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Multnomah County Facilities Management
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Re: Geotechnical Engineering Feasibility Assessment Task 1 and 2
Due Diligence Services – Multnomah County Courthouse
Hawthorne Bridge Head Site, Portland, Oregon
PBS Project No. 15194.869

INTRODUCTION AND BACKGROUND

PBS Engineering and Environmental, Inc. (PBS) is pleased to provide this feasibility/due diligence report for geotechnical engineering services in support of site selection for the Multnomah County Courthouse in Portland, Oregon (Figure 1, Vicinity Map). The Hawthorne Bridge Head (site) is being considered as the site of a new courthouse building. The 0.9-acre site and is bounded by SW Madison and SW Jefferson Street to the north and south, respectively, and SW Naito Parkway and SW 1st Avenue to the east and west (Figure 2, Site Plan). Based on available topographic data, ground surface elevations at the site range from 55 feet to 46 feet, at the northeastern and southeastern corners, respectively (WGS84 EGM96 Geoid).

An existing structure on the southwestern portion of the site includes a three-story reinforced concrete building with an attached, single-story brick restaurant. An asphalt concrete surface road (an abandoned, historic Hawthorne bridge approach) curves up and around from the southeastern to northeastern corners of the lot. The remainder of the site is covered with grass lawn, landscaping including shrubs and flowers, and occasional trees. The site has been used in this way since at least 1990 based on dated Google Earth™ imagery. Based on our conversations with Multnomah County (County) personnel and experience with similar projects, the development will include the following.

- A 14- to 17-story, steel-frame, high-rise building with one level below grade
- A building footprint 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, 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.

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 on the west by Tertiary marine sedimentary and volcanic rocks of the Coast Range and on the east by Tertiary and Quaternary volcanic and volcaniclastic 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 overly 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; Richter local magnitude [M_L] 5.2) and the Portland Northern Willamette Valley (M_L 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 M_L 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 observe d directly The northwest-trending East Bank Fault is located approximately 1½ 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 2½ 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 ±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 completed at the site and referenced in the 1999 GRI report and exploration completed on the site northeast of the subject site discussed in the 2006 GeoDesign Report both indicate the presence of silt, sand, and gravel fill underlain by alluvial silt, sand, and gravel. The depth of explorations completed on-site were not discussed. However, explorations on the site to the northwest ranged from about 65 feet to 150 feet below the existing 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 depths of 5 to 15 feet bgs. Fill was underlain by alluvial sediments generally consisting of silt and 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 GeoDesign during drilling at the nearby site including blow counts per foot (N-values [standard penetration resistance]), dry densities, and moisture content ranges.

Table1: Summary of Subsurface Conditions within Hawthorne Bridge Head Site[a]

Soil Type	Depth Range (ft[b] bgs)	N Values (blows/ft)	Consistency / Relative Density	Moisture Content Range (%)	Dry Density (pcf)[c]
Fill (Silt with some sand and gravel; Gravel with silt and sand)	8 to 14	Between 3 and 7 (silt); 8 and 57 for 11-inch (gravel)	Soft to medium stiff; medium dense to very dense	30 to 46	74 to 87
SILT and SAND	8 to 20	Between 2 and 8	Soft (silt); very loose to medium dense (sand)	32 to 43	71 to 83
GRAVEL (with variable sand and silt interbeds)	14 to 20+	14 to 50+ for 2 inches or less	Medium dense to very dense	74 to 90	N/A

a Information summarized from GeoDesign, Inc. 2006, Report of Geotechnical Engineering Services.
b. b = feet
c. pcf = pounds per cubic foot

GROUNDWATER

Groundwater information in the site vicinity was obtained from the Oregon Department of Water Resources (OWRD) local well logs, previous geotechnical explorations in the site vicinity, and USGS groundwater contours. Nearby explorations indicate groundwater is likely present at depths of 30 to 35 feet bgs in the site vicinity. USGS regional groundwater contours show groundwater may be present at a depth of about 32 feet bgs (elevation 15 feet City of Portland [COP] datum) and we anticipate this level could fluctuate between 20 and 30 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 400 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 Table 2 as follows. 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. HazVu, FEMA).

Table 2: Summary of Potential Geologic and Seismic Hazards at Hawthorne Bridge Head

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[a]	Low
	Standing Water	Low
Hydrogeology and Groundwater	Shallow Groundwater[b]	Low
	Seepage and Piping	Low
	Permeability and Percolation	Low
Seismic Hazards	Ground Amplification[c,d]	Moderate
	Local Fault Rupture[d]	Low
	Liquefaction and Lateral Spread[c,d]	Low[e]
	Seismically-Induced Settlement	N/A
	Ground Lurching or Cracking	Low
	Seismically-Induced Slope Instability	N/A
	Tsunami	N/A
	Seiches	Low

[a] 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.
[b] Groundwater is assumed to be hydraulically connected to the Willamette River elevation. Anticipated to be approximately between 30 and 35 feet bgs.
[c] Information from the Department of Geology and Mineral Inustries (DOGAMI), GMS-79, plates 1 and 2.
[d] Information from the Department of Geology and Mineral Industries (DOGMAI), Oregon HazVu: Statewide Geohazards Viewer, http://www.oregongeology.org/hazvu/.
[e] Although DOGAMI mapping indicates a potential for liquefaction at the site, subsurface conditions (i.e. non-cohesive soils and depth of groundwater) would correspond to a low hazard that will impact the building design and site development.

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, and ground shaking (refer, Figure 4, Local Fault Map and Figure 6, Ground Motion Amplification). Current mapping shown in GMS-79 (Plate 1) and through the online HazVu program indicate liquefaction and lateral spreading may be potential hazards at the site. However, based on subsurface conditions in the site vicinity and depth to groundwater, our current opinion is that the risk of liquefaction and lateral spreading at the site is low. 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 400 feet east, could affect the site.

The 2014 OSSC methodology defines six soil categories that are based on average shear-wave velocity in the upper 100 feet (30 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 C.

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 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 Fa, and Fv, and based on site classification, C as described above. However, if site-specific testing and analyses indicates 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 Hawthorne Bridge Head site without excavation down to the gravel expected at depths of about 20 to 30 feet bgs. Even then, the presence of interbedded silt and sand layers below that elevation may preclude the use of a mat. Subsequently, deep foundations will probably 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 mitigating 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 Hawthorne Bridge Head Site

Deep Foundation Type	Advantages	Disadvantages
Driven Pile	• Commonly available materials • Easily installed using readily available equipment • Displaces, vibrates, remolds and consolidates soil around pile • Structural integrity is better known when compared with other deep foundation types • Pile capacity can be inferred from driving resistance • Relatively unaffected by groundwater	• Capacity is limited when compared with drilled shaft foundations • Installation can be loud and create vibrations • Mobilization can be expensive and the general weight of the equipment can be relatively heavy • Piles may need to be stored onsite constraining the work area • Flexure of the piles is higher than other deep foundations and durability can be a concern • Installation equipment may need a large clearance area.
Drilled Shafts	• Can be relatively larger in diameter than driven piles possibly reducing the cost of construction (by reducing the number of piles) • Can be constructed in dense soils where driven piles aren't practical. • Pile strength can be increased by increasing the diameter and reinforcing • Reduced disturbance and vibration compared to driven piles • Soil conditions can be observed during construction. • Equipment is relatively light when compared with pile driving equipment • Relatively better for resisting lateral loads due to larger possible diameter	• Can be relatively expensive compared to driven piles • Shaft cleaning is necessary and can be difficult • Drilling can be difficult in cobbles boulders • For looser soils, stabilization may be needed which may include casing or slurry • Where groundwater is present, casing may be required • Concrete installation can be difficult which may result in mud inclusions and air voids within the shaftEnd bearing capacity is generally negligible.
Drilled Augercast Pile	• Provides continuous support of drilled hole sidewalls eliminating the need for shoring in soft, loose, or saturated soils • Pile strength can be increased by increasing the diameter and reinforcing • Reduced disturbance and vibration compared to driven piles	• Cannot achieve significant penetration into gravels and cobbles • Produces spoils that must be transported off-site • Disturbance to the ground surface from spoils • Concrete installation can be difficult which may result in mud inclusions and air voids within the pile • End bearing capacity is generally negligible.

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, or sheet piles with braces or struts. Sheet piles 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 showing 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.

Due to the presence of brick structures on the site that will remain, it may be necessary to consider shoring that is generally more rigid and can be constructed in a manner to provide continuous support of soils supporting the foundations of these structures. Possible shoring systems that meet this criteria could include a soldier pile and lagging wall using sheet pile or steel sheets as lagging that is installed prior to excavation. Alternatively, a tangent pile wall that consists of drilled concrete piles installed immediately adjacent or relatively close to one another to act as lagging. Tangent pile walls designed to cantilever (without tiebacks) will require reinforcing such as H-piles or rebar cages. Depending on the depth of excavation and subsurface conditions, reinforcing is typically installed in every other to every third or fourth pile. If drilled concrete piles are used for building support, it may be feasible to use the same equipment to constructed the shoring around the existing structures and reduce mobilization costs if using another system.

Vibration

Vibration monitoring and controls may be necessary during construction. Although blasting is unlikely at the site, the City of Portland’s 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 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 Table 4 as follows.

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)[a]	2
Reinforced Concrete Structures[a]	4
Steel Structures[a]	4
Buried Utilities[a]	2
Wells and Aquifers[a]	2
Green Concrete (Less than 7 days)[a]	1
Institutional land uses with primarily daytime use[b]	3.11[c]
TV Studios[b]	1[c]

[a] 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), accessed 12/4/2013, effective date January 9, 2014

[b] Federal Transit Administration, *Transit Noise and Vibration Impact Assessment*, Section 8.0 Vibration Impact Criteria (May 2006)

[c] 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 the 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 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 Hawthorne Bridge Head site is suitable for the proposed development but will require specific geotechnical considerations during design and construction. The geotechnical-related considerations include the following.

- 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 significant 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 1.4 and 2.5.
- Liquefaction and lateral spreading, as mapped at the site by GMS-79 Plate 1 and HazVu, are potential hazards of low to moderate concern. However, based on the depth of groundwater, our current opinion is that the risk of liquefaction and lateral spreading at the site is low.
- Groundwater levels will likely fluctuate with changes in the Willamette River stages. In general, groundwater is anticipated to be about 30 feet bgs according to regional groundwater mapping by the USGS and other resources.
- The Site Class is anticipated to be C based on 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; and therefore, vibration impacts assessment and monitoring during construction would be highly advisable.
- Due to the presence of older structures to remain on the property, specialty shoring and/or underpinning of the existing building foundations may be necessary to accommodate excavation of one level below-grade for construction of the new courthouse.

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.

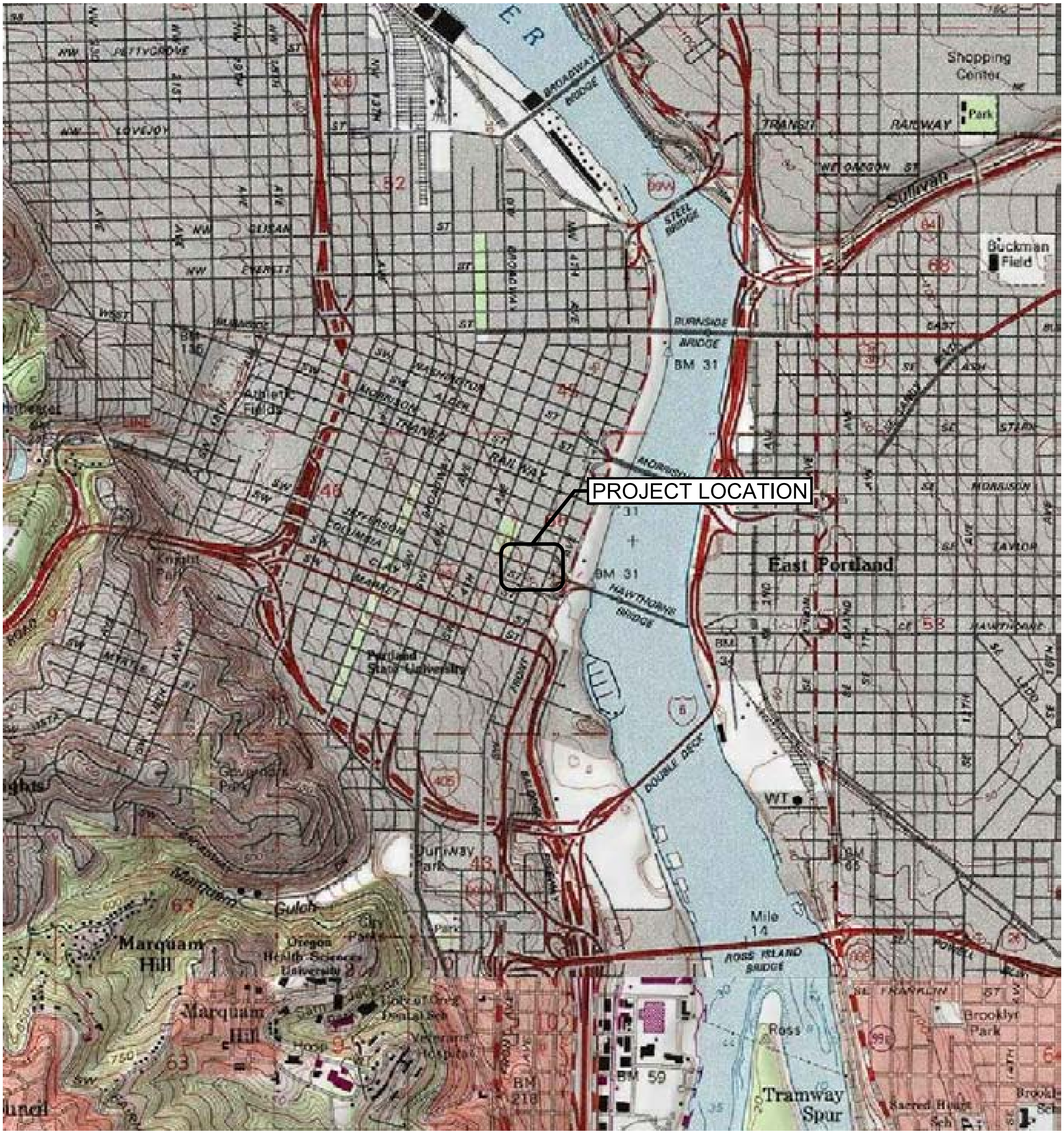
Mark W. Swank
Mark Swank, RG, CEG
Senior Engineering Geologist

Ryan White, PE, GE
Geotechnical Discipline Lead

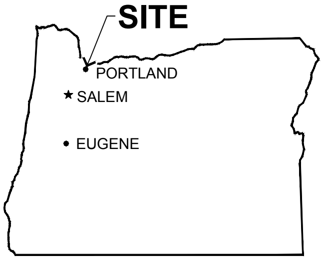
Arlan H. Rippe, PE, GE, D.GE
Senior Geotechnical Consultant

MS/RW/AR/rd

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



SOURCE: USGS PORTLAND OR QUADRANGLE 1990.



OREGON

0' 1,000' 2,000' 4,000'

SCALE: 1" = 2,000'

PREPARED FOR: MULTNOMAH COUNTY

PBS	PROJECT # 15194.869 PHASE: 0002	VICINITY MAP HAWTHORNE BRIDGE HEAD PORTLAND, OREGON	FIGURE 1
	DATE JAN 2015		

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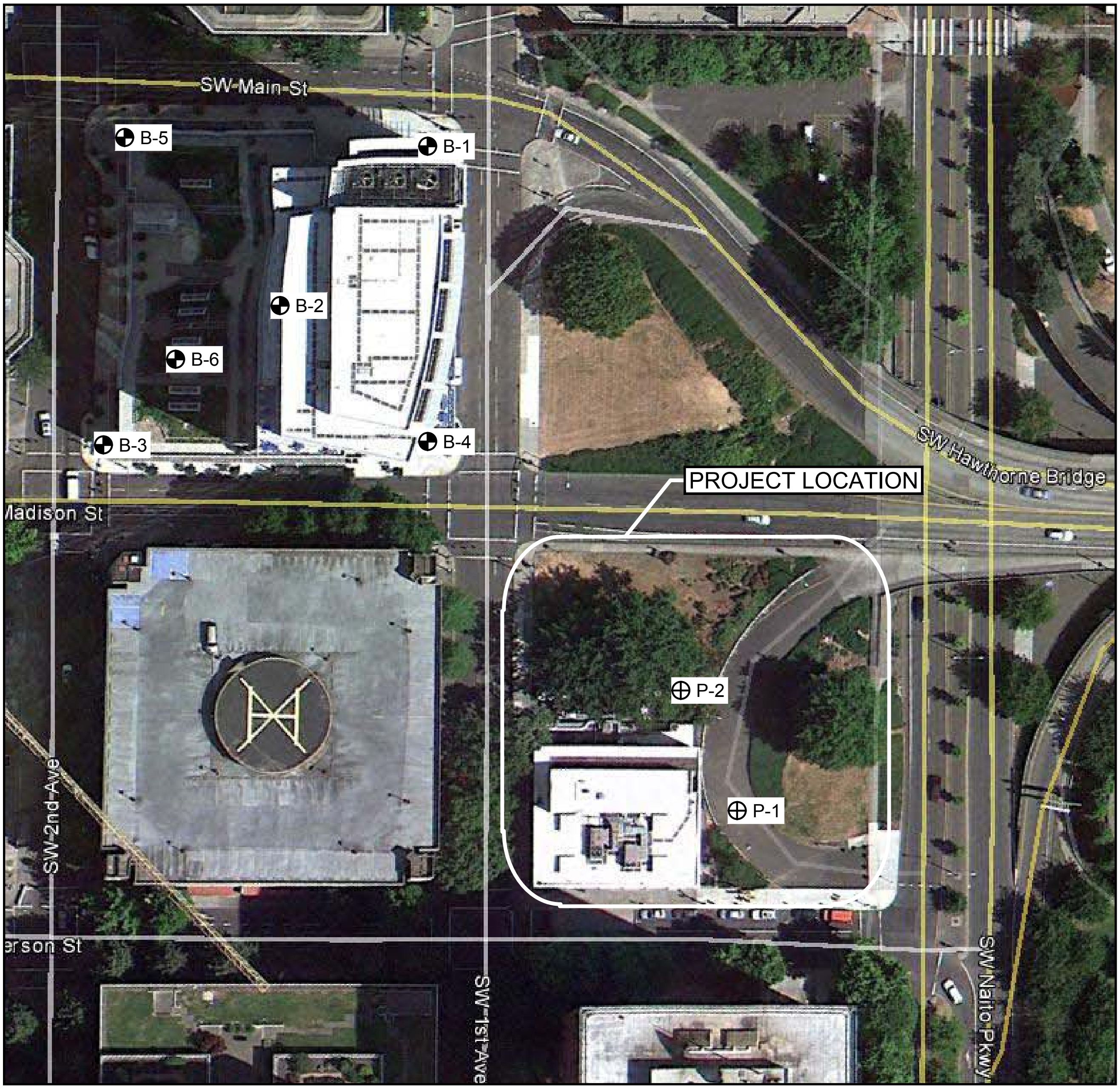

SOURCE: © 2011 GOOGLE EARTH PRO, © 2012 GOOGLE

LEGEND

◐ B-1 BORING NUMBER AND LOCATION FROM GEODESIGN REPORT DATED JUNE 2001 (B-1 THROUGH B-5) AND APRIL 2006 (B-6)

⊕ P-1 BORING NUMBER AND LOCATION FROM GRI REPORT DATED APRIL 1999

0' 50' 100' 200'

SCALE: 1" = 100'

PREPARED FOR: MULTNOMAH COUNTY

PBS

PROJECT #
15194.869
PHASE: 0002

DATE
JAN 2015

SITE PLAN

HAWTHORNE BRIDGE HEAD
PORTLAND, OREGON

FIGURE
2

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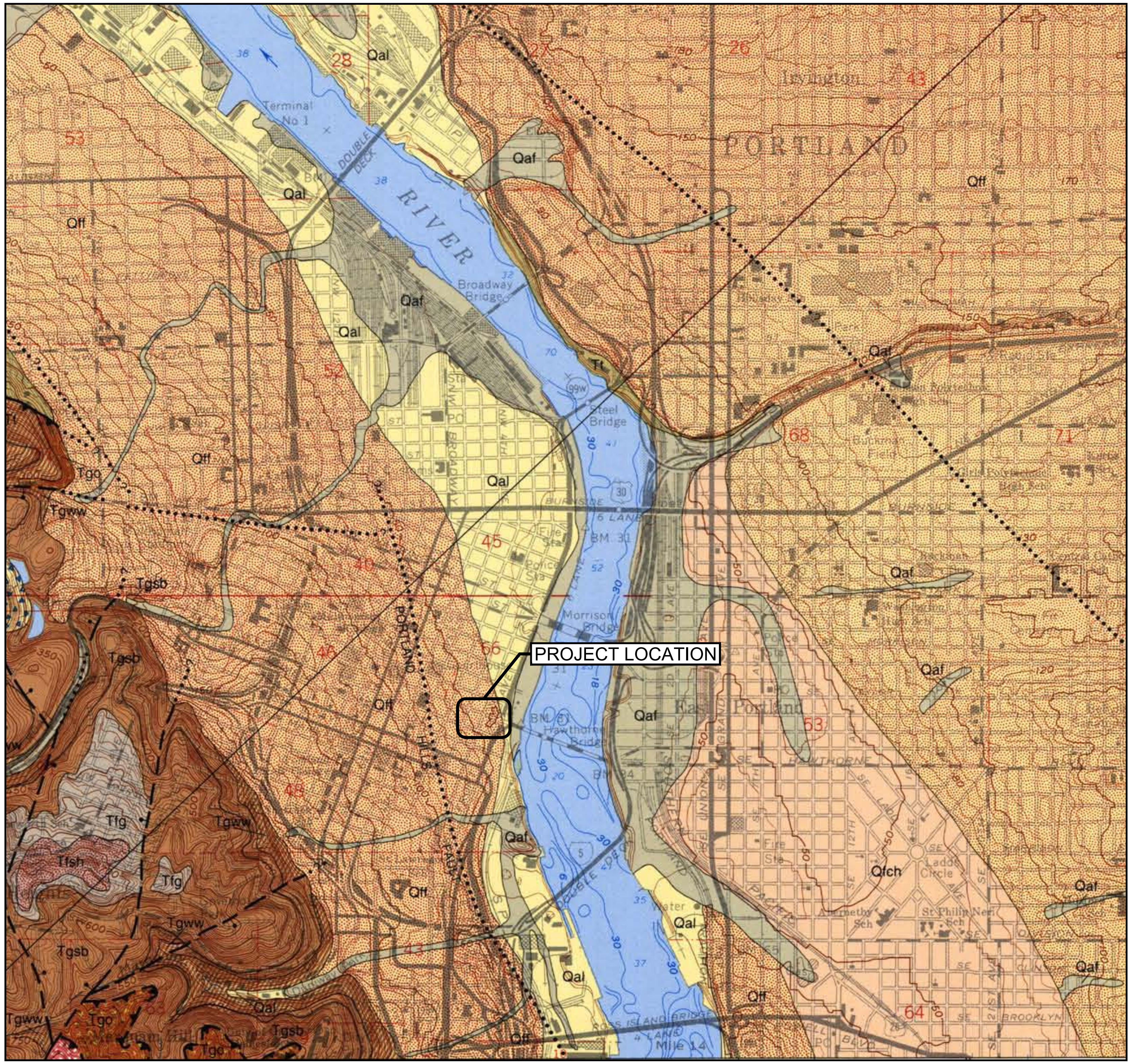

Qaf	**Artificial fill (Holocene)** — Sand, silt, and clay fills with subordinate amounts of gravel, debris, and local concentrations of sawdust and mill ends. Unit **Qaf** is mapped only where fill has eliminated lakes, sloughs, marshes, or gullies delineated during 1898 survey for earliest topographic 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
Qal	**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
Qfch	**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 **Qfc**) 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
Qff	**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)
Tt	**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, andesite, 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
Tfsh	**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_2O_5 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
Tgsb	**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 $^{40}Ar/^{39}Ar$ date of approximately 15.6 Ma for youngest flows of unit on Columbia Plateau
Tgww	**Winter Water unit (middle Miocene)** — Within map area, two flows are present, formerly designated as "-3 flow" of Beeson and others (1975) or "N_2 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)
Tgo	**Ortley unit (middle Miocene)** — Within map area, one or two flows are present, formerly designated as "N_2 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)

GEOLOGY MAP

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

0' 1,000' 2,000' 4,000'

SCALE: 1" = 2,000'

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HAWTHORNE BRIDGE HEAD

MULTNOMAH COUNTY
PORTLAND, OREGON

GEOLOGY
MAP

PROJECT: 15914.869
PHASE 0002

DATE: JANUARY 2015

FIGURE:
3

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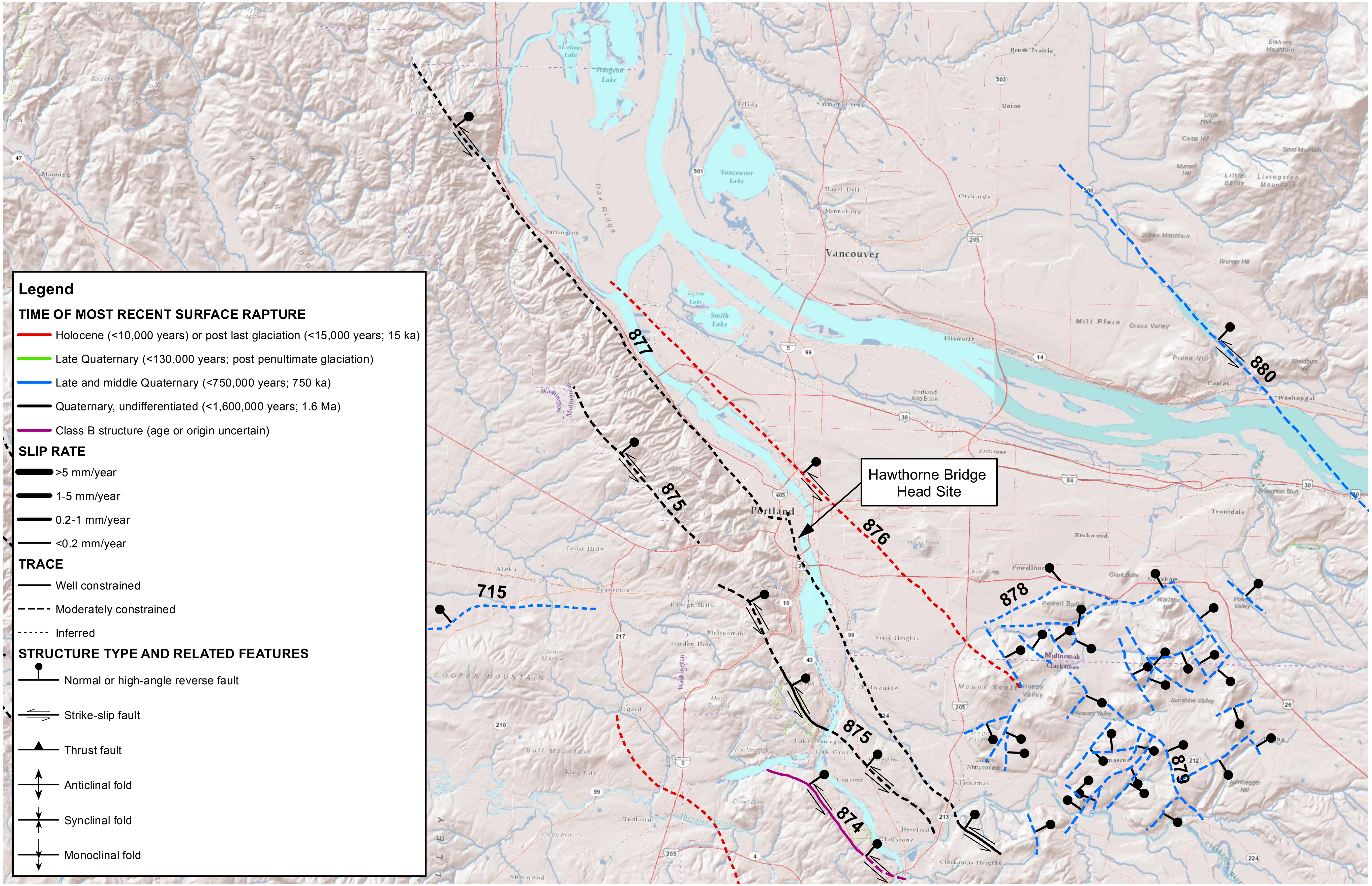

SOURCES: U.S. Geological Survey, 2006, Quaternary fault and fold database for the United States, accessed 01/14/2015, from USGS web site: http//earthquakes.usgs.gov/regional/qfaults/.

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LOCAL FAULTS

3 1.5 0 3 Miles

1 inch = 3 miles

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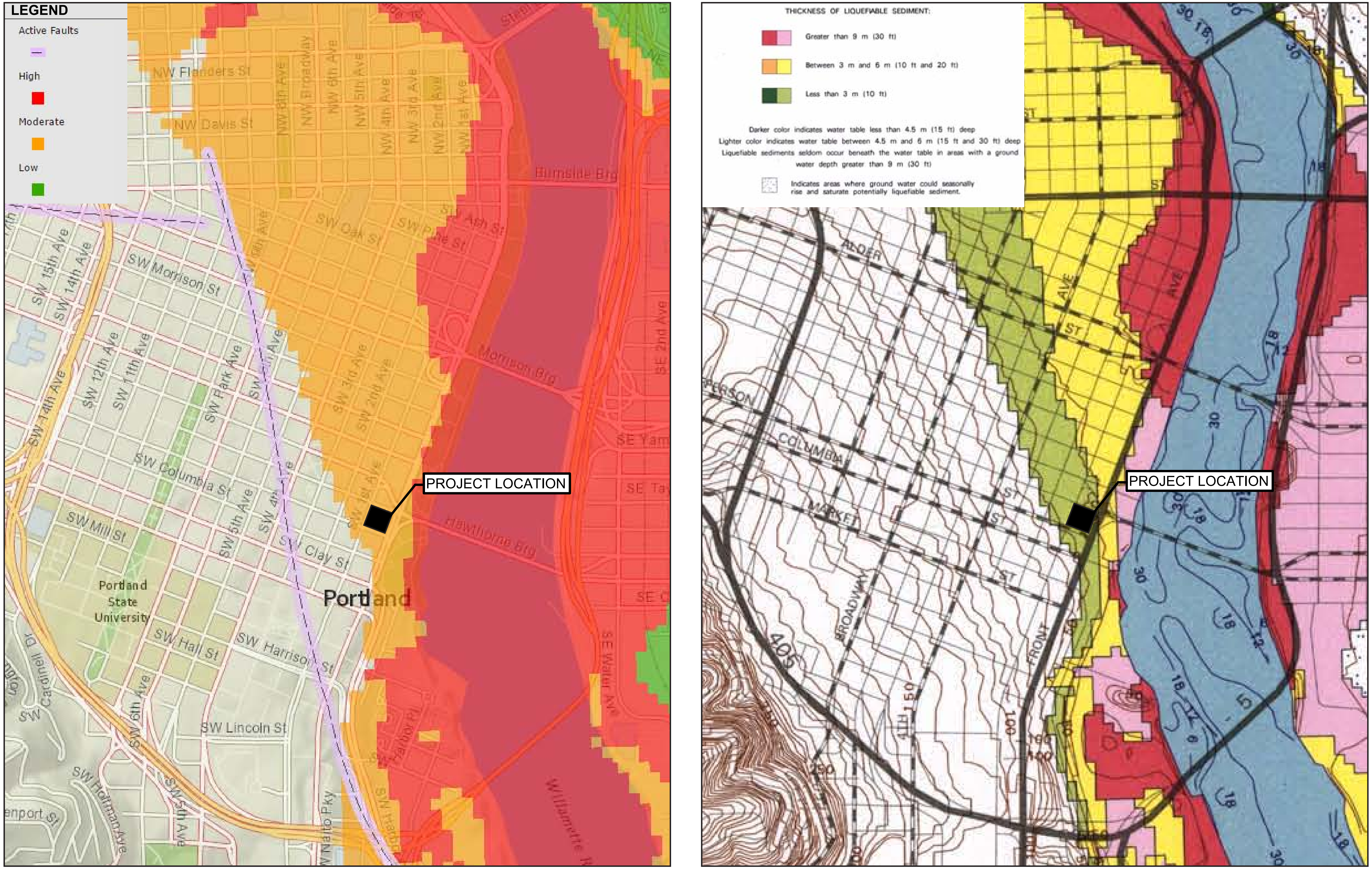
PORTLAND, OREGON

PROJECT	15914.869 PHASE 0002
DATE	JAN 2015
FIGURE	4



EARTHQUAKE LIQUEFACTION HAZARD

0' 500' 1,000' 2,000'
SCALE: 1" = 1,000'

SOURCES: DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES, OREGON HAZVU, STATEWIDE GEOHAZARDS VIEWER, WEBSITE: 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

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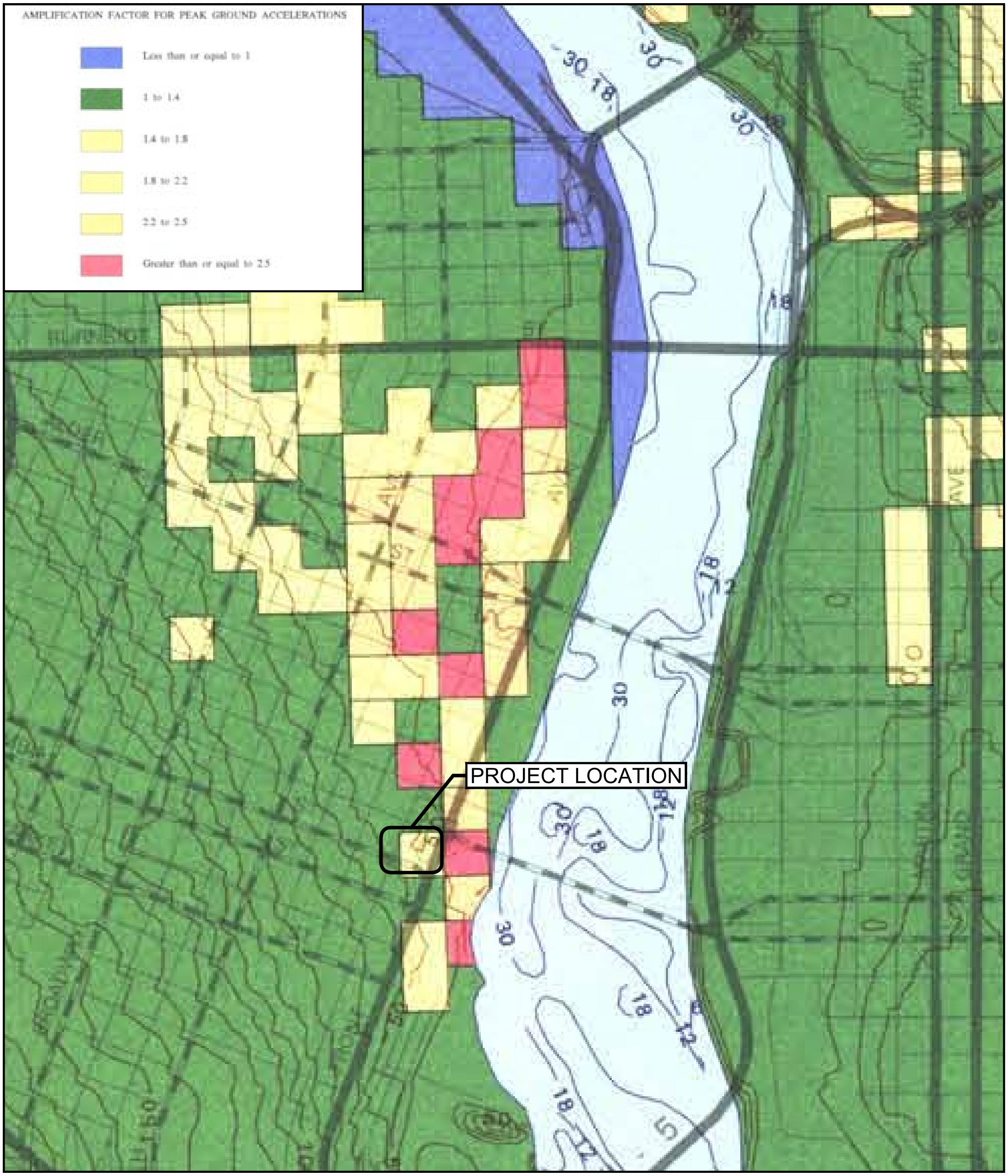
LIQUEFACTION
SEISMIC HAZARDS

PROJECT: 15914.869 PHASE 0002
DATE: JANUARY 2015
FIGURE: 5



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

0' 1,000' 2,000' 4,000'

SCALE: 1" = 2,000'

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PROJECT #
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DATE
JAN 2015

GROUND MOTION AMPLIFICATION

HAWTHORNE BRIDGE HEAD
PORTLAND, OREGON

FIGURE
6

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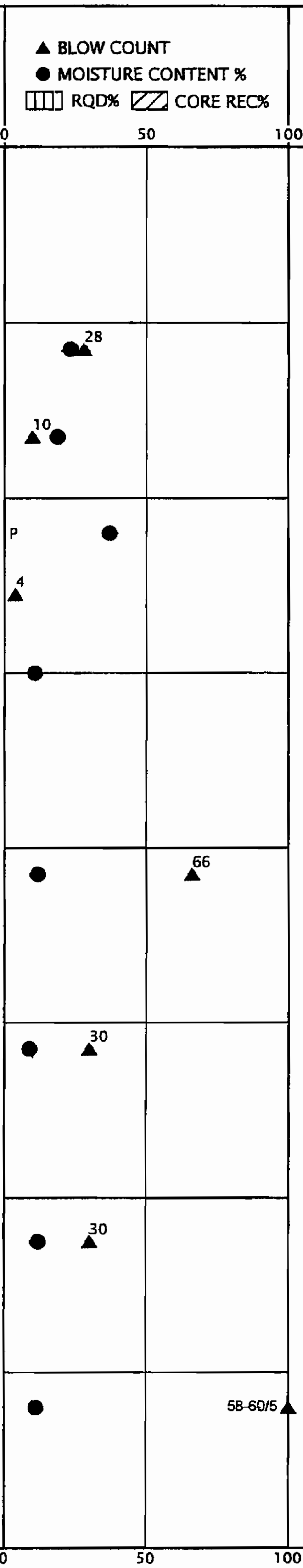
DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (4 inches thick)	0 3		
	GRAVEL base rock (8 inches thick)	1 0		
	Medium dense gray fine, sandy GRAVEL moist (fill)			
5				
	Loose light brown fine silty SAND with trace gravel moist	8 0	P200	P200 = 4%
10			DD DS	DD = 81 pcf
	Very dense gray-brown sandy GRAVEL with some silt, sand and cobbles, moist subrounded	14 3	P200	P200 = 7%
20			P200	P200 = 8%
25				
30	with trace silt at 30 0 feet		HYD SIEV	
35	Very dense gray brown GRAVEL moist subrounded	35 0		
40				



06/13/01 27 3 feet

DRILLED BY Geo-Tech Explorations Inc

LOGGED BY JGH

COMPLETED 06/12/01

BORING METHOD mud rotary (see report text)

BORING BIT DIAMETER 5 7/8-inch

GEODESIGN INC

15575 SW Sequoia Parkway Suite 100
Portland OR 97224
Off 503 968 8787 Fax 503 968 3068

EQUITYOFF 6 02

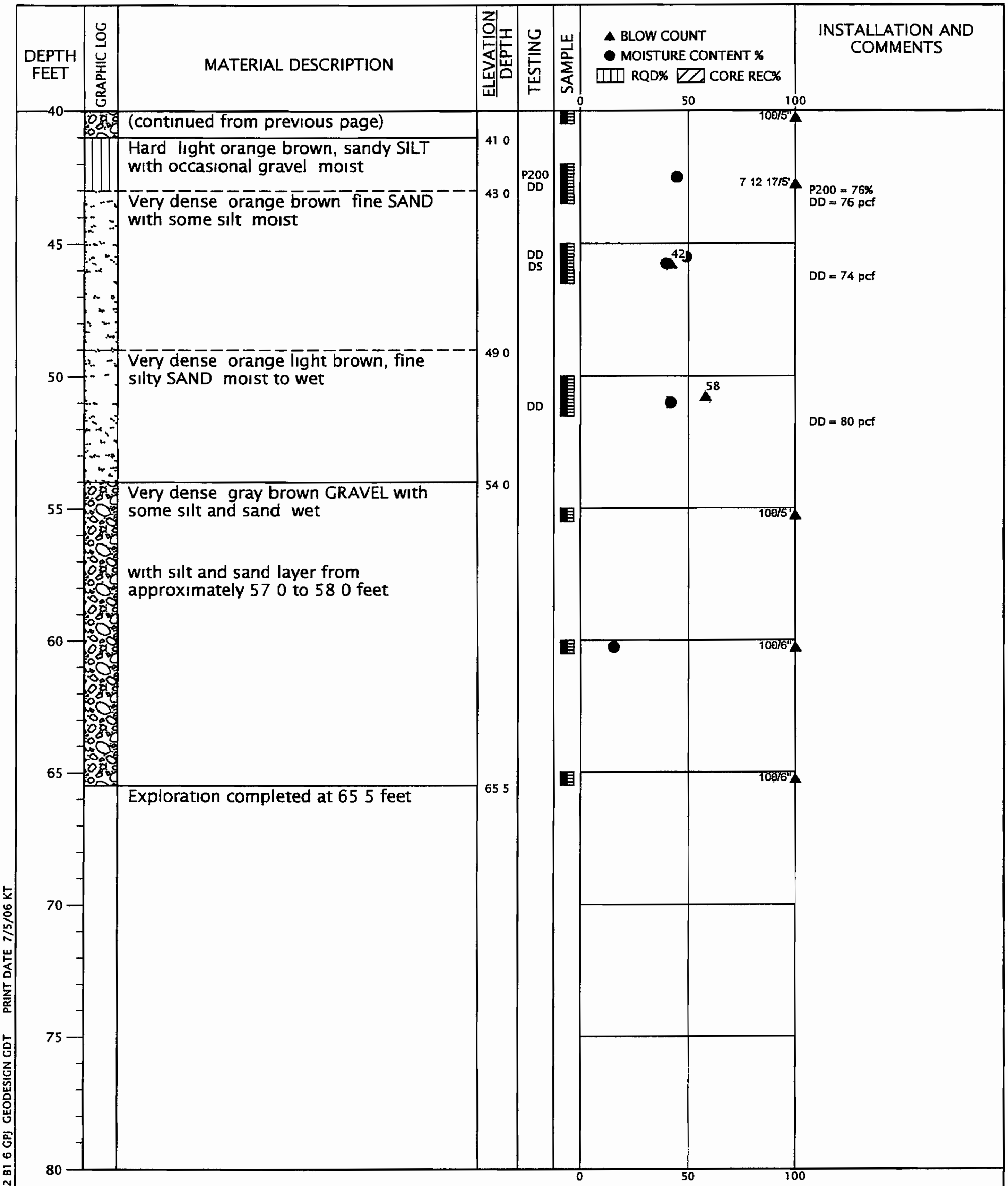
JULY 2006

BORING B 1

PROPOSED TWO MAIN PLACE OFFICE BUILDING
PORTLAND OR

FIGURE A 1

BORING LOG EQUITYOFF 6 02 B1 6 GPJ GEODESIGN GDT PRINT DATE 7/5/06 KT



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
40	(continued from previous page)			
	Hard light orange brown, sandy SILT with occasional gravel moist	41 0	P200 DD	P200 = 76% DD = 76 pcf
	Very dense orange brown fine SAND with some silt moist	43 0	DD DS	DD = 74 pcf
	Very dense orange light brown, fine silty SAND moist to wet	49 0	DD	DD = 80 pcf
	Very dense gray brown GRAVEL with some silt and sand wet with silt and sand layer from approximately 57 0 to 58 0 feet	54 0		
	Exploration completed at 65 5 feet	65 5		

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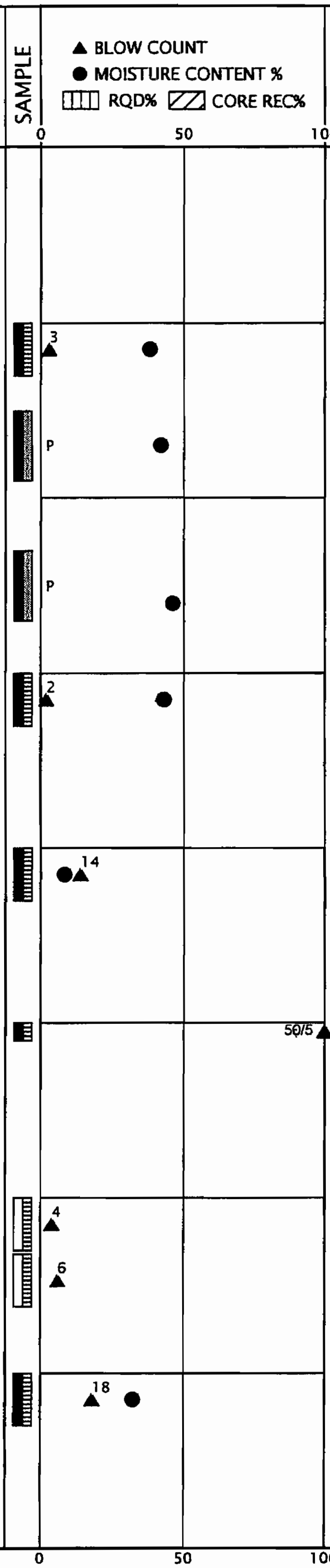
LOGGED BY JGH
COMPLETED 06/12/01

BORING METHOD mud rotary (see report text)
BORING BIT DIAMETER 5 7/8-inch

GEODESIGN INC 15575 SW Sequoia Parkway Suite 100 Portland OR 97224 Off 503 968 8787 Fax 503 968 3068	EQUITYOFF 6 02	**BORING B-1** (continued)	
	JULY 2006	PROPOSED TWO MAIN PLACE OFFICE BUILDING PORTLAND OR	**FIGURE A 1**

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (4 inches thick)	0 3		
	GRAVEL base rock (8 inches thick)	1 0		
	Soft red-brown SILT with some fine sand and trace gravel, moist (fill)			
5				
			DD	DD = 78 pcf
10				Lost approximately 600 gallons of drilling mud to formation prior to reaching a depth of 22 0 feet.
			DD DS	DD = 74 pcf
	Very loose orange light brown silty SAND moist to wet	14 0		
15				
	Medium dense gray light brown GRAVEL with some sand and trace silt moist	17 0		
20				06/15/01 21 8 feet
25	becomes very dense at 25 0 feet			
30	Medium dense gray brown fine SAND with trace silt wet	30 0		
35	grades to medium dense at 35 0 feet		P200	P200 = 20%
	Very dense gray-brown GRAVEL with some sand wet	37 0		
40				



DRILLED BY Geo-Tech Explorations Inc

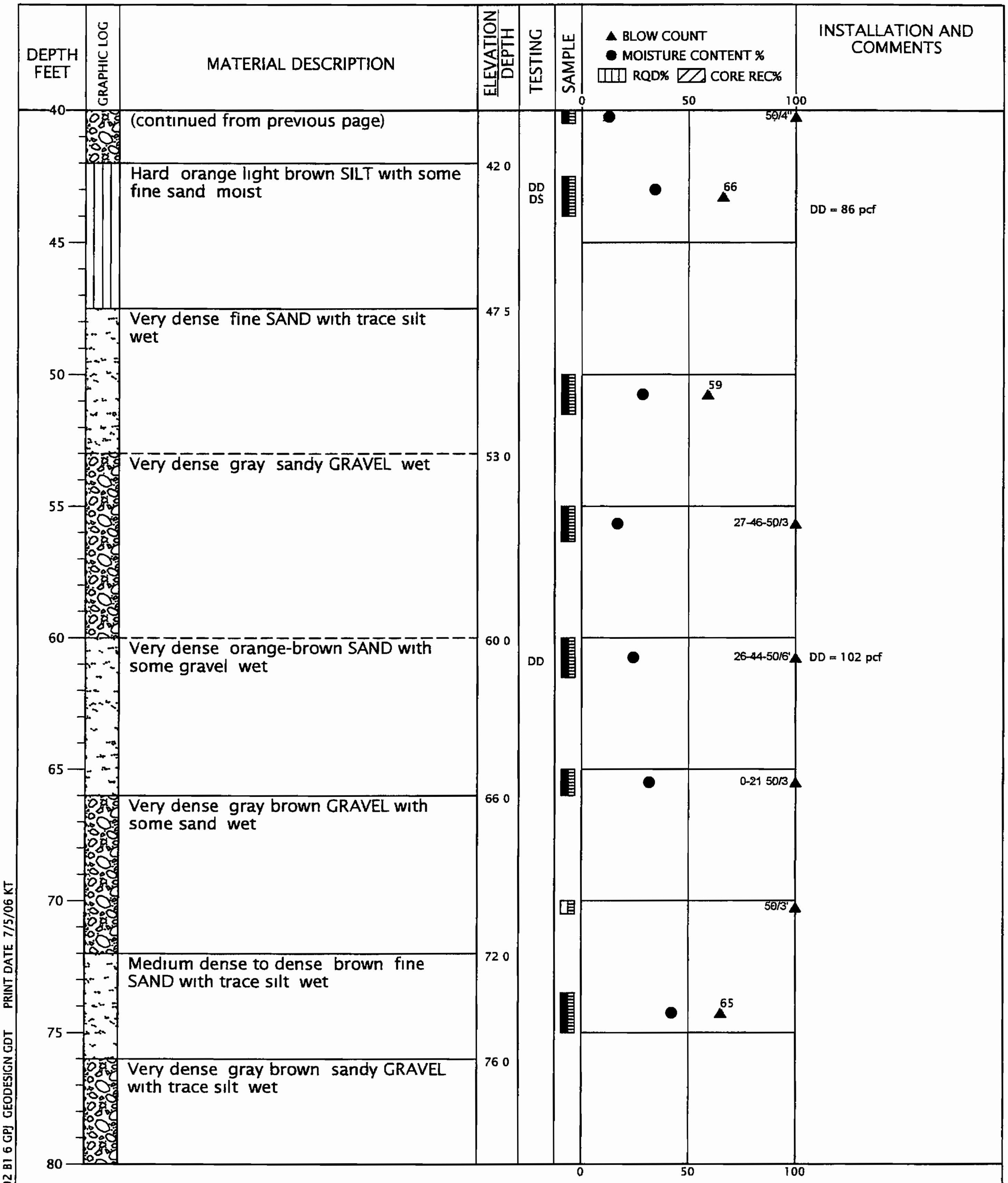
LOGGED BY JGH

COMPLETED 06/14/01

BORING METHOD mud rotary (see report text)

BORING BIT DIAMETER 5 7/8-inch

GEODESIGN INC 15575 SW Sequoia Parkway Suite 100 Portland OR 97224 Off 503 968 8787 Fax 503 968 3068	EQUITYOFF 6 02	**BORING B-2**
	JULY 2006	PROPOSED TWO MAIN PLACE OFFICE BUILDING PORTLAND OR
		FIGURE A 2



MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
(continued from previous page)			50/4"	
Hard orange light brown SILT with some fine sand moist	42 0	DD DS	66	DD = 86 pcf
Very dense fine SAND with trace silt wet	47 5		59	
Very dense gray sandy GRAVEL wet	53 0		27-46-50/3	
Very dense orange-brown SAND with some gravel wet	60 0	DD	26-44-50/6' 0-21 50/3	DD = 102 pcf
Very dense gray brown GRAVEL with some sand wet	66 0		50/3'	
Medium dense to dense brown fine SAND with trace silt wet	72 0		65	
Very dense gray brown sandy GRAVEL with trace silt wet	76 0			

DRILLED BY Geo-Tech Explorations Inc

LOGGED BY JGH

COMPLETED 06/14/01

BORING METHOD mud rotary (see report text)

BORING BIT DIAMETER 5 7/8-inch

BORING LOG EQUITYOFF 6 02 B1 6 GPJ GEODESIGN GDT PRINT DATE 7/5/06 KT

GEODESIGN INC

15575 SW Sequoia Parkway Suite 100
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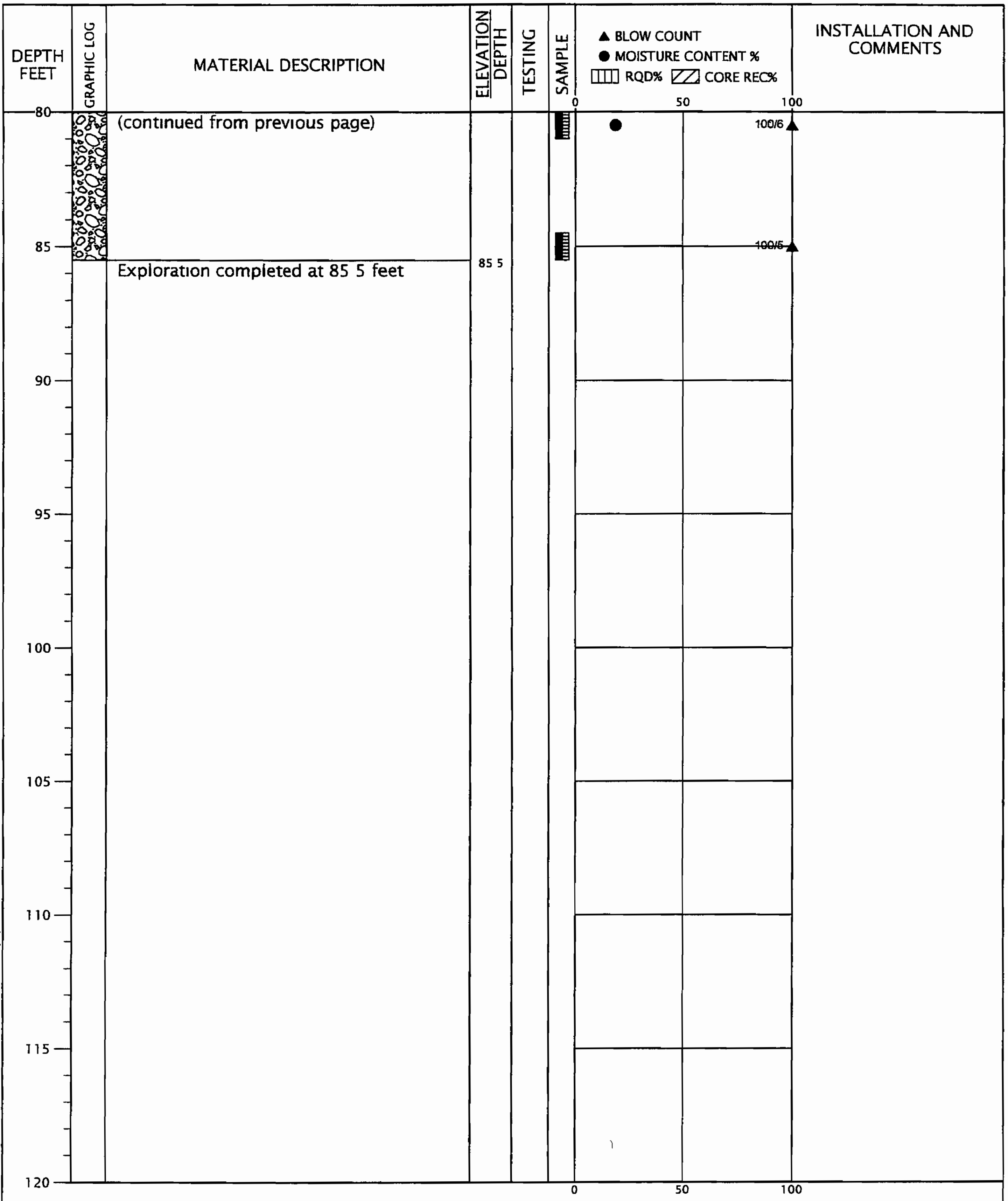
EQUITYOFF 6 02

JULY 2006

BORING B-2
(continued)

PROPOSED TWO MAIN PLACE OFFICE BUILDING
PORTLAND OR

FIGURE A 2



(continued from previous page)

Exploration completed at 85 5 feet

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BORING METHOD mud rotary (see report text)

BORING BIT DIAMETER. 5 7/8-inch

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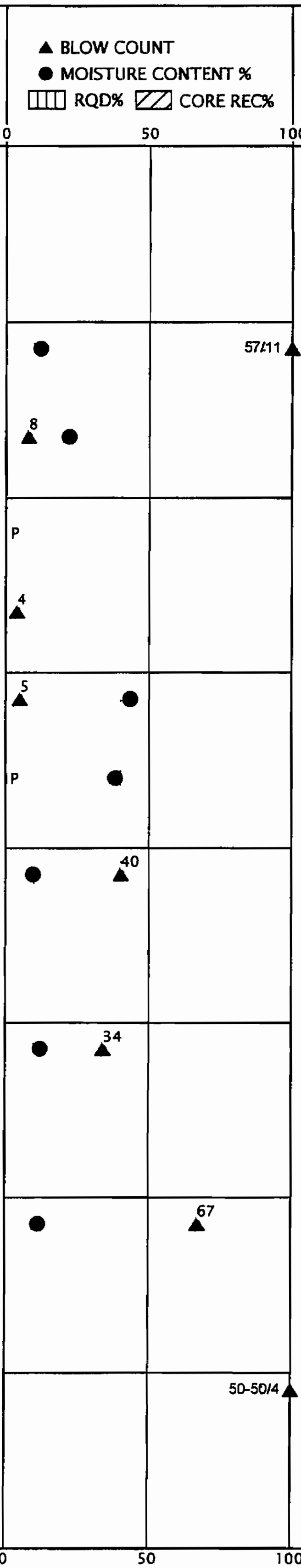
BORING B 2
(continued)

PROPOSED TWO MAIN PLACE OFFICE BUILDING
PORTLAND OR

FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (4 inches thick)	0 3		
	GRAVEL base rock (8 inches thick)	1 0		
	Very dense black gray silty GRAVEL with some sand, moist (fill)		P200	P200 = 12%
	grades to loose and sandy with some silt at 8 0 feet			
10	Medium stiff light brown SILT with some fine sand moist (fill)	10 5		
	Loose orange brown, silty SAND with trace gravel moist	12 0	P200	P200 = 77%
15	Medium dense orange brown fine SAND with some silt moist to wet	16 0		
20	Dense gray brown sandy GRAVEL with some silt moist	20 0	P200	P200 = 8%
30	grades to very dense orange, gray, and brown, wet at 30 0 feet			
	Very dense orange-gray and brown GRAVEL with some sand and trace silt wet	33 5		
40				



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BORING B 3

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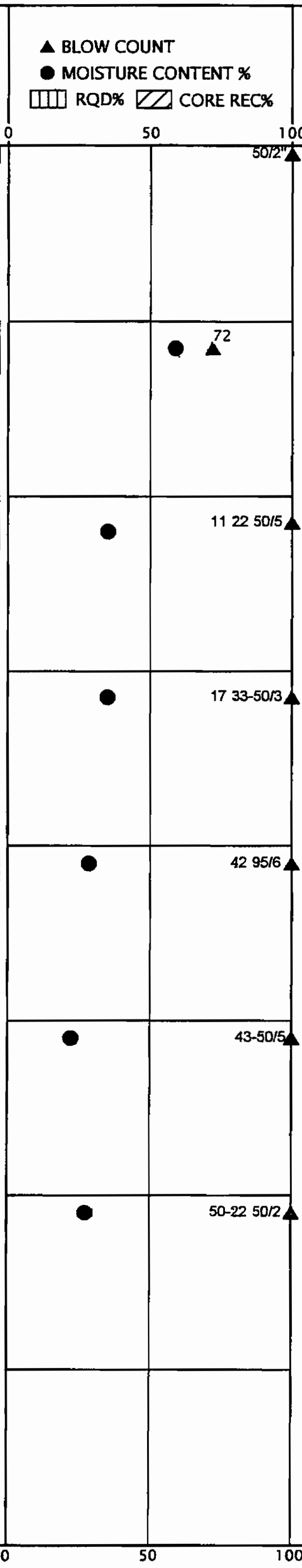
PROPOSED TWO MAIN PLACE OFFICE BUILDING
PORTLAND OR

FIGURE A 3

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
40	(continued from previous page)			50/2"	
	Hard gray SILT moist	44 0			
45			CON	72	
	Very dense orange brown SAND with trace silt wet	47 0			
50			DD DS	11 22 50/5	DD = 87 pcf
55				17 33-50/3	
	gravel layer from approximately 57 0 to 59 5 feet				
60	Very dense gray brown SAND with trace gravel wet	60 0	DD	42 95/6	DD = 95 pcf
65	becomes orange-brown with some gravel at 65 0 feet			43-50/5	
	Very dense orange brown fine, silty SAND wet	69 0			
70				50-22 50/2	
	Exploration completed at 71 0 feet	71 0			
75					
80					



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BORING BIT DIAMETER 5 7/8-inch

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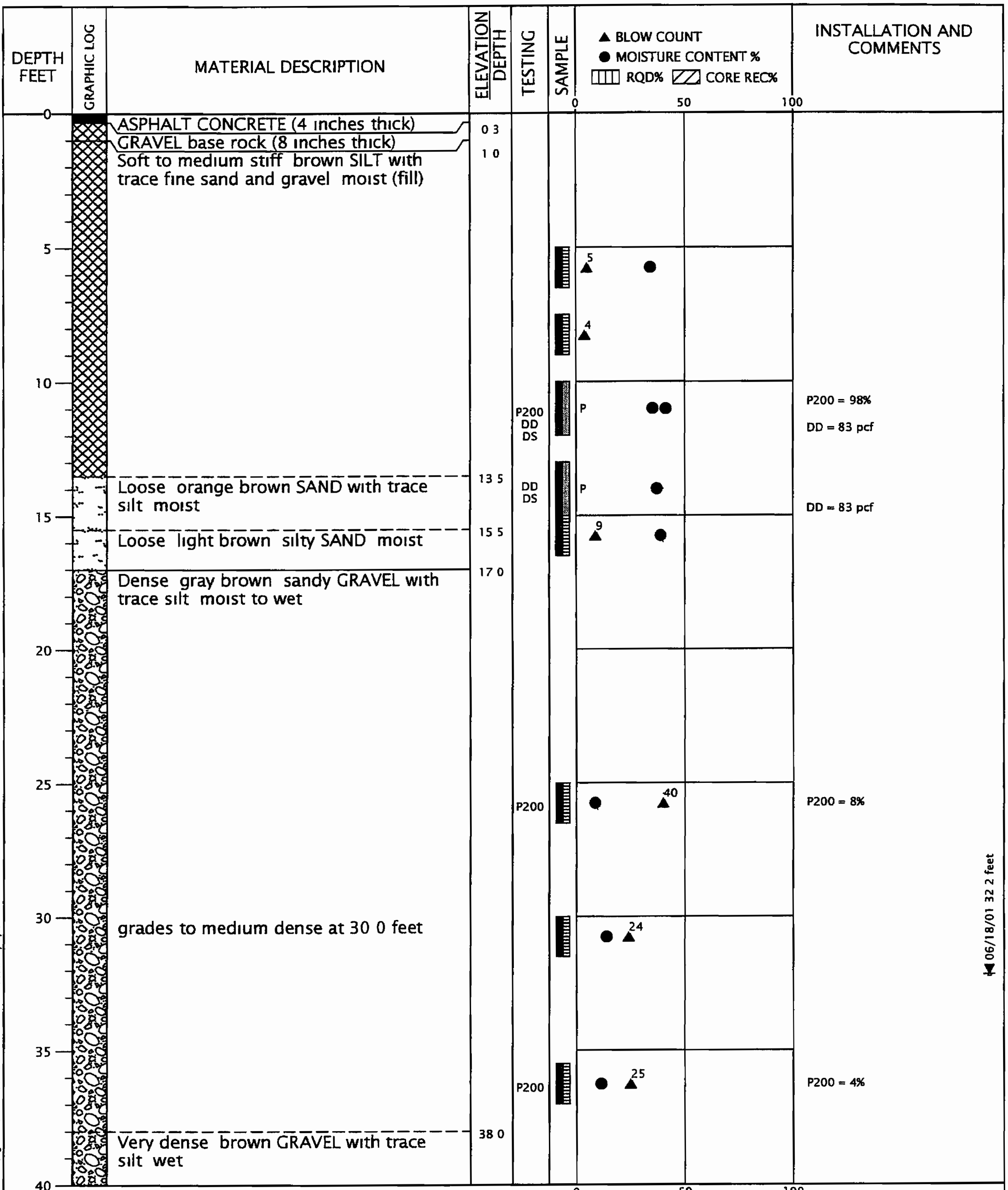
BORING B 3
(continued)

PROPOSED TWO MAIN PLACE OFFICE BUILDING
PORTLAND OR

FIGURE A 3

BORING LOG EQUITYOFF 6 02 B1 6 GPJ GEODESIGN GDT PRINT DATE 7/5/06 KT



ELEVATION DEPTH	MATERIAL DESCRIPTION	TESTING	INSTALLATION AND COMMENTS
0 3	ASPHALT CONCRETE (4 inches thick)		
1 0	GRAVEL base rock (8 inches thick)		
	Soft to medium stiff brown SILT with trace fine sand and gravel moist (fill)	P200 DD DS	P200 = 98% DD = 83 pcf
13 5	Loose orange brown SAND with trace silt moist	DD DS	DD = 83 pcf
15 5	Loose light brown silty SAND moist		
17 0	Dense gray brown sandy GRAVEL with trace silt moist to wet	P200	P200 = 8%
	grades to medium dense at 30 0 feet	P200	P200 = 4%
38 0	Very dense brown GRAVEL with trace silt wet		

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BORING METHOD mud rotary (see report text)
BORING BIT DIAMETER 5 7/8-inch

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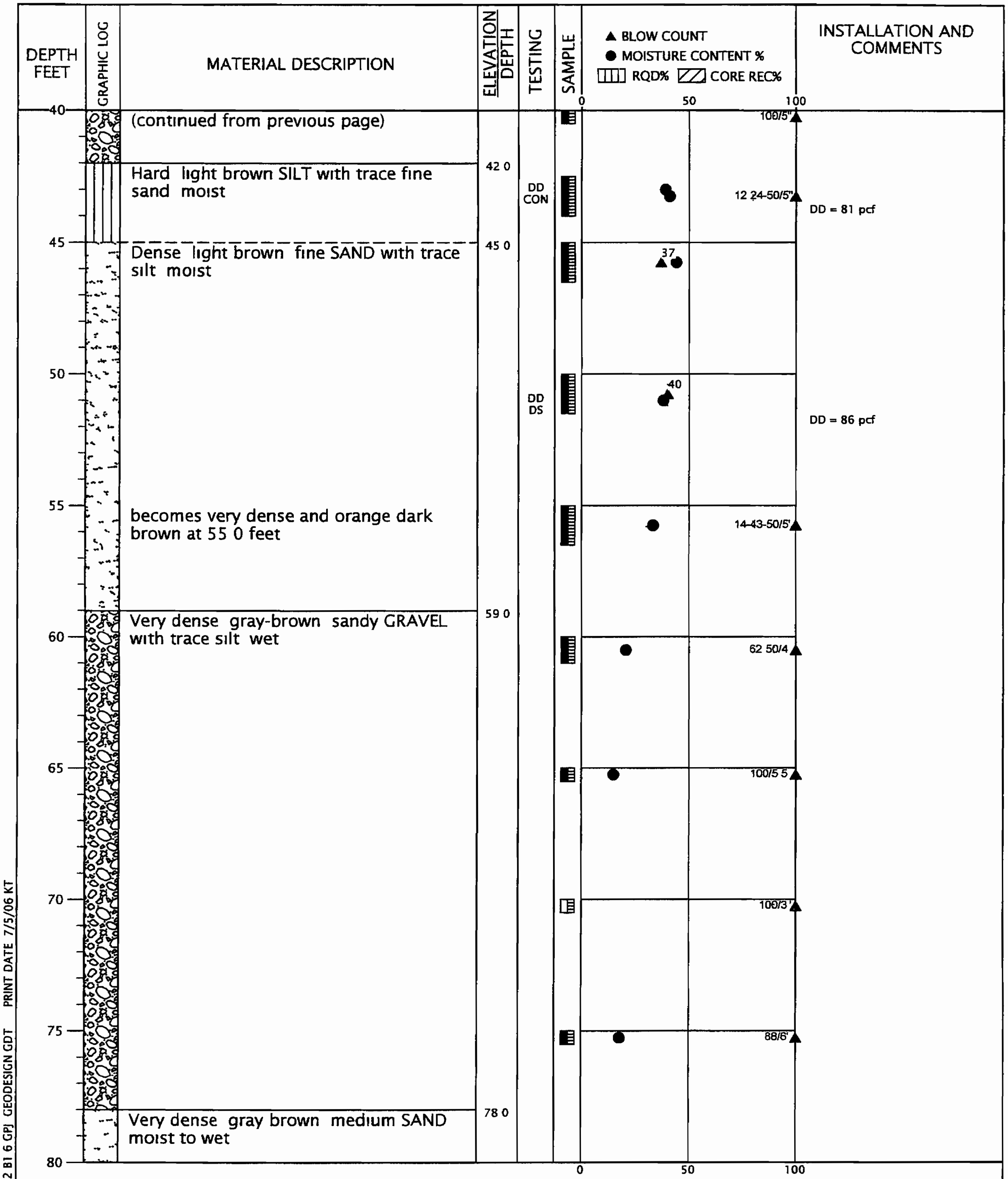
BORING B 4

PROPOSED TWO MAIN PLACE OFFICE BUILDING
PORTLAND OR

FIGURE A 4



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
40	(continued from previous page)			100/5"	
	Hard light brown SILT with trace fine sand moist	42 0	DD CON	12 24-50/5"	DD = 81 pcf
45	Dense light brown fine SAND with trace silt moist	45 0		37	
50			DD DS	40	DD = 86 pcf
55	becomes very dense and orange dark brown at 55 0 feet			14-43-50/5"	
	Very dense gray-brown sandy GRAVEL with trace silt wet	59 0			
60				62 50/4	
65				100/5 5	
70				100/3"	
75				88/6"	
	Very dense gray brown medium SAND moist to wet	78 0			
80					

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BORING BIT DIAMETER 5 7/8-inch

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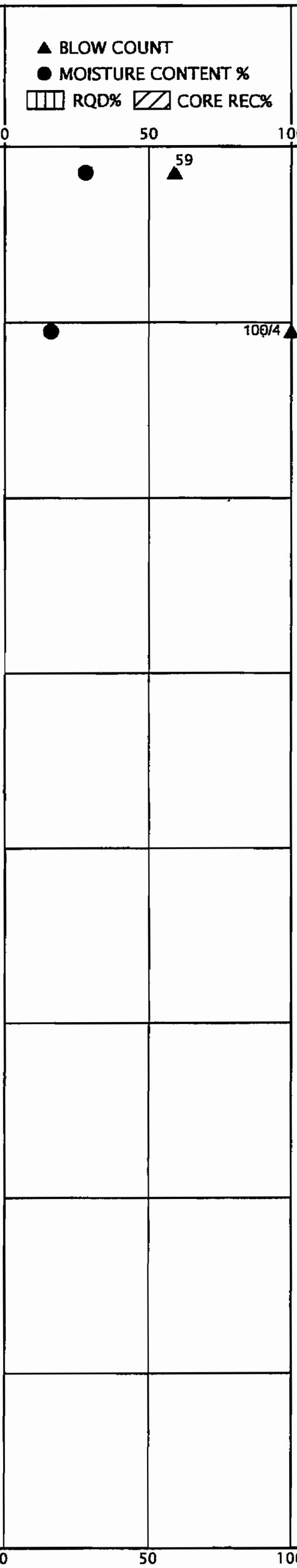
BORING B 4
(continued)

PROPOSED TWO MAIN PLACE OFFICE BUILDING
PORTLAND OR

FIGURE A 4

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	INSTALLATION AND COMMENTS
80		Very dense dark gray brown SAND wet	80 0	DD		DD = 85 pcf
		Very dense, gray-brown sandy GRAVEL with trace silt wet	82 0			
85		Exploration completed at 85 3 feet	85 3			
90						
95						
100						
105						
110						
115						
120						



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BORING BIT DIAMETER 5 7/8-inch

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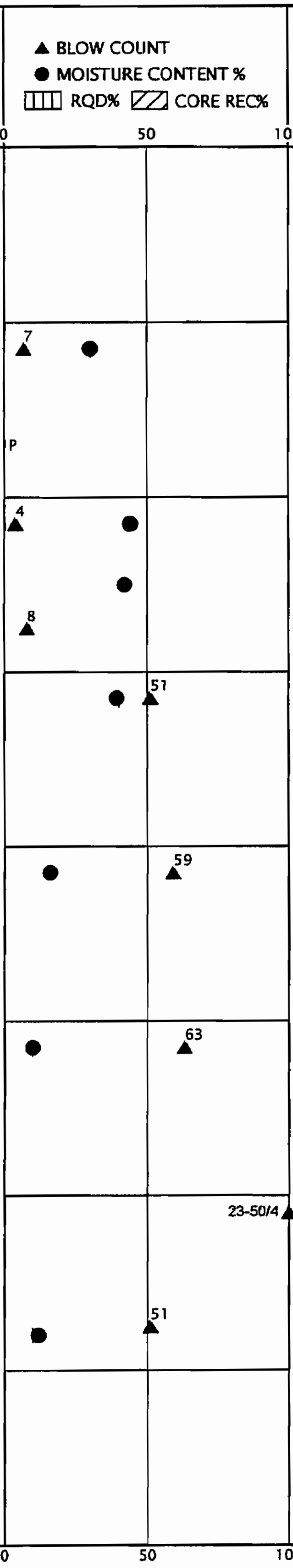
EQUITYOFF 6 02	**BORING B 4** (continued)	
JULY 2006	PROPOSED TWO MAIN PLACE OFFICE BUILDING PORTLAND OR	**FIGURE A 4**

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (4 inches thick)	0 3		
	GRAVEL base rock (8 inches thick)	1 0		
	Medium stiff gray brown SILT moist (fill)			
5			DD DS	DD = 87 pcf
	becomes light brown at 7 5 feet			
10	Soft brown SILT with trace fine sand moist	10 0	DD	DD = 79 pcf
	Loose brown fine to medium SAND with some silt moist	12 0	DD DS	DD = 71 pcf
			DS	
15	Very dense, gray brown GRAVEL with some silt moist subrounded	15 5	P200	P200 = 82%
20			P200	P200 = 29%
	with trace silt and sand at 23 0 feet			
25				
	Very dense brown GRAVEL with some silt moist	29 0		
30				
	Very dense, gravelly SAND moist subrounded gravel	32 5	DD	DD = 128 pcf
35	Very dense brown GRAVEL with trace silt and sand moist subrounded	35 5		
40				



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BORING METHOD hollow stem auger (see report text) BORING BIT DIAMETER 5 7/8-inch

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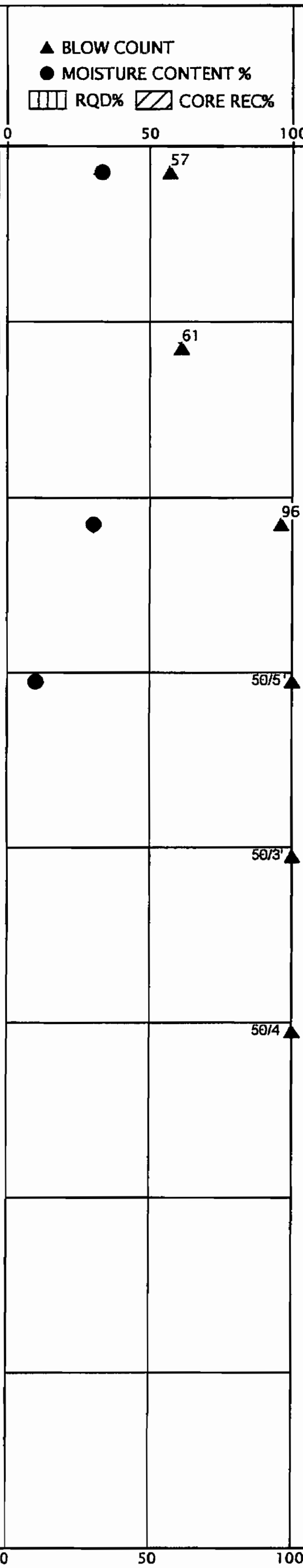
	JULY 2006	PROPOSED TWO MAIN PLACE OFFICE BUILDING PORTLAND OR
		FIGURE A 5

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	INSTALLATION AND COMMENTS
40		Hard orange-brown SILT with trace fine sand moist with gravel layer from 41 5 to 44 5 feet	40 0			
45		Very dense brown fine SAND with some silt moist	44 5			
50		becomes fine to medium at 50 0 feet				
55		becomes medium to coarse with trace silt wet at 55 0 feet		DD		DD = 127 pcf
60		grades to coarse with some gravel and silt at 60 0 feet				
65		Exploration completed at 65 5 feet	65 5			
70						
75						
80						



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BORING METHOD hollow stem auger (see report text)

BORING BIT DIAMETER 5 7/8-inch

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BORING B 5
(continued)

PROPOSED TWO MAIN PLACE OFFICE BUILDING
PORTLAND OR

FIGURE A 5

▲ BLOW COUNT
● MOISTURE CONTENT %
▥ RQD% ▨ CORE REC%
0 50 100

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (2 inches thick)	0 2		
	BASE ROCK with some silt (12 inches thick)	1 2		
	Medium dense brown fine to coarse GRAVEL with some silt and sand moist rounded to angular (fill)	2 0		
5	Medium stiff gray SILT with brown mottles and trace clay, moist with trace fine rootlets from 4 5 to 5 0 feet grades to gray at 7 5 feet			
10	grades with some fine sand without clay at 11 0 feet			
15	Medium dense fine brown SAND with trace silt moist	12 5		
	Medium stiff brown SILT with some fine sand moist to wet low plasticity grades to trace fine sand at 17 0 feet	16 0		
20	Dense brown GRAVEL with some silt and trace sand and cobbles moist subrounded grades to silty from 20 0 to 21 0 feet grades to very dense at 21 0 feet and encountered cobbles from 21 0 to 22 0 feet grades to trace silt at 22 5 feet	19 0		
25	grades to some silt at 24 5 feet grades to some cobbles from 25 0 to 27 0 feet grades to trace silt at 27 0 feet sandy from 27 0 to 27 5 feet			
30				
35	layer of stiff silt (6 inches thick) with some gravel and trace fine sand at 34 5 feet grades with weak cementation and some silt at 36 0 feet grades to no cementation and some silt at 38 0 feet			⊻ 40 0 feet during drilling
40				

0 50 100

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BORING METHOD sonic drilling (see report text)
BORING BIT DIAMETER 8-inch

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BORING B 6

PROPOSED TWO MAIN PLACE OFFICE BUILDING
PORTLAND OR

FIGURE A 6

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
40	grades to weak to moderate cementation at 40 0 feet	41 0	
45	Dense brown orange silty SAND moist grades to gray-dark gray at 43 5 feet grades to brown with trace to some sand and weak to moderate cementation zones, moist to wet at 47 0 feet	48 5	
50	Dense brown fine to medium SAND with trace silt, moist to wet, weak to moderate cementation zones grades to sandy and gravelly wet at 50 0 feet	54 0	
55	Dense brown gray sandy GRAVEL with some sand wet rounded to subrounded	56 0	
	Medium dense brown SAND with some gravel moist to wet sand lense sand lense at approximately 58 0 feet	60 0	Driller comments hard drilling last 2 feet getting tight
60	Medium dense to dense brown gray GRAVEL with some sand and trace silt wet poorly graded rounded	62 5	
	Medium dense brown fine silty SAND moist	64 0	
65	Dense brown gray GRAVEL with some sand and trace silt and cobbles wet grades to sandy moist to wet at 65 0 feet	67 0	
70	Medium dense to dense brown gravelly SAND with trace silt moist to wet grades to gray with some sand at 69 0 feet grades to brown and sandy at 70 5 feet grades to gray with some sand at 71 5 feet grades to brown and sandy from 72 0 to 72 5 feet grades to gray with some sand at 72 5 feet brown sandy zone from 74 0 to 74 5 feet	75 0	
75	Medium stiff to stiff light gray CLAY with some silt, moist (ash)	77 0	
	Medium stiff brown, sandy SILT with trace gravel, wet	77 5	
80			

TESTING | SAMPLE | ▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% — 0 50 100

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BORING B 6
(continued)

PROPOSED TWO MAIN PLACE OFFICE BUILDING
PORTLAND OR

FIGURE A 6

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC%	INSTALLATION AND COMMENTS
80	Dense brown gray GRAVEL with some sand, silt, and cobbles wet rounded to subrounded grades to trace silt at 78 5 feet	80 0				
		81 0				
	Dense brown gray GRAVEL with some sand and trace silt and cobbles, wet	82 5				
85	Medium dense brown SAND with trace silt and gravel, moist					
	Dense brown GRAVEL with some sand and cobbles and trace silt moist to wet rounded to subrounded grades to brown gray with cobbles (up to 4 5 inch diameter) at 85 5 feet					
90						
95	grades to gray with weak cementation at 92 5 feet grades with some light gray coarse gravel with cementation on sides at 93 5 feet	95 0				
	Dense to very dense gray dark gray GRAVEL with some silt sand and cobbles moist rounded to subangular weak cementation in clay matrix					
100	Dense gray brown GRAVEL with some sand trace silt, and trace to some cobbles wet rounded to subrounded no cementation observed grades to sandy at 103 5 feet	100 0				
105						
110	Medium dense to dense gray fine to medium SAND with trace silt and trace to some gravel moist to wet	109 5				
	Dense gray GRAVEL with some sand and cobbles and trace silt moist to wet rounded to subrounded grades to sandy at 113 5 feet	111 3				
115	Medium dense gray gravelly SAND with some cobbles wet	115 0				
	Dense gray GRAVEL with some sand and cobbles and trace silt wet grades to poorly graded at 118 0 feet with sand lense (6 inches thick) at 119 0	116 5				
120					0 50 100	

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BORING METHOD sonic drilling (see report text)

BORING BIT DIAMETER 8-inch

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	JULY 2006	PROPOSED TWO MAIN PLACE OFFICE BUILDING PORTLAND OR	**FIGURE A 6**

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC% 0 50 100	INSTALLATION AND COMMENTS
120	feet grades with occasional boulders and well graded at 120 0 feet			☒		Driller comments caving at 121 5 feet End of day 4/28/06 at 1655 Start drilling 4/29/06 at 7 00 AM Advance casing water at 40 0 feet.
	Dense, gray GRAVEL with trace silt and some sand and cobbles moist to wet rounded to subrounded slight cementation on individual gravel particles grades to some silt at 125 0 feet	123 0		☒		
125						
	Dense gray GRAVEL with some sand and trace to some silt and cobbles moist occasional weak cementation on individual gravel particles	126 0		☒		
130				☒		Driller comments caving after advancing sample not an open hole due to water from 40 0 to 50 0 feet
135				☒		
	grades to wet at 139 0 feet			☒		
140						
	grades to moist at 142 0 feet			☒		
	Dense gray GRAVEL with some sand and trace to some cobbles and silt, moist	143 0		☒		
145				☒		
150	Exploration completed at 150 0 feet	150 0		☒		Advance casing to 150 0 feet.
155						
160					0 50 100	

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BORING B 6
(continued)

PROPOSED TWO MAIN PLACE OFFICE BUILDING
PORTLAND OR

FIGURE A-6