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Re: Geotechnical Engineering Feasibility Assessment Task 1 & 2  
Due Diligence Services – Multnomah County Courthouse  
Block 128, Portland, Oregon  
PBS Project No. 15194.869

## INTRODUCTION AND BACKGROUND

PBS Engineering and Environmental, Inc. (PBS) is pleased to provide this feasibility/due diligence letter report for geotechnical engineering services in support of site selection for the Multnomah County Courthouse in Portland, Oregon (Figure 1, Vicinity Map). Block 128 (site) is being considered as the site of a new courthouse building. The 1.1-acre site and is bounded by SW Columbia Street and SW Clay Street to the north and south, respectively, and SW 1st Avenue and SW 2nd Avenue to the east and west (refer, Figure 2, Site Plan). Based on available topographic data, ground surface elevations at the site range from 74 feet to 59 feet at the southwestern and northeastern corners, respectively (WGS84 EGM96 Geoid).

An existing structure on the southwestern portion of the site includes a one-story covered concrete entrance to underground parking and loading docks under the adjacent KOIN Center building. The approximate 13,600-square-foot structure extends along SW Clay Street, while outdoor parking stalls with associated drive lanes cover the northeastern half of the site. Planter areas line the perimeter of the site. The site has been used in this way since at least 1990 based on dated Google Earth<sup>TM</sup> imagery. Based on our conversations with Multnomah County (County) personnel and experience with similar projects, the development will include the following.

- A single 14- to 17-story, steel-frame, high-rise building with one level below grade
- A building foot print of approximately 28,000 square feet
- An assumed column load between 1,700 and 2,500 kips

The County has requested PBS identify potential geotechnical issues that could affect the proposed plan. The purpose of our geotechnical engineering services was to review existing geotechnical reports, and geologic hazards and seismic hazards maps of the area to provide opinions regarding the geotechnical feasibility of development. No subsurface explorations were included as part of this phase of engineering services. The project stakeholders, including Multnomah County, will utilize the information in completing their due diligence.

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## **GEOLOGIC SETTING**

The project site is located in the northern portion of the Willamette Valley physiographic province within the Puget-Willamette Lowland. In general, the Willamette Lowland is a broad alluvial basin bordered to the west by Tertiary marine sedimentary and volcanic rocks of the Coast Range, and to the east by Tertiary and Quaternary volcanic and volcanoclastic rocks of the Cascade Range. The northern boundary of the Willamette Lowland is generally recognized as the uplifted area north of the Lewis River in southwestern Washington. The southern boundary is generally defined as the convergence of the Coast and Cascade Ranges south of Eugene, Oregon.

Four separate basins are generally recognized in the Willamette Lowland: 1) the southern Willamette Valley south of, and including, the Salem and Waldo Hills; 2) the central Willamette Valley between Salem and the Waldo Hills and the Chehalem Mountains; 3) the Tualatin basin northeast of the Chehalem Mountains and southwest of the Tualatin Mountains; and 4) the Portland Basin (Gannett and Caldwell, 1998). Narrow ridges underlain by the Columbia River Basalt Group separate the basins. The Tualatin Mountains separate the Portland and Tualatin basins, the Chehalem Mountains separate the Tualatin basin and northern Willamette Valley, and the Salem and Waldo Hills separate the northern Willamette Valley and southern Willamette Valley (Yeats and others, 1996).

Basins within the Willamette Valley and the tributary valleys are filled with over 1,600 feet of unconsolidated alluvial deposits derived from the surrounding uplands and the Columbia River Basin (Gannett and Caldwell, 1998; O'Connor and others, 2001). These deposits rest unconformably on a basement complex comprised principally of the Columbia River Basalt Group. Fine-grained Miocene(?) and Pliocene fluvial-lacustrine deposits occur near the bottom of the basin-fill deposits. Coarse-grained fluvial deposits derived from the Cascade Range and the Missoula Floods generally comprise the upper 300 feet of the basin fill deposits.

The Missoula Floods had significant impacts on the geomorphology and depositional history of the Willamette Valley. Widespread inundation of the valley occurred during these large-volume glacial outburst floods that originated in eastern Montana approximately 12,000 to 15,000 years ago. Up to 250 feet of silt, sand, and gravel were deposited in the Portland Basin, and up to 130 feet of silt, known as the Willamette Silt, were deposited elsewhere in the valley (Woodward and others, 1998).

According to Beeson and others (1991), the geology at the site consists of Pleistocene fine-grained facies (Qff) consisting of coarse-grained sands to silt deposited by catastrophic floods. These alluvial deposits are underlain at depth by conglomerates of the Miocene to Pliocene Troutdale Formation (Tt) and siltstone, sandstone, and claystone of the Miocene to Pliocene Sandy River Mudstone Formation (Tsr). The basement bedrock consists of the upper Eocene to middle Oligocene Columbia River Basalt Group (CRBG). Based on local geotechnical borings near the site, we anticipate variable undocumented-anthropogenic (artificial) fill deposits will overlie the Qff unit.

## **SEISMIC SETTING**

Several fault zones are located within 50 miles of the project site and the Cascadia Subduction Zone (CSZ) is located approximately 80 miles from the site off the Oregon Coast. In addition, depending on the reference, the Portland Hills fault may trace through the southwestern corner of the property. There are several types of seismic sources in the Pacific Northwest, which are discussed as follows (Wong & Silva, 2006). Volcanic sources beneath the Cascade Range are not considered further in this study; since they rarely exceed about magnitude M 5.0, and thus, are not considered to pose a significant ground-shaking hazard to the project site.

Information on the historical record of Oregon earthquakes dates back to approximately 1841. Prior to 1900, approximately 30 earthquakes were documented. Several hundred earthquakes were documented in the state since 1900, especially since the 1980s when the University of Washington established a recording station in northwest Oregon. Catalogues of earthquake events are available from Berg and Baker (1963); Johnson, et al. (1994); and Wong, et al. (2000). Also provided, is a summary of Oregon earthquakes. Research completed over the last 10 years by DOGAMI and Oregon State University (Goldfinger, 2012) has uncovered evidence of historic earthquakes along the Oregon coast extending back on the order of 10,000 years. The research indicates over 40 events have occurred with as many as 19 of magnitude 9.0 or greater.

Oregon as a region has a relatively low to medium record of historical seismicity. Clusters of earthquakes are recorded in the Klamath Falls region (Magnitude [M] 6.0), northeast Oregon (M 5.0 Umatilla and M 6.5 Milton Freewater), Portland-Vancouver (1962; ML 5.2) and the Portland Northern Willamette Valley (ML 5.6 Mount Angel). Based on the current understanding of the potential associated with the CSZ and local faults, the relative regional seismicity would be considered high.

### **Crustal Earthquakes and Faults**

Due to their proximity, the crustal faults are possibly the most significant seismic sources for strong ground motion in the Portland metropolitan area. There are at least 55 faults or fault zones in northwest Oregon and southwest Washington (within 200 kilometers [km] of Portland). However, recorded seismicity generated by crustal sources in the site vicinity is relatively limited with only a few recorded earthquakes exceeding local magnitude ML 5 in the Portland region. Studies (Yelin & Patton, 1991) of small earthquakes in the region indicate most crustal earthquake activity is occurring at depths of 10 to 20 km.

The three most significant faults in the site vicinity include the Portland Hills Fault, East Bank Fault, and the Oatfield Fault. The nearest mapped fault is the northwest-trending Portland Hills Fault which, depending on the reference map or seismic sources, is located either tracing through the southwest corner of the property or one block to the west (Madin, 1990; Geomatrix, 1995). The Portland Hills Fault is not listed as active or potentially active (Geomatrix, 1995; Wong, 2000). The location of the Portland Hills Fault is interpreted and has not been observed directly. The northwest-trending East Bank Fault is located approximately 1 ½ miles east of the site (Madin, 1990; Geomatrix, 1995) and is not listed as active or potentially active (Geomatrix, 1995; Wong, 2000). The northwest-trending Oatfield Fault is located approximately 2 ½ miles west of the site (Madin, 1990; Geomatrix, 1995) and is not listed as active or potentially active (Geomatrix, 1995; Wong, 2000).

#### ***Portland Hills Fault***

The Portland Hills fault is mapped along the northeastern margin of the Tualatin Mountains (Portland Hills) and the southwestern margin of the Portland basin (refer, Figure 4 – Local Faults). The crest of the Portland Hills is defined by the northwest-striking Portland Hills anticline. Displacement on the Portland Hills fault is poorly known and controversial. No fault scarps on surficial Quaternary deposits have been described along the fault, but some geomorphic and geophysical evidence suggest Quaternary displacement (Personius, 2002).

#### ***East Bank Fault***

The East Bank fault lies in the Portland basin. The fault lies a few km east of, and generally runs parallel to, the Portland Hills fault which forms the southwestern margin of the basin. No fault scarps on

surficial Quaternary deposits have been described along the fault, and the fault is mapped by interpretation as buried by latest Pleistocene Missoula flood deposits (Personius, 2002).

#### *Oatfield Fault*

The Oatfield fault forms northeast-facing escarpments in volcanic rocks of the Miocene Columbia River Basalt Group in the Tualatin Mountains and northern Willamette Valley. No fault scarps on surficial deposits have been described, but exposures in a light-rail tunnel showing offset of Boring Lava across the fault indicate Quaternary displacement (Personius, 2002).

### **Cascadia Subduction Zone**

#### *Interface Earthquakes*

The CSZ megathrust represents the boundary between the subducting Juan de Fuca tectonic plate and the overriding North American tectonic plate. Recurrence intervals for subduction zone earthquakes are based on studies of the geologic record. Based on these studies, recurrence interval estimates have been generated ranging from about 300 to 600 years. Geologic evidence suggests the most recent earthquake occurred in January 1700. The 1700 earthquake probably ruptured much of the approximate 620 miles (1,000 km) length of the CSZ, and was estimated at moment magnitudes  $M_w$  9.0. The horizontal distance from the edge of the CSZ megathrust, located offshore from Portland, is approximately 90 miles (150 km) with an uncertainty of  $\pm 30$  miles (50 km) (Wong & Silva, 2000). The current U.S. Geological Survey (USGS [2008]) risk-based maximum credible earthquake for CSZ megathrust is  $M_w$  9.2.

#### *Intraslab Earthquakes*

A number of researchers have noted the complete absence of intraslab seismicity in Western Oregon (Ludwin et al., 1991; Rogers et al., 1996). With the possible exception of 1873 Richter Magnitude 6.75 Crescent City Earthquake, no moderate to large intraslab earthquakes have occurred in the CSZ from south of Puget Sound to Cape Mendocino. These earthquakes are postulated to have a deep focus of 40 to 70 km in the subducted Juan de Fuca Plate (Wong, 2005), and theoretical magnitudes of up to  $M$  7.8. These earthquakes are expected to have epicenters for 50 to 100 km from the site.

### **ANTICIPATED SUBSURFACE CONDITIONS**

Previous explorations were completed at the site and observed by Geocon, Inc. (Geocon), and included two borings to depths of 44.0 to 45.5 feet below the ground surface (bgs). Interpreting the borings indicate the area is underlain by variable subsurface conditions. Based on materials reviewed, deposits of undocumented fill are expected to be present extending to at least 4 feet from the ground surface. This fill is likely dense with low compressibility, but may contain debris. Fill was underlain by alluvial sediments generally consisting of stiff to very stiff, sandy silt with lenses of medium dense to very dense, poorly-graded sand. In general, the alluvial sediments are underlain by the gravel. The gravel has very dense gravels and cobbles with interbedded sand and silt layers. The following Table 1 summarizes observations made by Geocon during drilling, including blow counts per foot (N-values [standard penetration resistance]), dry densities, and moisture content ranges.



**Table1: Summary of Subsurface Conditions within Block 128<sup>a</sup>**

Soil Type	Depth Range (ft bgs) <sup>b</sup>	N Values (blows/ft)	Consistency / Relative Density	Moisture Content Range (%)	Dry Density (pcf)
Fill (Gravel with brick and debris)	4 to 9	Between 24 and 50 for 1 inch	Medium dense to very dense	18 to 23	N/A
SILT and SAND	4 to 45	Between 11 and 50	Stiff to hard; medium to very dense	19 to 38	87 to 90
GRAVEL with silt	44 +	50 for 4 ½ inches	Very dense	28	N/A

a. Information summarized from Geocon, Inc. 2001 Geotechnical Investigation report.

b. ft bgs = feet below the ground surface

## GROUNDWATER

Groundwater information in the site vicinity was obtained from the Oregon Department of Water Resources (OWRD) local well logs, previous geotechnical explorations, and USGS groundwater contours. Groundwater was not documented during Geocon's explorations. However, the 2001 USGS regional groundwater contours show groundwater may be present at a depth of about 50 feet bgs (elevation 15 feet COP Datum). Therefore, we anticipate this level could fluctuate between 40 and 50 feet bgs during the year.

In general, groundwater is likely hydraulically connected to the Willamette River and has a down-gradient dip toward the river that is about 700 feet to the east. Perched groundwater may be encountered throughout the project site due to the variations in fill and alluvial deposits.

## GEOLOGIC AND SEISMIC HAZARDS

Geologic and seismic hazards are defined as those conditions associated with the geologic and seismic environment that could influence existing and/or proposed improvements. In general, the geologic and seismic hazards most commonly associated with the physical and chemical characteristics of near-surface soil, rock, and groundwater include the following.

- Slope stability
- **Adverse soils**
- Land subsidence
- Subsurface voids
- **Hydrology and drainage**
- Hazardous Minerals
- Erosion and sedimentation
- **Hydrogeology and groundwater**
- Volcanic hazards
- Permafrost and freeze-thaw
- **Seismic hazards (liquefaction, lateral spreading, earthquake-induced landslides, ground shaking, fault ground rupture, tsunamis and seiches)**

Those shown in bold above are geologic and seismic hazards that could affect the site development and should be considered in the planning process. Specific hazards are presented in the following Table 2. The "Level of Concern" is a qualitative assessment based on our engineering and geological judgment. Where noted, the terminology is taken from a specific source (i.e., Geohazards Viewer [HazVu], Federal Emergency Management [FEMA]).

**Table 2: Summary of Potential Geologic and Seismic Hazards at Block 128**

Geologic and Seismic Hazard	Examples	Level of Concern
Adverse Soils	Artificial Fill	High
	Expansive Soil	Low
	Compressible Soil	High
	Organic-Rich Soil	Low
	Sensitive Clay	N/A
Hydrology and Drainage	Flooding <sup>a</sup>	Low
	Standing Water	Low
Hydrogeology and Groundwater	Shallow Groundwater <sup>b</sup>	Moderate
	Seepage and Piping	Low
	Permeability and Percolation	Low
Seismic Hazards	Ground Amplification <sup>c,d</sup>	Moderate
	Local Fault Rupture <sup>d,e</sup>	Possible
	Liquefaction and Lateral Spread <sup>c,d</sup>	Low
	Seismically-Induced Settlement	Low
	Ground Lurching or Cracking	Low
	Seismically-Induced Slope Instability	N/A
	Tsunami	N/A
	Seiches	Low

<sup>a</sup> Information from the Portland Maps, <http://www.portlandmaps.com>. Site is not within the FEMA 100- and 500-year flood zones or 1996 inundation zone.

<sup>b</sup> Groundwater is assumed to be hydraulically connected to the Willamette River elevation. Anticipated to be approximately between 40 and 50 feet bgs.

<sup>c</sup> Information from the Department of Geology and Mineral Industries (DOGAMI), GMS-79, plates 1 and 2.

<sup>d</sup> Information from the Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: Statewide Geohazards Viewer, <http://www.oregongeology.org/hazvu/>.

<sup>e</sup> An interpreted trace of the Portland Hills fault crosses the southwest corner of the site based on the USGS Faults and Folds Database. However, the Oregon HazVu program shows the fault to be crossing one block west of the site.

The primary geologic hazard to consider in the site's planning and development is the presence of the undocumented, variable fill materials, and potential groundwater within the excavation depths. These materials may consist of backfill in the form of dense gravels with brick fragments and trash debris. Subsurface voids may be encountered due to fill placement and its material types. Shallow groundwater may require dewatering during construction and future management.

The primary seismic hazards are liquefaction/lateral spreading; possible local fault ground rupture and ground shaking (Figure 4, Local Fault Map and Figure 6, Ground Motion Amplification). Subsurface conditions indicate liquefaction and lateral spreading may be potential hazards at the site, regardless of current mapping shown in GMS-79 (Plate 1), and through the online HazVu program. Liquefaction is a phenomenon in which shaking of a saturated soil causes its material properties to change so that it behaves as a liquid. Soils that liquefy tend to be young, loose, granular soils that are saturated with water (National Research Council, 1985). Unsaturated soils will not liquefy, but they may settle during a seismic event. Typical displacements could be on the order of several inches. Thus, if the soil at a site liquefies, the damage resulting from an earthquake can be dramatically increased over what shaking alone might have caused. The liquefaction hazard analysis is based on the age and grain size of the geologic unit, the thickness of the unit, and the relative density and the propagating shear-wave velocity. In addition, if significant liquefaction takes place during an earthquake, lateral spreading that may occur toward the banks of the Willamette River located about 700 feet east, could affect the site.

The Portland Hills Fault is possibly located tracing through the southwest corner of the property and under the KOIN tower. The fault is concealed in the Portland downtown area and different references show it in different locations in the area. Evidence of recent activity is also inconclusive, and consequently, is currently considered to be inactive. Seismic activity on crustal faults could appear at some distance from mapped locations and may not even have a surface expression.

The 2014 OSSC methodology defines six soil categories that are based on average shear-wave velocity in the upper 100 feet (30 meters [m]) of the soil column. The shear-wave velocity is the speed with which a particular type of ground vibration travels through a material, and can be measured directly by several techniques. The six soil categories are Hard Rock (A), Rock (B), Very Dense Soil and Soft Rock (C), Stiff Soil (D), Soft Soil (E), and Special Soils (F). Based on these criteria, which is consistent with the amplification factor on Figure 6, the site probably would be Site Class D.

## **CONSIDERATIONS**

Several geotechnical-related considerations should be assessed in detail prior to the site development and building design. This feasibility study provides an initial assessment of the seismic, foundation-type, and construction considerations based upon the highly limited information and assumptions described.

### **Seismic Considerations**

Assuming new buildings are designed and constructed in the near future; these would be completed in accordance with the requirements of the 2014 Oregon Structural Specialty Code (OSSC), which is the 2012 International Building Code (IBC) with Oregon-specific amendments. The 2014 OSSC requires buildings be designed to consider ground motions from the risk-targeted, maximum considered earthquake ( $MCE_R$ ), defined by the OSSC as an earthquake with a 2,500-year return interval (probability of exceedance of 2 percent in 50 years).

Based on review of the OSSC, the spectral response accelerations at the site,  $S_s$  and  $S_1$ , corresponding to periods of 0.2 and 1.0 second, are approximately 1.0 and 0.4 g, respectively. The OSSC recommends that the effects of site conditions on building response be determined using site factors,  $F_a$ , and  $F_v$ , based on site classification, D as described above. However, if site-specific testing and analyses indicate liquefaction is probable at the site, then it would be classified as Site Class F. Site Class F requires that site response analyses be completed to develop site-specific coefficients for use in design by the structural engineer. The site class used in design should be based on site-specific exploration and testing using current, code-based standards.

### **Soil Improvement**

Settlement from liquefaction can sometimes be addressed by supporting the structure (and slab) on piles that derive their capacity from deeper, non-liquefiable soils. However, the forces associated with lateral spreading and available lateral resistance in liquefied soil could likely preclude the use of piles for mitigation of lateral spreading at this site. Another option is soil improvement. Densifying or amending site soils below the foundation elevation in the saturated liquefaction susceptible zone would reduce or eliminate the risk of liquefaction settlement and lateral spreading.

A relatively common method to mitigate liquefaction in the area for the conditions encountered at the site would be vibro-replacement (e.g., stone columns). Vibro-replacement incorporates a large, vibratory probe that is advanced to the target depth with the void filled with compacted, crushed rock as the probe is extracted, creating a series of stone columns. Advancing the probe as it vibrates can

densify loose, cohesionless, liquefaction susceptible soils, while the replacement with crushed rock acts to improve, soft, compressible, fine-grained soils that cannot be densified due to their poor drainage characteristics, by reinforcing them with better materials.

Depending on the application, stone columns can be 2 to 4 feet in diameter and installed in a grid at 4 to 8 feet on center. The extent beyond the intended area of improvement should be approximately half the depth of improvement.

### **Foundation Considerations**

A previous, general assessment of high-rise building foundations and excavations in the downtown Portland area was performed by Squier Associates (1997). The report summarizes the depths and characteristics of the geologic deposits, the types of foundations for the buildings, and the related parameters used in the design. Based on our engineering judgment and supported by the information in this report, shallow foundations (spread footing or mat) are likely not feasible at the Block 128 site without excavation down to the gravel encountered at depths of about 45 feet bgs, and deep foundations will be required for the proposed building type with estimated column loads of 1,700 to 2,500 kips. In addition, ground improvement (e.g., stone columns) may also need to be considered to mitigate the risk of liquefaction and lateral spreading during an earthquake.

Several deep foundation alternatives can be considered for building support; however, based on our experience and what has been constructed in the vicinity of the site, driven H-piles or pipe piles, drilled shafts, or drilled augercast piles are likely choices. Advantages and disadvantages of the three alternatives are shown in Table 3 as follows.

**Table 3: Deep Foundation Alternatives for Block 128 Site**

Deep Foundation Type	Advantages	Disadvantages
Driven Pile	<ul style="list-style-type: none"> <li>Commonly available materials</li> <li>Easily installed using readily available equipment</li> <li>Displaces, vibrates, remolds and consolidates soil around pile</li> <li>Structural integrity is better known when compared with other deep foundation types</li> <li>Pile capacity can be inferred from driving resistance</li> <li>Relatively unaffected by groundwater</li> </ul>	<ul style="list-style-type: none"> <li>Capacity is limited when compared with drilled shaft foundations</li> <li>Installation can be loud and create vibrations</li> <li>Mobilization can be expensive and the general weight of the equipment can be relatively heavy</li> <li>Piles may need to be stored onsite constraining the work area</li> <li>Flexure of the piles is higher than other deep foundations and durability can be a concern</li> <li>Installation equipment may need a large clearance area.</li> </ul>
Drilled Shafts	<ul style="list-style-type: none"> <li>Can be relatively larger in diameter than driven piles possibly reducing the cost of construction (by reducing the number of piles)</li> <li>Can be constructed in dense soils where driven piles aren't practical.</li> <li>Pile strength can be increased by increasing the diameter and reinforcing</li> <li>Reduced disturbance and vibration compared to driven piles</li> <li>Soil conditions can be observed during construction.</li> <li>Equipment is relatively light when compared with pile driving equipment</li> <li>Relatively better for resisting lateral loads due to larger possible diameter</li> </ul>	<ul style="list-style-type: none"> <li>Can be relatively expensive compared to driven piles</li> <li>Shaft cleaning is necessary and can be difficult</li> <li>Drilling can be difficult in cobbles boulders</li> <li>For looser soils, stabilization may be needed which may include casing or slurry</li> <li>Where groundwater is present, casing may be required</li> <li>Concrete installation can be difficult which may result in mud inclusions and air voids within the shaft end bearing capacity is generally negligible.</li> </ul>
Drilled Augercast Pile	<ul style="list-style-type: none"> <li>Provides continuous support of drilled hole sidewalls eliminating the need for shoring in soft, loose, or saturated soils</li> <li>Pile strength can be increased by increasing the diameter and reinforcing</li> <li>Reduced disturbance and vibration compared to driven piles</li> </ul>	<ul style="list-style-type: none"> <li>Cannot achieve significant penetration into gravels and cobbles</li> <li>Produces spoils that must be transported off-site</li> <li>Disturbance to the ground surface from spoils</li> <li>Concrete installation can be difficult which may result in mud inclusions and air voids within the pile</li> <li>End bearing capacity is generally negligible.</li> </ul>

### Construction Considerations

Detailed construction considerations and related designs would be provided during the geotechnical engineering phase of the project. These would include, for example, structural fill material-types and compaction, detailed excavation parameters, wet weather construction, shoring types and performance requirements, and dewatering options. The considerations provided as follows are for general purposes.

### *Excavation*

In general, all vegetation, topsoil, and existing structural elements (slabs, footings, etc.) should be removed from new building and pavement areas. Construction of the proposed new building probably will include basement levels, which would require below-grade excavation and associated export of soil from the site. Subsequently, reuse of on-site soils for fill was not considered.

Due to the anticipated presence of silt and clay in the near-surface materials, construction equipment may have difficulty operating when soils are above the optimum moisture; that is, above the liquid limit. Construction of granular haul roads placed over geotextile stabilization fabric may help reduce disturbance of site soils. The thickness of the granular material for haul roads and staging areas will depend on the amount and type of construction traffic working on site.

### *Shoring (Temporary)*

A wide variety of shoring systems are available for temporary shoring. Among the most commonly used shoring walls in the area are soldier piles with tiebacks, soil nails, sheet piles with braces, or struts. Sheet pile walls may not be feasible for this excavation due to the limits on driving or vibrating piles, as well as the gravel content of the subsurface soils. In our current opinion, a soldier pile wall, combined with braces and struts or tiebacks, which would include driven piles or piles installed into drilled holes into the underlying dense gravel, may be used for shoring. These shoring systems are discussed in general terms in the following paragraphs.

Soldier pile walls (with tiebacks) are generally constructed using steel H-piles placed into augered holes drilled or driven at intervals along the wall alignment. The holes are then backfilled with weak concrete. The soil in front of the wall is excavated from the top down. As the soil is exposed, the weak concrete is chipped away and lagging is fitted between the H-piles. Lagging is inserted behind the flanges or attached to the face of the flanges. The lagging usually consists of wood planks or steel plates. The soil is temporarily supported by arching between adjacent steel H-piles until the lagging is installed. However, soft/loose soils (and debris fill) common to the site vicinity, typically slough into the excavation until the lagging is installed and soil is in contact with the lagging.

Tieback soil anchors are installed to provide additional lateral resistance. These can be installed at any location along the wall, but are most commonly placed in rows after excavation reaches certain design levels. The location, number, and capacities are designed to provide the lateral load capacity needed to resist the applied earth pressures with a suitable factor of safety.

Soil nail walls have been used successfully in areas above the water table with soil of moderate to good competency. Soil nails are relatively short anchors that are placed in rows and spaced about 5 feet vertically and horizontally. With each level of nail placement, the supported soil is covered with shotcrete. Then the next level of nails is installed. Care must be exercised to match the strain relaxation of the retained soil with the strain required in the soil nail to support the lateral soil loading. Soil nails are less compatible with collapsing soil, or where perched water may be encountered. Also, soil nails are less suitable where settlement of the retained soil should be limited because of possible adverse impact on adjacent structures and utilities.

Braces and struts are also a temporary shoring support alternative, and typically are used in conjunction with different shoring wall types at locations where external supports, such as tieback anchors, "dead-man" anchors, and soil nails, cannot be used. Internal supports may include "cross-lot" braces and diagonal struts or "rakers". Braces and struts span across the excavation. These are

probably the least practical for use at the site due to the relatively wide spans to be shored, and their interference with internal construction activity.

### Vibration

Vibration monitoring and controls may be necessary during construction. Although blasting is unlikely at the site, the City of Portland Technical Manual, Section 3 – Public Safety that include sections 3.7.3.2 – *Pre-Blast Survey Documentation*, 3.7.3.3 – *Pre-Blast Survey Conditions Report*, and 3.7.9 – *Vibration Limits and Ground Vibration Monitoring* provides building and utility survey guidelines and vibration monitoring information that could be applicable during the installation of shoring, deep foundations, and ground improvement. Similar pre-construction surveys and vibration monitoring during construction is highly recommended even though blasting is unlikely. Construction ground settlement and vibrations must be limited to avoid potential disturbance or damage to adjacent buildings. Additional resources include the Oregon Department of Transportation (ODOT) SP335, Section 00335 Blasting Methods and Protection of Excavation Slopes (January 2014), and Federal Transit Administration, *Transit Noise and Vibration Impact Assessment*, Section 8.0, Vibration Impact Criteria (May 2006). A combination of these resources and limit of vibrations is provided in the following Table 4.

**Table 4: Specifications for Maximum PPV on Specific Structures and Building Usage**

Structure	Maximum Peak Particle Velocity (PPV) at the Structure (Inch/Second)
Standard Construction (timber frame, brick, concrete buildings) <sup>a</sup>	2
Reinforced Concrete Structures <sup>a</sup>	4
Steel Structures <sup>a</sup>	4
Buried Utilities <sup>a</sup>	2
Wells and Aquifers <sup>a</sup>	2
Green Concrete (Less than 7 days) <sup>a</sup>	1
Institutional land uses with primarily daytime use <sup>b</sup>	3.11 <sup>c</sup>
TV Studios <sup>b</sup>	1 <sup>c</sup>

<sup>a</sup> ODOT SP335, Section 00335 Blasting Methods and Protection of Excavation Slopes ([http://www.oregon.gov/ODOT/HWY/SPECS/Pages/2008\\_special\\_provisions.aspx#Part\\_00300](http://www.oregon.gov/ODOT/HWY/SPECS/Pages/2008_special_provisions.aspx#Part_00300)), accessed 12/4/2013, effective date January 9, 2014

<sup>b</sup> Federal Transit Administration, *Transit Noise and Vibration Impact Assessment*, Section 8.0 Vibration Impact Criteria (May 2006)

<sup>c</sup> Converted from VdB

### Dewatering

The presence of groundwater in the zone of construction has a variety of potential impacts ranging from direct effects on construction, to indirect effects away from the construction zone. The method of controlling or handling groundwater depends upon a number of factors. These factors include: depth to groundwater; the depth of excavation; expected quantity; water quality (especially the presence of groundwater contamination); recharge source(s); soil type; and the hydrologic and engineering properties of the native material above and below the excavation base, and presence or absence of a deeper aquifer.

Potential hydrologic effects of temporary drawdown and changes in groundwater flow paths also may reach out for a great distance. Consequently, the potential off-site impact, due to construction control of the groundwater, must also be considered. Potential adverse impacts include such effects as induced settlement of surrounding facilities due to drawdown and the handling and disposal of collected water. In addition, the issue of hydrologic reach or extent of the drawdown effects must be considered in order to evaluate potential changes in groundwater flow patterns. This issue might affect migration of

groundwater contamination plumes, and may result in the spreading of contamination into areas that are not currently contaminated. Furthermore, it could adversely affect the efforts of other third parties in their efforts of controlling the spread or in mitigating groundwater contamination plumes, thereby imposing a potential liability burden on the property developer.

## **PRELIMINARY CONCLUSIONS**

Based on our research and anticipated subsurface conditions, the Block 128 site is suitable for the proposed development but will require specific geotechnical considerations during design and construction. The geotechnical-related considerations include:

- The site uses of the property throughout its history have resulted in the presence of undocumented fill with variable content and consistency. Previous development of the site is discussed in more detail in the Phase I report prepared by PBS. This should be considered in the foundation selection and required excavation such that foundations and slabs are not supported on this material. Our current understanding is that there is no evidence that these materials or associated obstructions would restrict or impeded development of the property. Based on the limited information described, previous site usage does not appear to restrict the use of this property for the intended purpose.
- Based on GMS-79 Plate 2 and DOGAMI's HazVu program, the site should expect ground shaking from crustal and CSZ earthquakes. Amplifications could vary due to variations in subsurface soil conditions in conjunction with the building height that will require a site response spectral analysis. Based on Mabey, et. al.'s (1993) analysis, ground motion amplification could be between 1.0 and 1.4.
- Liquefaction and lateral spreading, though not mapped at the site by GMS-79 Plate 1 and HazVu, are potential hazards of low to moderate concern based on the subsurface conditions.
- The Portland Hills Fault might trace through the southwest corner of the property. However, since there is no evidence or knowledge that this fault is potentially active, impact to the site is highly unlikely.
- Groundwater levels will likely fluctuate with changes in the Willamette River stages. In general, groundwater is anticipated to be about 10 to 20 feet bgs according to regional groundwater mapping by the USGS and other resources.
- The Site Class is anticipated to be C, D, or F, based on the proposed building configuration and existing subsurface information. Further liquefaction analysis is necessary to refine the site class, which would impact the site response spectra used for structural design.
- Deep excavations and foundations will require shoring and dewatering considerations; therefore, vibration impacts assessment and monitoring during construction would be highly advisable.

## **Data Sources**

Several data sources were used to provide the information included in this letter report. Available engineering reports in our files and from the City of Portland and other documents including readily-available well logs and online resources from the Department of Geology and Mineral Industries (DOGAMI) were reviewed. The primary documents used in this feasibility study are as follows.



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## **LIMITATIONS**

Our evaluations and preliminary conclusions are based upon review of the limited referenced documents. No subsurface explorations were completed during this work to verify the type and depth of fill, soil, bedrock, or depth of groundwater at the site. We should be contacted to review the proposed site development plan to evaluate their possible affect on the site property. A geotechnical engineering report that includes site-specific explorations will be required prior to design.

We understand, based on our conversations with you, that the information provided in this report is only for your information, for use in feasibility planning associated with the site and you will not hold PBS liable in any regard for decisions related to due diligence, purchase, or design and construction estimating. Site-specific exploration and engineering is required in order to refine the very general discussion of subsurface conditions (based on previous work) provided in this report.

## CLOSING

We trust this feasibility report meets your current needs. If you have any questions or wish to further discuss our observations, conclusions, and recommendations, please contact Ryan White at 503.417.7608 or Mark Swank at 503.417.7738.

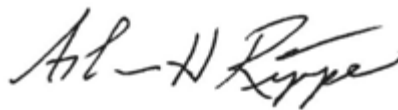
Sincerely,  
PBS Engineering and Environmental Inc.



Mark Swank, RG, CEG  
Senior Engineering Geologist



Ryan White, PE, GE  
Geotechnical Discipline Lead



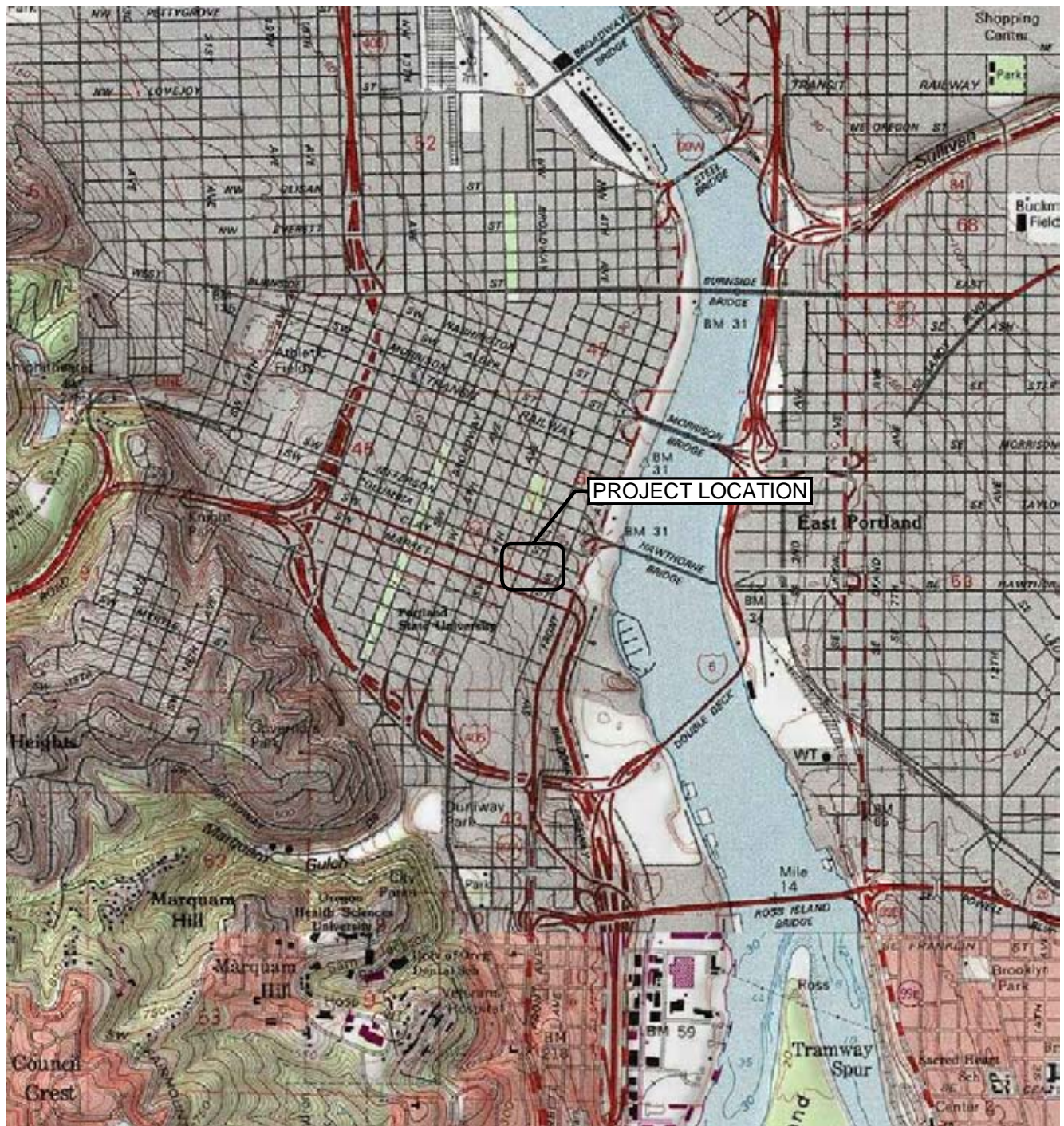
Arlan H. Rippe, PE, GE, D.GE  
Senior Geotechnical Consultant

MS/RW/AR/rd

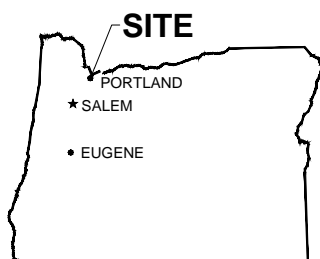
Figures:      Figure 1 – Vicinity Map  
                    Figure 2 – Site Plan  
                    Figure 3 – Geology Map  
                    Figure 4 – Local Faults  
                    Figure 5 – Liquefaction Hazard  
                    Figure 6 – Ground Motion Amplification



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SOURCE: USGS PORTLAND OR QUADRANGLE 1990.



OREGON



SCALE: 1" = 2,000'

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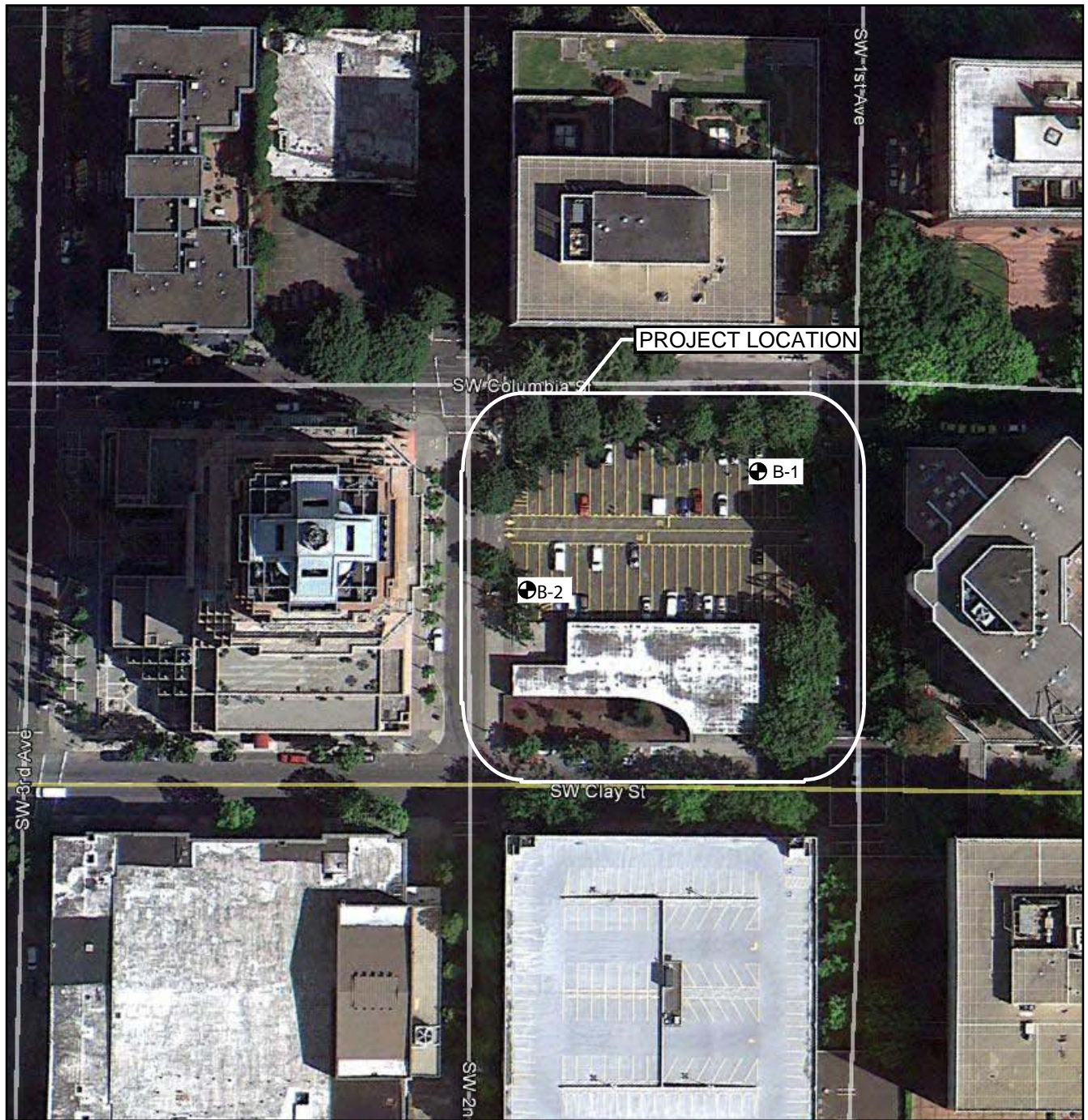
DATE  
JAN 2015

VICINITY MAP  
BLOCK 128  
PORTLAND, OREGON

FIGURE

1





SOURCE: © 2011 GOOGLE EARTH PRO, © 2012 GOOGLE

### LEGEND

- B-1 BORING NUMBER AND LOCATION FROM GEOCON REPORT  
DATED MAY 2001



SCALE: 1" = 100'

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PHASE: 0002

DATE  
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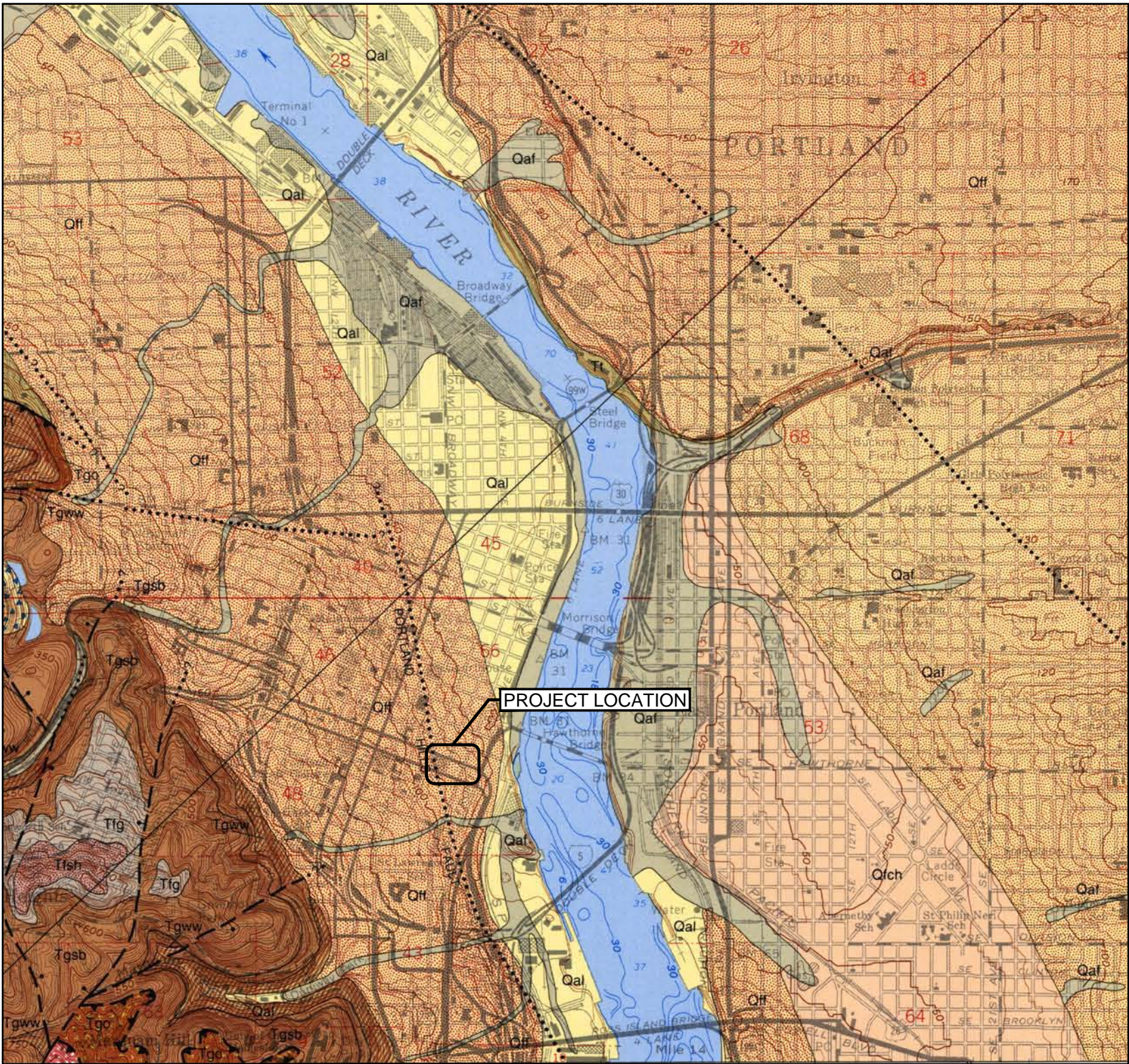
**SITE PLAN**  
BLOCK 128  
PORTLAND, OREGON

FIGURE

**2**



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**GEOLOGY MAP**

SOURCE: GEOLOGIC MAP OF THE PORTLAND QUADRANGLE, MULTNOMAH AND WASHINGTON COUNTIES, OREGON, AND CLARK COUNTY, WASHINGTON, GMS-75, 1991



**Artificial fill (Holocene)** — Sand, silt, and clay fills with subordinate amounts of gravel, debris, and local concentrations of sawdust and mill ends. Unit **Qaf** is mapped only where fill has eliminated lakes, sloughs, marshes, or gullies delineated during 1898 survey for earliest topographic map of Portland (U.S. Geological Survey, 1905). Fill areas mapped with queried contacts represent lakes and marshes that may have been drained rather than filled. Fill 1.5 to 5 m thick is common in developed areas of Columbia and Willamette floodplains, but thickness and distribution are highly variable, and it is not depicted on this map.

**Alluvium (Quaternary)** — River and stream deposits of silt, sand, and organic-rich clay with subordinate gravel of mixed lithologies; largely confined to Columbia and Willamette River channels and valley bottoms of tributary streams; may include local lacustrine, paludal, and eolian deposits. Unit **Qal** reaches maximum thickness of 45 m.

**Channel facies (Pleistocene)** — Complexly interlayered and variable silt, sand, and gravel deposited in major flood channel. Channel is cut in earlier and/or contemporaneous fine and coarse flood sediments (units **Qff** and **Qfe**) and retains much of original morphology. Irregular postflood surfaces of these deposits have been locally filled by bog or pond sediments and by overbank alluvium from minor streams. Channel deposits are typically 5 to 15 m thick.

**Fine-grained facies (Pleistocene)** — Coarse sand to silt deposited by catastrophic floods. Silt and fine sand composed predominantly of quartz and feldspar with white mica. Coarser sand composed predominantly of Columbia River basalt. Poorly defined beds of 30-cm to 1-m thickness are observed in outcrop. Locally, beds are separated by accumulations of clay and iron oxide 1 to 6 cm thick, which may be paleosols. Modern soil development commonly introduces abundant clay and iron oxides into upper 2 to 3 m of deposits. Fine sediments are locally thick in lower elevations of area and extend upslope as mantle to elevations between 90 and 105 m. Unit **Qff** reaches maximum thickness of 30 to 40 m. Unit **Qff** is equivalent to Willamette Silt of Allison (1953) and includes lacustrine sand, lacustrine silt and clay, and sand and silt deposits of Trimble (1963).

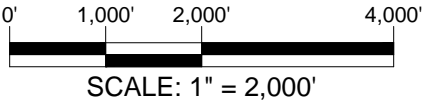
**Troutdale Formation (Miocene to Pliocene)** — Friable to moderately strong conglomerates with minor interbeds of sandstone, siltstone, and claystone. In Tualatin Mountains, conglomerates consist of well-rounded pebbles and cobbles of Columbia River basalt and exotic volcanic, metamorphic, and plutonic rocks. Troutdale conglomerates exposed east of Willamette River consist of Boring Lava and High Cascade basalt, and dacite in addition to Columbia River basalt and exotic clasts. Conglomerate matrix and interbeds in both areas contain varying amounts of feldspathic, quartzo-micaceous, and volcanic lithic and vitric sediment. Lithology of sediments and ratio of conglomerate to sandstone and siltstone vary widely throughout area. Unit **Tt** reaches maximum thickness of 60 to 90 m in map area and is up to 275 m thick in other parts of Portland Basin (Swanson, 1986). Trimble (1963), Swanson (1986), and Tolan and Beeson (1984) have shown that rocks mapped as Troutdale Formation in many parts of Portland area are Miocene to late Pliocene. Age of unit **Tt** in Portland quadrangle is unknown.

**Basalt of Sand Hollow (middle Miocene)** — Two flows are present within map area. Flows are typically blocky to columnar jointed but occasionally display entablature/colonnade jointing style. Fresh exposures are dark gray to black, weathered surfaces typically greenish gray to black. These Sand Hollow flows (probably “+4” and “+5” units of Beeson and others, 1975) are fine to coarse grained, occasionally diktytaxitic, and sparsely plagioclase-phyric, with phenocrysts <2 cm in size. Unit thickness is variable, ranging to >60 m. Sand Hollow flows can be distinguished from Ginkgo flows on the basis of stratigraphic position, lithology, and composition (Beeson and others, 1989b). Only low-P<sub>2</sub>O<sub>5</sub> compositional type (Beeson and others, 1985) is present within map area. Beeson and others (1985) report average K-Ar date of 15.3 Ma for this unit.

**Sentinel Bluffs unit (middle Miocene)** — Within map area, two flows are present. These were formerly designated as “-1” and “-2” flows of Beeson and Moran (1979). Each, however, may consist of two or three flow units along east side of Tualatin Mountains in Washington Park. Flows typically display blocky to columnar jointing and rarely entablature/colonnade jointing pattern. Fresh exposures are light to dark gray, weathered surfaces greenish gray to dark gray. Lower flow is typically fine- to medium-grained basalt and sparsely plagioclase-phyric, with small (<0.5 cm), tabular plagioclase phenocrysts. Upper flow is fine to medium grained, commonly diktytaxitic, and aphyric. Unit is up to 50 m thick within map area. Sentinel Bluffs flows are distinguished from both younger Frenchman Springs units and older Grande Ronde units on the basis of stratigraphic position, composition (Beeson and others, 1989b), lithology, and normal paleomagnetic polarity (see Reidel and others, 1989; Beeson and others, 1989a). Long and Duncan (1982) report “Ar” Ar date of approximately 15.6 Ma for youngest flows of unit on Columbia Plateau.

**Winter Water unit (middle Miocene)** — Within map area, two flows are present, formerly designated as “-3 flow” of Beeson and others (1975) or “N<sub>2</sub> low-MgO flows” of Beeson and Moran (1979). Winter Water flows display wide range of jointing patterns, from columnar to entablature/colonnade. Fresh exposures are dark gray to black, weathered surfaces greenish gray to grayish black. Both flows are typically glassy to fine grained and phyric to abundantly phyric, with small (<0.3 cm) plagioclase glomerocrysts that often display distinctive radial or spoke-shaped habit. Distribution of glomerocrysts is often uneven and tends to be less abundant in basal portion of flow. Unit thickness ranges from 7.5 to 30 m within map area. Winter Water flows are distinguished from other Grande Ronde units on the basis of lithology, composition (Beeson and others, 1989b), stratigraphic position, and normal paleomagnetic polarity (see Reidel and others, 1989; Beeson and others, 1989a).

**Ortley unit (middle Miocene)** — Within map area, one or two flows are present, formerly designated as “N<sub>2</sub> low-MgO flows” of Beeson and Moran (1979). Ortley flows commonly display entablature/colonnade jointing style. Fresh exposures are gray to black, weathered surfaces greenish gray to dark gray. Flows are commonly glassy to very fine grained and aphyric. Unit thickness ranges from 7.5 to >60 m within map area. Ortley flows are both compositionally (Beeson and others, 1989b) and lithologically similar to older Grouse Creek unit of Reidel and others (1989) but can be distinguished on the basis of their normal paleomagnetic polarity (see Reidel and others, 1989; Beeson and others, 1989a).



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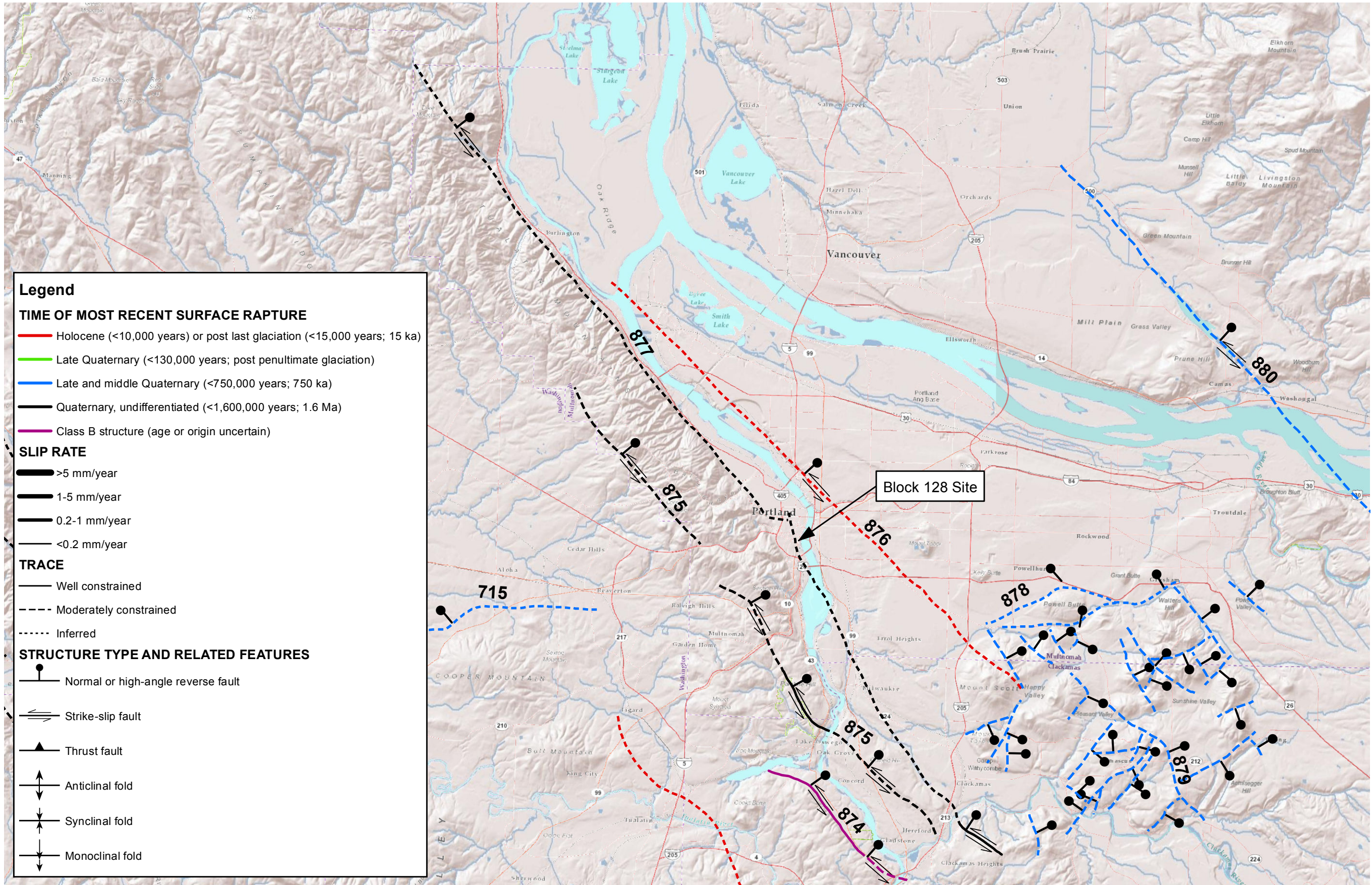
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www.pbsenv.com

MULTNOMAH COUNTY COURTHOUSE  
BLOCK 128  
MULTNOMAH COUNTY  
PORTLAND, OREGON

**GEOLOGY**  
**MAP**  
PROJECT: 15914.869  
PHASE 0002  
DATE: JANUARY 2015  
FIGURE:  
**3**



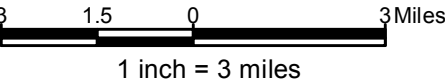
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SOURCES: U.S. Geological Survey, 2006, Quaternary fault and fold database for the United States, accessed 01/14/2015, from USGS web site: <http://earthquakes.usgs.gov/regional/qfaults/>.

# LOCAL FAULTS

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# PBS

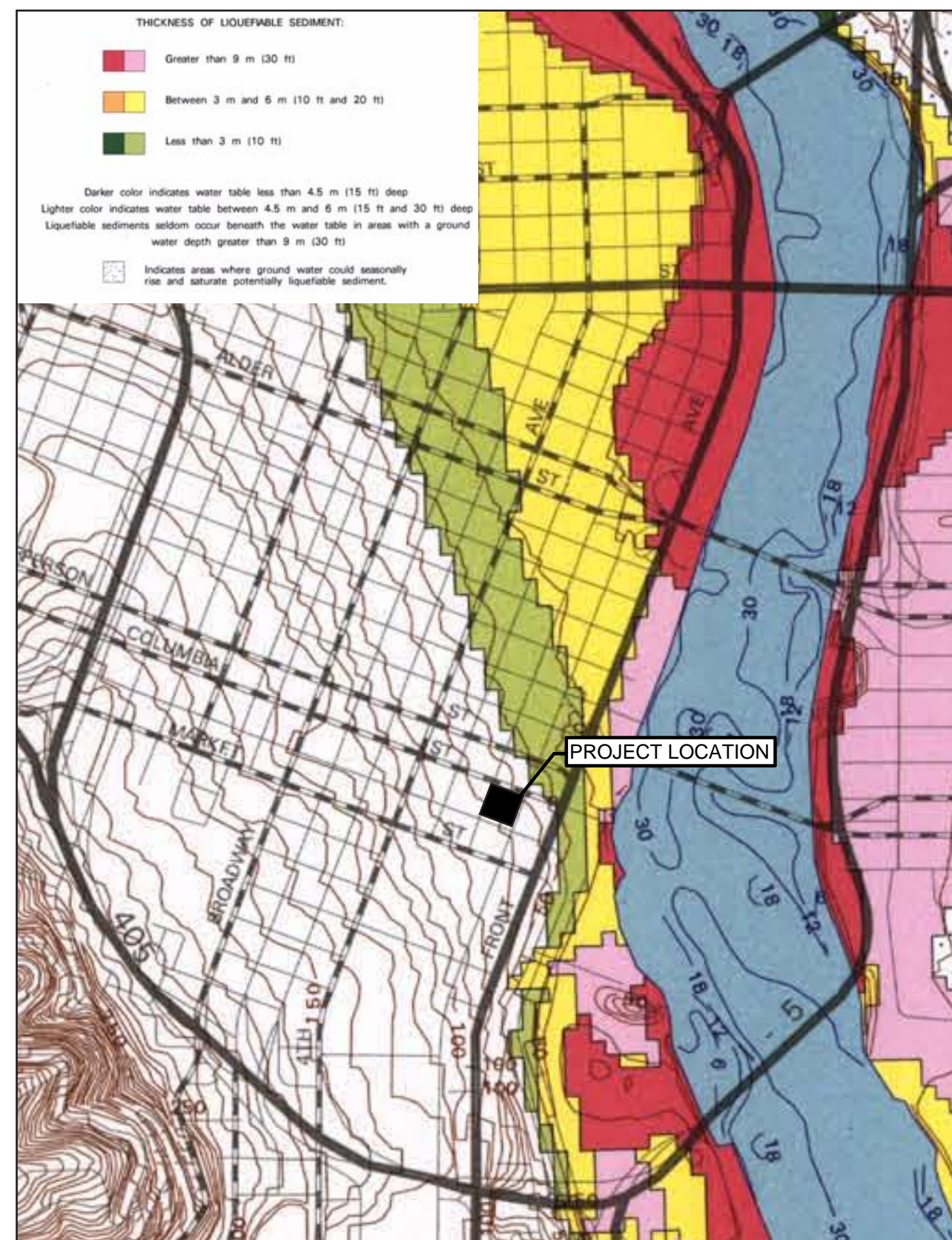
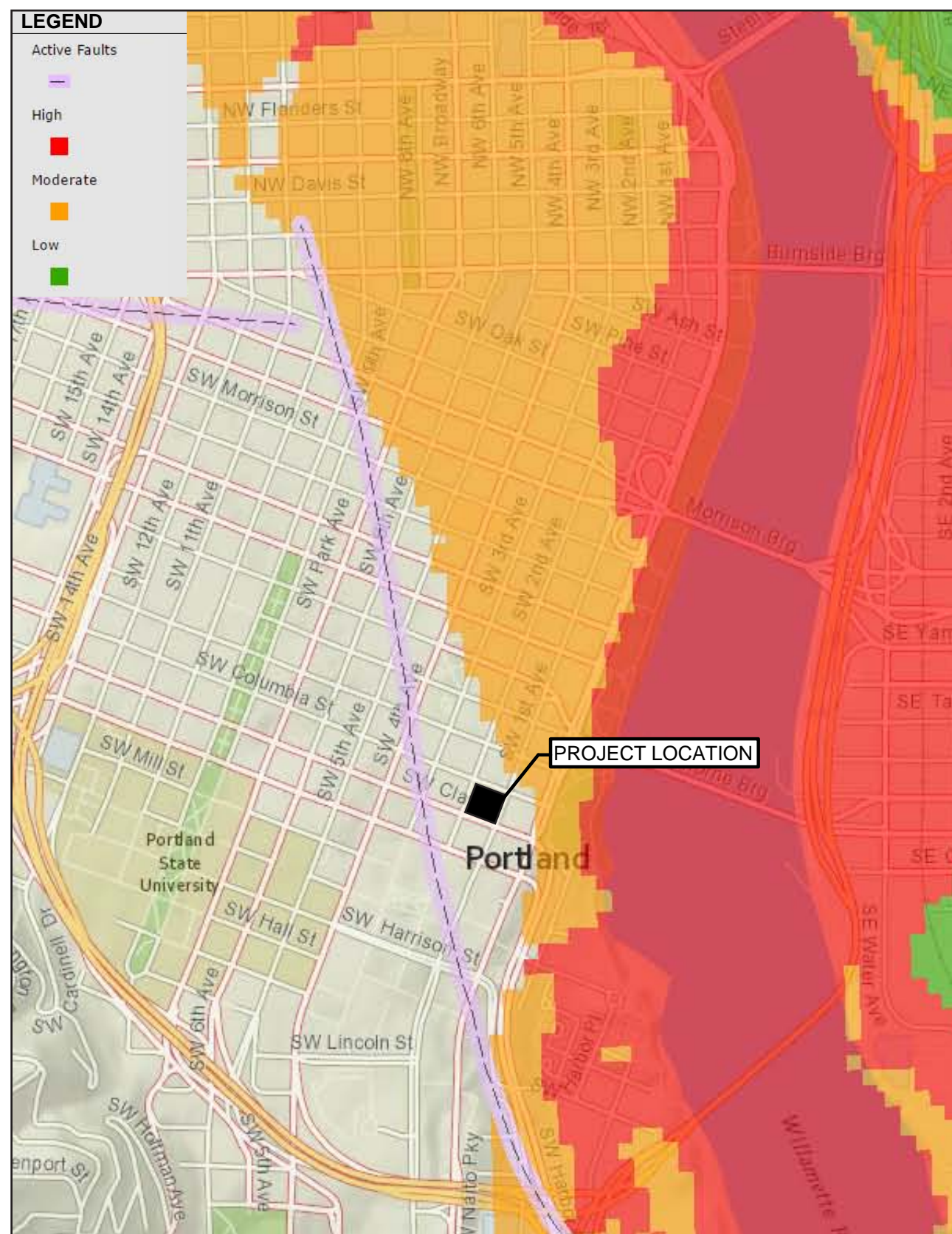
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## MULTNOMAH COUNTY COURTHOUSE BLOCK 128 MULTNOMAH COUNTY PORTLAND, OREGON

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PHASE	0002
DATE	JAN 2015
FIGURE	

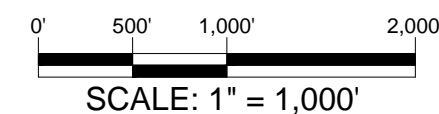
4





## EARTHQUAKE LIQUEFACTION HAZARD

SOURCES: DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES, OREGON HAZVU, STATEWIDE GEOHAZARDS VIEWER, WEBSITE: [HTTP://WWW.OREGONGEOLOGY.ORG/HAZVU/](http://www.oregongeology.org/HAZVU/)  
DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES, LIQUEFACTION SUSCEPTIBILITY MAP, PORTLAND QUADRANGLE, MULTNOMAH AND WASHINGTON COUNTYES,  
OREGON, AND CLARK COUNTY WASHINGTON, GMS-79, PLATE 1, 1993



MULTNOMAH COUNTY COURTHOUSE  
BLOCK 128

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MULTNOMAH COUNTY  
PORTLAND, OREGON

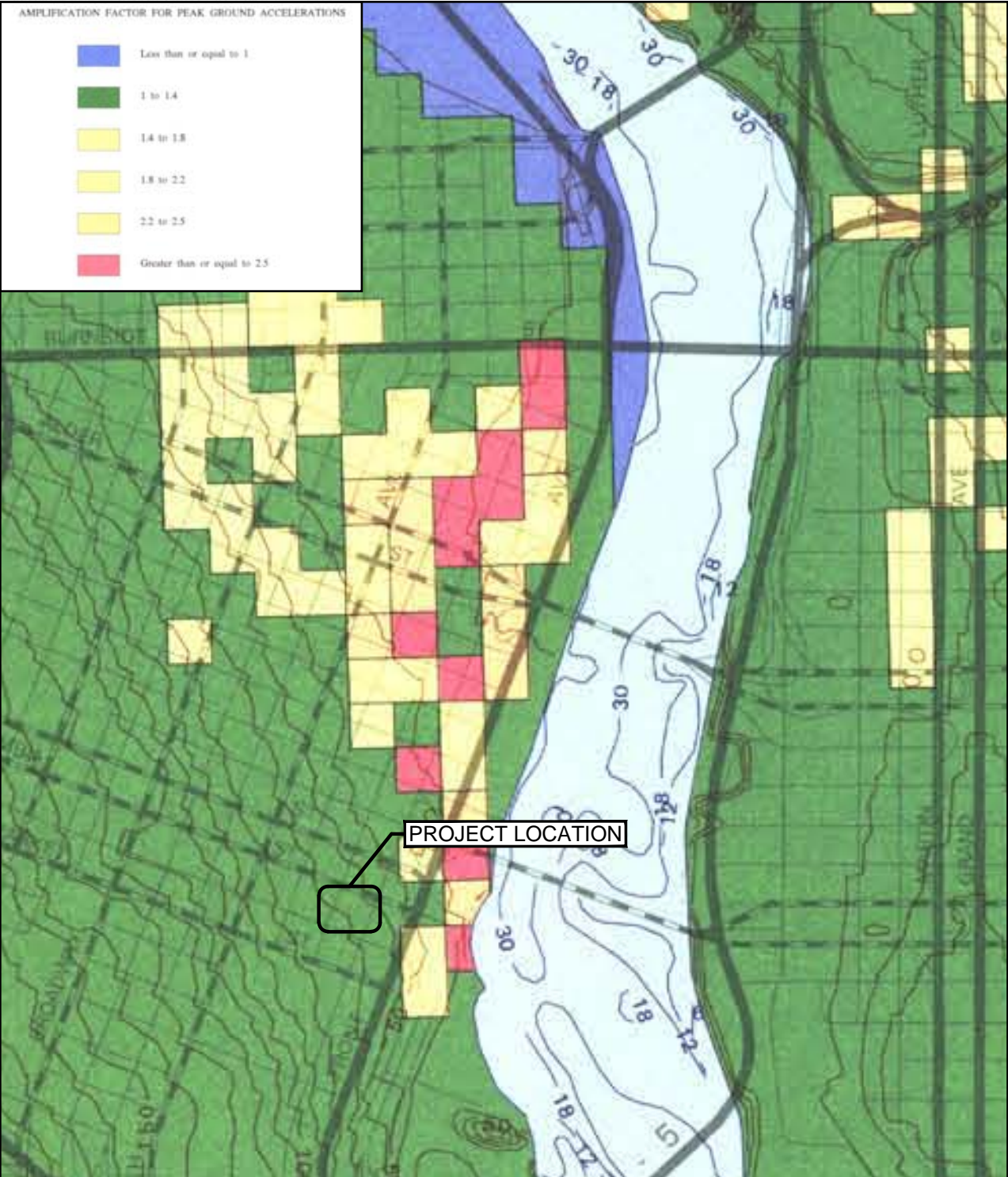
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DATE:	JANUARY 2015

FIGURE:

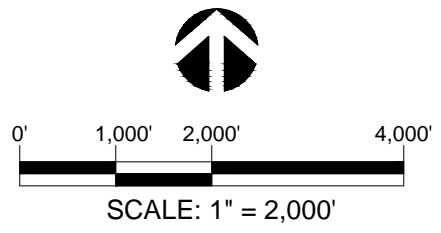
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PREPARED FOR: MULTNOMAH COUNTY

SOURCES: DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES, LIQUEFACTION SUSCEPTIBILITY MAP, PORTLAND QUADRANGLE, MULTNOMAH AND WASHINGTON COUNTIES, OREGON, AND CLARK COUNTY WASHINGTON, GMS-79, PLATE 2, 1993



	PROJECT # 15194.869 PHASE: 0002	<b>GROUND MOTION AMPLIFICATION</b>  BLOCK 128 PORTLAND, OREGON	FIGURE  <b>6</b>
	DATE JAN 2015		

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DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	<b>BORING B 1</b>		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.)	DATE COMPLETED			
					60	4/1/01			
					EQUIPMENT B-57 MUD ROTARY				
0					MATERIAL DESCRIPTION				
					Approximately 2 inches ASPHALT				
2	B1-1			GM	<b>FILL</b> Dense, moist, GRAVEL, occasional brick				41
4	B1-2								50/1"
6	B1-3								24
8									22.6
10	B1-4			ML-SM	<b>WILLAMETTE SILT</b> Stiff to very stiff, interbeds of moist, brown, fine-grained Sandy SILT.				30
12					And Medium dense, moist, brown, fine-grained SAND with varying percentages of silt.				87.2
14									20.3
16	B1-5				Occasional lenses of medium dense, fine to medium-grained SAND				14
18									29.5
20	B1-6								18
22									37.5
24									
26	B1-7								17
28									27.5

Figure A-1, Log of Boring-B 1

100CT

## 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. P1084-05-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	<b>BORING B 1</b>		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) <u>60</u>	DATE COMPLETED <u>4/1/01</u>			
					EQUIPMENT <u>B-57 MUD ROTARY</u>				
					MATERIAL DESCRIPTION				
30	B1-8				Stiff, wet, light brown SILT		12		36.3
32									
34									
36	B1-9			ML-SM	-Slight mottling		16		36.0
38									
40					-Becomes softer at 39 feet				
42									
44					<b>TROUTDALE FORMATION</b> Very dense GRAVELS and COBBLES  BORING TERMINATED AT 44 FEET (Drilling rate ~ 12" per hour) Groundwater was not encountered				

Figure A-2, Log of Boring-B 1

100CT

SAMPLE SYMBOLS	<input type="checkbox"/> ... SAMPLING UNSUCCESSFUL	<input type="checkbox"/> ... STANDARD PENETRATION TEST	<input checked="" type="checkbox"/> ... DRIVE SAMPLE (UNDISTURBED)
	<input checked="" type="checkbox"/> ... DISTURBED OR BAG SAMPLE	<input checked="" type="checkbox"/> ... CHUNK SAMPLE	<input checked="" type="checkbox"/> ... 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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	<b>BORING B 2</b>		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.)	DATE COMPLETED			
					65	4/1/01			
					EQUIPMENT B-57 MUD ROTARY				
0					MATERIAL DESCRIPTION				
					Approximately 2 inches ASPHALT				
2	B2-1			GM	FILL Dense, moist GRAVEL, occasional brick				
4							50		
6	B2-2			ML-CL	WILLAMETTE SILT Stiff, moist, brown/gray Clayey SILT to Silty CLAY				34.5
8	B2-3						11		
10	B2-4						13		38.0
12					Interbeds of stiff to very stiff, moist, brown, fine-grained Sandy SILT				
14					And Medium dense, moist, brown, Silty fine-grained SAND				
16	B2-5						16		20.9
18					Lenses of medium dense, moist, brown, fine to medium-grained SAND				
20	B2-6						27	87.9	34.0
22							23		26.5
24									
26	B2-7						19		19.4
28									

Figure A-3, Log of Boring-B 2

100CT

## 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. P1084-05-01

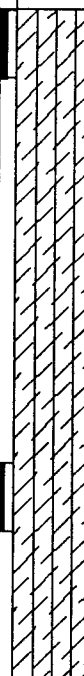

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 2			PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.)	65	DATE COMPLETED			
					EQUIPMENT			B-57 MUD ROTARY		
MATERIAL DESCRIPTION										
30	B2-8			ML-CL	Very dense, moist, brown, fine-grained SAND, varying percentages of silt			50	90.1	19.1
32										
34										
36										
38										
40	B2-9				-Color change to brown/rust and black			45		27.6
42										
44										
	B2-10			GM	TROUTDALE FORMATION Very dense GRAVELS and COBBLES			50/4.5"		28.4
BORING TERMINATED AT 45.5 FEET (Drilling rate ~ 12" per hour) Groundwater was not encountered *Thin-walled sample										

Figure A-4, Log of Boring-B 2

100CT

SAMPLE SYMBOLS	□ ... SAMPLING UNSUCCESSFUL	■ ... STANDARD PENETRATION TEST	■ ... DRIVE SAMPLE (UNDISTURBED)
	▣ ... DISTURBED OR BAG SAMPLE	▣ ... CHUNK SAMPLE	▽ ... WATER TABLE OR SEEPAGE

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