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil	
Relative Density	**Standard Penetration Resistance (N-values) blows per foot**
very loose	0 - 4
loose	4 - 10
medium dense	10 - 30
dense	30 - 50
very dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils		
Consistency	**Standard Penetration Resistance (N-values) blows per foot**	**Torvane Undrained Shear Strength, tsf**
very soft	0 - 2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

Grain-Size Classification	
Description	**Size**
Boulders	12 - 36 in.
Cobbles	3 - 12 in.
Gravel	¼ - ¾ in. (fine)
	¾ - 3 in. (coarse)
Sand	No. 200 - No. 40 Sieve (fine)
	No. 40 - No. 10 sieve (medium)
	No. 10 - No. 4 sieve (coarse)
Silt/Clay	Pass No. 200 sieve

Modifier for Subclassification	
Adjective	**Percentage of Other Material In Total Sample**
Clean	0 - 2
Trace	2 - 10
Some	10 - 30
Sandy, Silty, Clayey, etc.	30 - 50



Soil and Rock Description

2 inches asphalt concrete (AC) at ground surface.
5 inches medium dense, gray-brown, GRAVEL FILL with some sand and silt; moist. (baserock).

Stiff, brown, SILT; moist.

grades to with trace fine sand at 7.5 feet.

grades to with some fine sand at 15.0 feet.

becomes stiff to very stiff and grades to sandy SILT at 20.0 feet.

Medium dense, brown, silty fine SAND; moist.

Boring completed at 36.5 feet.
Falling-head infiltration tests completed at 6.0 and 25.0 feet.

Samples and Data

8	w = 29%
10	w = 34%
8	w = 31%
11	w = 30%
9	w = 34%
16	w = 28%
23	w = 22%
27	w = 22%
24	w = 25%

w = moisture content
γ_d = dry unit weight
N_{60} = SPT blowcount
* = No recovery

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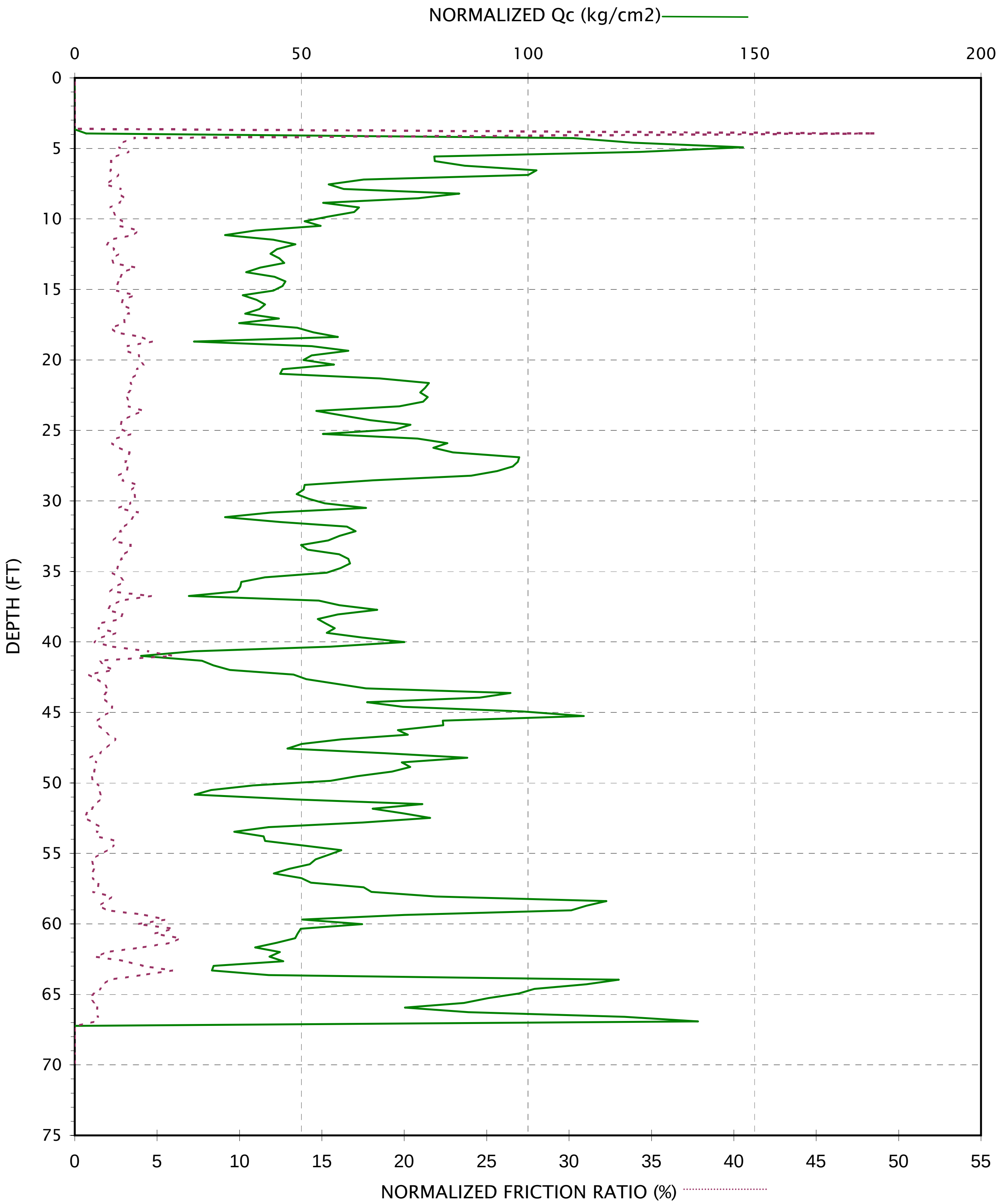
BORING B-1
cedarwood-07-01-gi



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CONE PENETROMETER LOG, P-1

Cedarwood-07-01-gi

Exploration	Depth, ft	Fines Content
B-1	15.0	77
B-1	20.0	60
B-1	25.0	46

SEISMIC HAZARD STUDY

General

We have evaluated seismic hazards in accordance with the degree of complexity of the proposed project. Our seismic hazard evaluation is based on our site explorations and reconnaissance, analyses, vicinity experience, and review of available literature. Site specific testing was also completed, including cone penetrometer shear wave velocity testing.

Based on our geotechnical evaluation, liquefaction, tsunami inundation and dynamic slope instability hazards are low at this site. Due to the proximity (less than ½ mile to the east) of the surface projection of the Portland Hills fault, risk of fault rupture is considered low to moderate. Amplification and overall ground motion hazards are moderate to high. Site specific response spectra for crustal earthquakes at this site exceed code class D levels between periods of 0.12 and 0.24 seconds. Structures outside these periods can be accommodated by conventional code seismic design. A summary of the basis for these opinions is included in the following paragraphs.

Seismic Sources and Design Earthquakes

Three primary earthquake sources have been identified. These include Cascadia Subduction Zone (CSZ) intraplate and interface earthquakes, and local crustal earthquakes. CSZ intraplate earthquakes are presumed possible within the subducted Juan de Fuca plate, with estimated magnitudes of 7.0 to 7.5. These earthquakes are analogous to the 2001 Nisqually earthquake near Olympia as well as other large earthquakes historically beneath southern Puget Sound. The expected depth of these presumed earthquakes of 40 to 60 km, and when coupled with low seismicity in western Oregon, present a low to moderate hazard. A CSZ interface earthquake presents a low to moderate hazard for the site area. Such an event has an expected magnitude of 8 to 9 at a distance of 90 to 120 km from the site and recurrence intervals from 100 to 1100 years. A magnitude $M_w = 8.5$ is expected to correspond to roughly an average 10% chance of being exceeded in 50 years, with $M_w = 9.0$ corresponding roughly to 2% in 50 years. Local crustal earthquakes may occur from northwest trending faults in the region, from the Mt. Angel fault and the Portland Hills fault zone. For the Portland Hills Fault Zone, design recurrence corresponds to earthquake magnitudes less than 6.5 at depths less than 20km. This represents the design level earthquake for base shear at the site.

Amplification and Site Specific Ground Motions

The site is mapped in an area of 'moderate hazard' with respect to ground amplification (DOGAMI GMS-79). USGS probabilistic ground motion mapping indicates peak horizontal ground accelerations at the site of 0.19g and 0.41g for a 10% and 2% chance of exceedence in 50 years, respectively. This amplification is typical of the mapped site geology (stiff shallow depth soil columns over dense gravels and soft rock).

We evaluated three types of earthquake sources for site response, including crustal and CSZ intraplate and interface models. Ground motion records were selected from our strong motion database that are representative of expected motions for these sources. Consideration was given to recording station geology, fault rupture type and proximity, spectral shapes, free field conditions, and other issues relative to processing of the recordings. A list of the sources used as representative models are in the table below. From these models we scaled the accelerograms to the expected base rock accelerations (as spectral shapes were already part of the selection process) corresponding to the USGS listed values for a 2% chance of being exceeded in 50 years.

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Model	Earthquake	Fault type	Magn.	Pred. Period (sec)	Amax (g)	Spectral Accel. 0.3s	Spectral Accel. 1.0s
Crustal	San Fernando, 1971	thrust	6.6	0.20	0.21	0.20	0.04
	Northridge, 1994	blind thrust	6.7	0.17	0.23	0.34	0.05
	Taft, 1952	strike	7.5	0.33	0.19	0.41	0.16
CSZ Intraplate	Petrolia, 1992	strike	7.1	1.40	0.42	0.90	0.69
CSZ Interface	Michoacan, 1985	thrust/subd	8.1	0.52	0.16	0.40	0.17
	Miyagi-oki, 1978	thrust/subd	7.4	0.53	0.21	0.51	0.10

The preceding models were scaled and used as input motions for analyses using the computer program PROSHAKE. The soil profile for used in the analyses was based on our explorations and velocity testing, as well as those published for seismic explorations within the same mapped unit (DOGAMI OFR O-95-7). This model included 20 feet of silt with some sand with an average Vs (shear wave velocity) of 600 ft/sec., underlain by 47 feet of silty sand with a Vs of 1,000 ft/sec., in turn underlain by Columbia River Basalt with a Vs of 2,500 ft/sec. Sensitivity analyses were completed on possible soil column period and damping variations by altering the thickness, modulus reduction and damping, and velocity.

The ground surface spectral response plots for the crustal earthquakes are attached with the code design spectra for site class D overlain (class D was determined from code procedures). The results indicate that amplification is high in the 0.12 to 0.24 second range. This is typical when short stiff soil columns are subjected to the higher energy short period crustal earthquake motions. The CSZ models also indicate high amplification in this period range, again responding to the soil column, but fall under the crustal spectra. Therefore, we recommend that the smoothed mean of the crustal models be used for design where it exceeds code level spectra.

Liquefaction, Fault Rupture and Tsunami Inundation

The soils at the site generally consist of a surficial layer of medium stiff, brown silt with trace to some sand, underlain by silty fine sand with ground water at a depth of approximately 41 feet. These soils are mapped as underlain by Columbia River Basalt, which likely presented refusal to the CPT probe at a depth of 67 feet. These unsaturated soils are generally not susceptible to liquefaction. The liquefaction hazard is therefore low. This assessment is consistent with GMS-79 (Mabey and Madin, DOGAMI, 1993)

The nearest mapped active quaternary fault (USGS – http://earthquakes.usgs.gov/qfaults/or/van.html) is the Portland Hills Fault mapped as approximately 0.5 miles east of the site (USGS fault map and description attached). This fault is off-site and thought to predate Pleistocene time with observed offsets indicating no conclusive evidence of activity in the last 15,000 years. Interface earthquakes from the CSZ are offshore, and intraplate CSZ earthquakes are deep within the subducted plate. Therefore, the hazard from potential on-site fault rupture is low to moderate.

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The site is located inland and outside tsunami inundation areas.

Earthquake Induced Slope Instability

There is a low potential for earthquake induced slope instability at the site (DOGAMI GMS-79).

Design and Limitations

Recommendations for design and limitations to this study are contained in the geotechnical engineering report.

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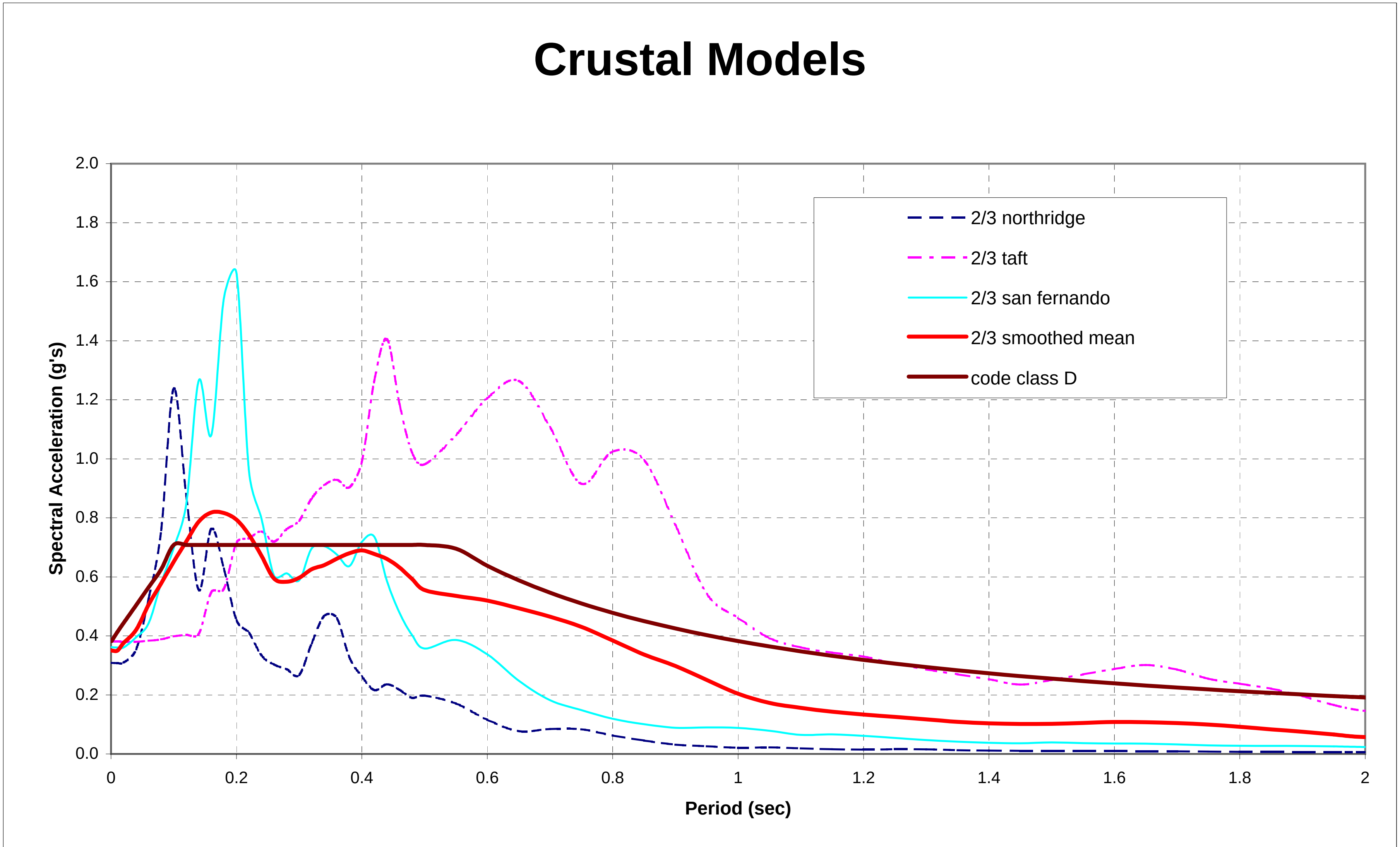

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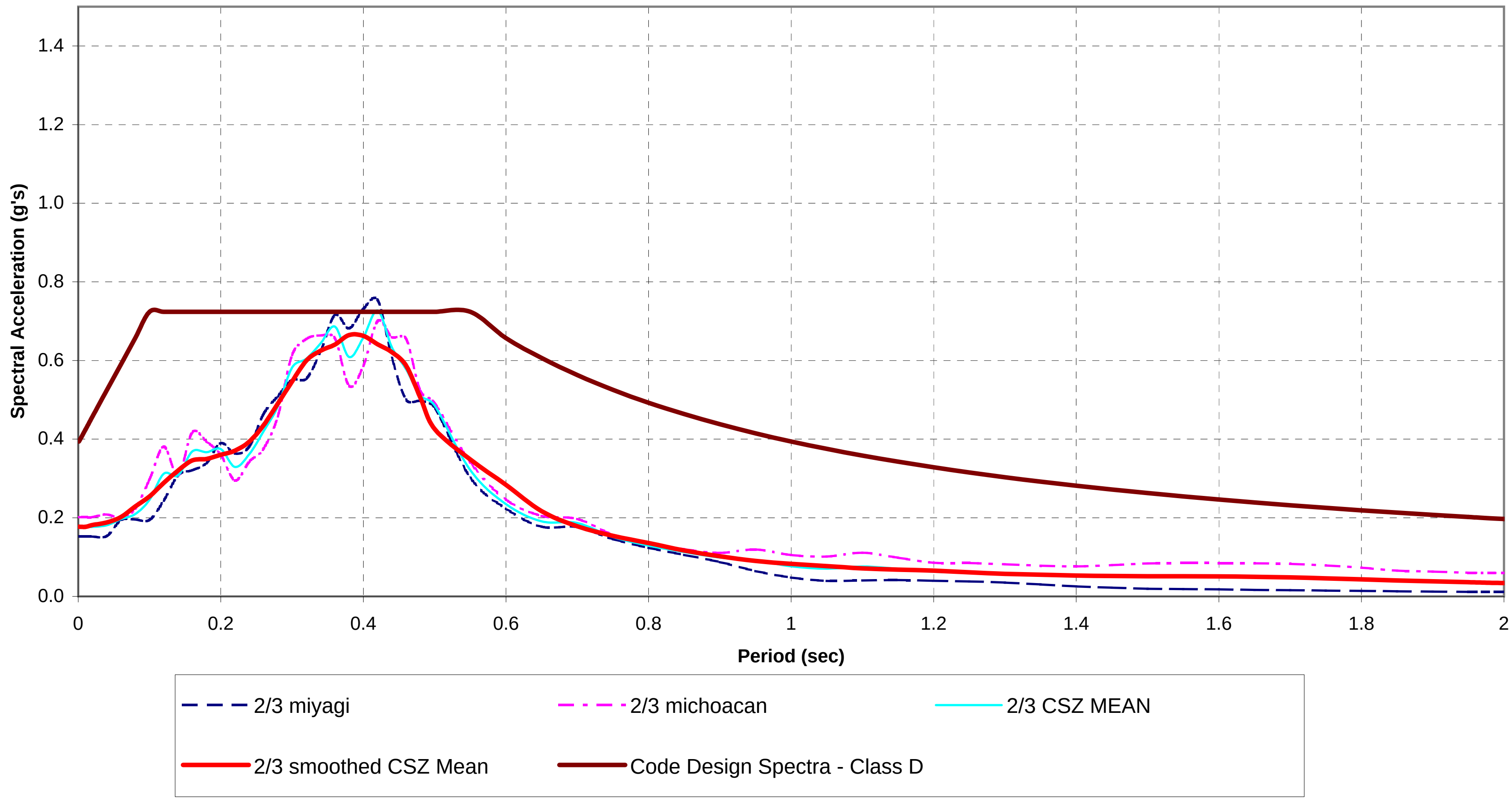
CSZ Interface Models



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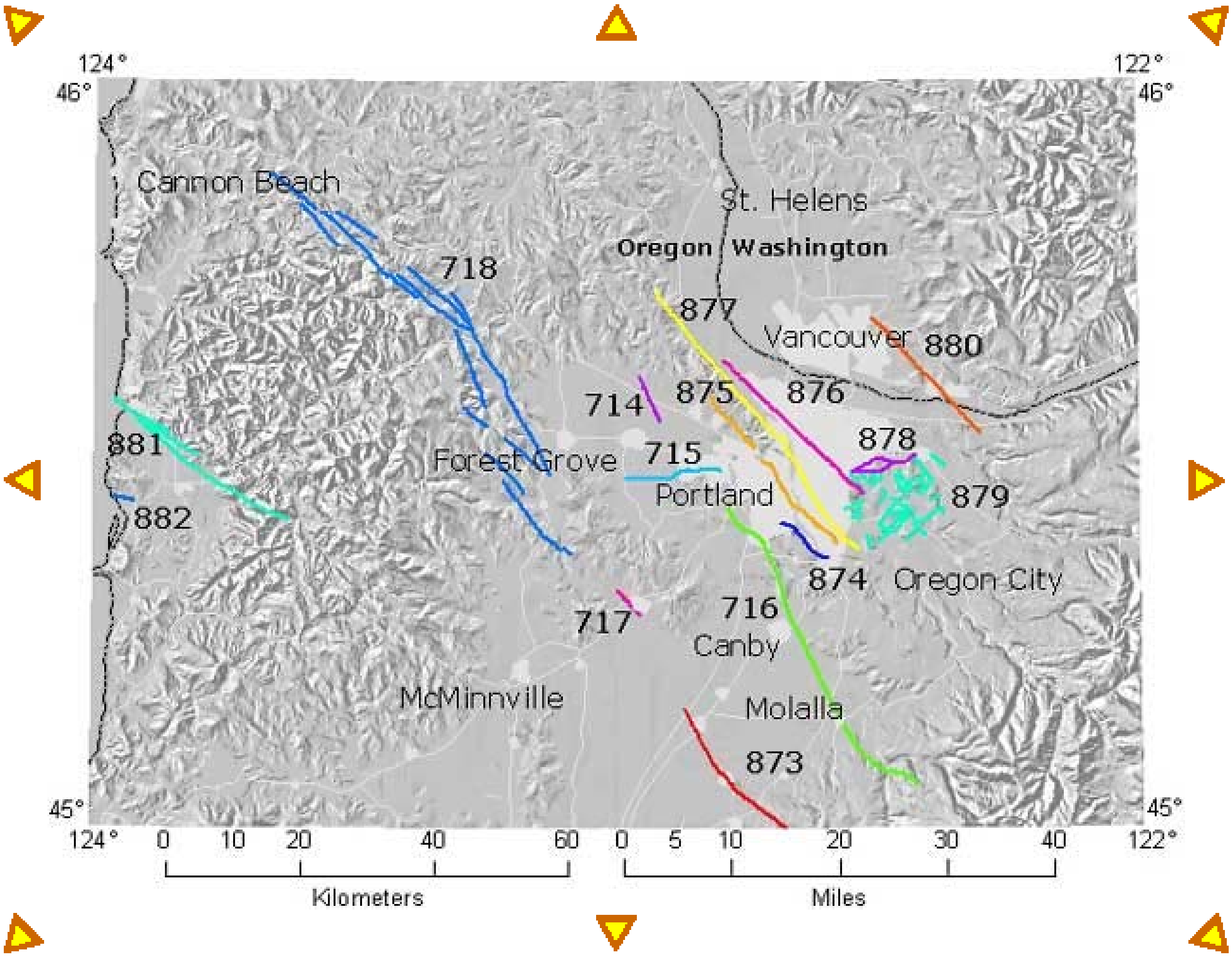
USGS
Quaternary Fault and Fold Database for the United States

Vancouver 1° x 2° Sheet

Home > US Map > Oregon



Number	Name
714	Helvetia fault
715	Beaverton fault zone
716	Canby-Molalla fault
717	Newberg fault
718	Gales Creek fault zone
873	Mount Angel fault
874	Bolton fault
875	Oatfield fault

876	East Bank fault
877	Portland Hills fault
878	Grand Butte fault
879	Damascus-Tickle Creek fault zone
880	Lacamas Lake fault
881	Tillamook Bay fault zone
882	Happy Camp fault

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Brief Report for Portland Hills fault (Class A) No. 877

Partial Report ||Complete Report

citation for this record: **Personius, S.F., compiler, 2002, Fault number 877, Portland Hills fault, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/regional/qfaults, accessed 12/06/2007 02:46 PM.**

Synopsis	The northwest-striking Portland Hills fault forms the prominent linear northeastern margin of the Tualatin Mountains (Portland Hills) and the southwestern margin of the Portland basin; this basin may be a right-lateral pull-apart basin in the forearc of the Cascadia subduction zone or a piggyback synclinal basin formed between antiformal uplifts of the Portland fold belt. The fault is part of the Portland Hills-Clackamas River structural zone, which controlled the deposition of Miocene Columbia River Basalt Group lavas in the region. The crest of the Portland Hills is defined by the northwest-striking Portland Hills anticline. Sense of displacement on the Portland Hills fault is poorly known and controversial. The fault was originally mapped as a down-to-the-northeast normal fault. The fault has also been mapped as part of a regional-scale zone of right-lateral oblique slip faults, and as a steep escarpment caused by asymmetrical folding above a southwest-dipping blind thrust. Reverse displacement with a right-lateral strike-slip component may be most consistent with the tectonic setting, mapped geologic relations, aeromagnetic data, and microseismicity in the area. No fault scarps on surficial Quaternary deposits have been described along the fault trace, but some geomorphic (steep, linear escarpment, triangular facets, oversteepened and knickpointed tributaries) and geophysical (aeromagnetic, seismic reflection, and ground-penetrating radar) evidence suggest Quaternary displacement.
County(s) and State(s)	MULTNOMAH COUNTY, OREGON WASHINGTON COUNTY, OREGON CLACKAMAS COUNTY, OREGON
AMS sheet(s)	Vancouver
Physiographic	

province(s)	PACIFIC BORDER
Length (km)	49 km.
Average strike	N37°W
Sense of movement	"RD?, T?"
Dip Direction	SW
Historic earthquake	
Most recent prehistoric deformation	Quaternary (<1.6 Ma)
Slip-rate category	Less than 0.2 mm/yr
Date and Compiler(s)	2002 Stephen F. Personius, U.S. Geological Survey