PBS | Engineering + Environmental

March 26, 2015

Multnomah County Facilities Management
Attn: Mr. Mike McBride
401 N Dixon Street
Portland, Oregon

Via Email: michael.mcbride@multco.us
cc: JD.Deschamps@multco.us

Re: Additional Geotechnical Engineering Services – Feasibility Assessment
Due Diligence Services – Multnomah County Courthouse
Block 128, Portland, Oregon
PBS Project No. 15194.869 Task 004

INTRODUCTION AND BACKGROUND

PBS Engineering and Environmental Inc. (PBS) is pleased to provide this supplemental feasibility/due diligence letter report for geotechnical engineering services in support of site selection for the Multnomah County Courthouse in Portland, Oregon (refer, Figure 1, Vicinity Map). Block 128 (site) is being considered as the site of a proposed new courthouse building. The 1.1-acre site 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, respectively (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). As part of the Multnomah County (County) due diligence efforts, PBS previously completed a Geotechnical Engineering Feasibility Assessment[1] for the site.

An existing structure on the southwestern portion of the site includes a one-story, covered, concrete entrance to underground parking and loading docks to 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™ imagery. Based on our conversations with County personnel and our experience with similar projects, the development will include the following.

- A single, 14- to 17-story, steel-frame 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

[1] PBS Engineering and Environmental (21 January 2015). *Geotechnical Engineering Feasibility Assessment Tasks 1 and 2, Due Diligence Services – Multnomah County Courthouse, Block 128, Portland Oregon.* Prepared for Mr. Mike McBride, Multnomah County Facilities Management. PBS Project No. 15194.869

The County has requested that PBS identify potential geotechnical issues that could affect the proposed plan. The purpose of our additional geotechnical engineering services was to complete limited site-specific explorations in order to better evaluate the risk of liquefaction, and to develop conceptual foundation recommendations for use in planning. Subsurface explorations were included as part of this additional phase of geotechnical engineering services. The project stakeholders, including the County, will utilize the information in completing their due diligence review.

PURPOSE AND SCOPE OF SERVICES

The limited site-specific subsurface explorations were completed to characterize probable geotechnical conditions and collect field test data that can be used to better evaluate potential for liquefaction and lateral spreading, and to develop a better understanding of possible foundation support.

1. **Subsurface Exploration:** Explored subsurface conditions at the site by completing two cone penetration test probes (CPT-1 and 2) to depths of 43 and 47 feet below the existing ground surface (bgs), respectively. The CPT probe locations are shown on the attached Figure 2. Seismic shear wave velocities were collected at one meter intervals in CPT-1. Pore-pressure dissipation testing to evaluate the presence and depth of groundwater was completed in both CPTs.
2. **Geotechnical Engineering Studies:** The data collected during the subsurface exploration and previous literature research was analyzed and used to develop an opinion regarding the geotechnical feasibility of the proposed site development.
3. **Deliverable:** This supplemental geotechnical feasibility assessment report was prepared containing the results of our services, including the following information.
   - Explorations logs and approximate exploration locations
   - Groundwater considerations
   - Seismic design criteria in accordance with the 2014 Oregon Structural Specialty Code (OSSC) and discussion regarding the need for additional study (if required)
   - Results of liquefaction and lateral spreading analyses
   - Excavation and shoring considerations
   - Discussion regarding foundation types and design considerations

SUBSURFACE EXPLORATION AND CONDITIONS

Explorations were completed at the site by Oregon Geotechnical Explorations, Inc. and included two cone penetration test probes (designated as CPT-1 and CPT-2) to depths of 43 and 47 feet bgs. Interpretation of the CPT probes suggests that the area is underlain by variable subsurface conditions. Based on the CPT data, CPT-1 and CPT-2 encountered similar materials, including medium stiff to stiff silt and clay with medium dense sand lenses. CPT-2 encountered predominately the same material with medium dense to dense sand lenses at varying depths. Both explorations were terminated in very dense gravel and cobbles. Subsurface conditions in the CPTs are similar to those logged by Geocon in their 2001 borings which encountered stiff to very stiff silt with sand interbeds and lenses of sand. The borings were terminated in gravel at depths of about 44 and 45 feet.

Field Procedures

Before the start of cone penetration testing, the truck is jacked up and leveled on four pads to provide a stable reaction for the cone thrust. During the test, the instrumented cone is hydraulically pushed into the ground at about 2 centimeters per second (cm/s), and readings of cone tip resistance, sleeve friction and pore pressure, are digitally recorded every second. As the cone advances, additional cone rods are added such that a “string” of rods continuously advances through the soil. As the test progresses, the CPT operator monitors the cone resistance and its deviation from vertical alignment. No samples of the soil are obtained for visual examination or other laboratory testing.

For CPT probing in which seismic data was collected, conventional CPT testing is temporarily halted at 2-meter intervals to collect seismic data. An accelerometer is integrated with the CPT cone, which is used to record the arrival time of seismic waves generated by striking a steel beam positioned on the ground surface at least 10 feet from the cone rods. It is coupled to the ground surface by the weight of the beam and operator to prevent the beam from moving when struck; imparting a shear wave down through the soil profile.

Each side of the beam is struck several times and each signal produced by a blow is closely examined for signal and noise content after which, the waveform is selected and the arrival time of the shear wave is picked and recorded. After a complete set of seismic data is recorded, the cone is advanced to the next test depth and the procedure is repeated until the probe is completed.

CPT Logs

In accordance with ASTM D 5778, recorded values include tip and shaft resistance and pore pressure. This information is used to calculate the friction ratio, which is the ratio of the resistance to advance a sliding sleeve to the resistance of advancing the leading cone. The cone resistance and friction ratio are correlated to indicate material types which are presented graphically in a column to the right. Soil type is determined as a function of the tip resistance and friction. Sand typically has higher tip resistances and lower friction ratios, while fine-grained soils exhibit proportionately lower tip resistances in relation to friction ratios. Also, this data can provide indications of material strengths and deformation properties. The CPT logs are attached as Figures A1 and A2. Shear wave velocities measured in CPT-1 are included in Figure A3.

GROUNDWATER

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

Groundwater cannot be directly measured in a CPT. However, pore pressure dissipation testing, which can be used to estimate the depth of groundwater, was completed at a depth of 47 feet bgs in CPT-2 , indicating groundwater was present at a depth of about 45 feet bgs. This is consistent with the mapped groundwater contours developed by the U.S. Geological Survey (USGS) for the Portland area. We anticipate groundwater levels could fluctuate throughout the year.

CONCLUSIONS AND RECOMMENDATIONS

Geotechnical Design and Considerations

Based on our conversations with County personnel and experience with similar projects, as well as the development assumptions as stated in the Introduction and Background sections of this report, our current opinion is that support of the proposed new building on shallow spread footings is not feasible because of the thick deposit of relatively soft silt and clay. The following has been considered in this report and are discussed further in the sections that follow.

- Thin layers of potentially liquefiable soil were encountered below the groundwater (a depth of approximately 40 feet bgs) which might result in less than one inch of seismically-induced settlement.
- Installation of deep foundations should consider the proximity of existing buildings next to the site and the effects of construction vibration, particularly from pile driving. This could limit deep foundations to those that can be drilled into the underlying dense gravel.
- Conventional shoring techniques, as discussed in our Feasibility Assessment[1], appear to be feasible.
- Temporary and permanent support of the existing entrance ramp structure on the site should be considered in the selection of shoring types and project staging in order to provide continuous lateral support and not undermine existing foundations.

Liquefaction and Lateral Spreading

Liquefaction is defined as a decrease of the shear resistance of loose, saturated, cohesionless soil (e.g., sand) or low plasticity silt soils due to the buildup of excess pore pressures generated during an earthquake. This results in a temporary transformation of the soil deposit into a viscous fluid. Liquefaction can result in ground settlement, foundation bearing capacity failure, and lateral spreading of ground.

If groundwater is present at depths of greater than 40 to 50 feet bgs, only thin lenses of sand and silty sand are potentially liquefaction susceptible. We evaluated liquefaction at the site for a design-level, Cascadia Subduction Zone (CSZ) earthquake and crustal earthquake along the Portland Hills Fault (PHF). The analyses were completed using magnitudes (M) and peak ground surface accelerations (PGA) consistent with the respective design earthquakes. The estimated settlement for both cases was similar, with less than an estimated ¼ inch of seismically-induced settlement for an assumed groundwater level at about 40 feet bgs. If a design groundwater is estimated below about a depth of 45 feet bgs, then the risk of liquefaction would be greatly reduced.

With the small amounts of liquefaction settlement that were calculated, lateral spreading probably would be less than a few inches across the site. Given the presence of several buildings supported on piles and mats underlain by gravel in the vicinity, the magnitude of lateral spreading would likely be less.

Mitigating the effects of liquefaction and lateral spreading could be accomplished by incorporating either soil improvement (e.g. stone columns, jet grouting, etc.) or designing the number and size of deep foundations to resist the associated increases in vertical and lateral loads.

Code-Based Seismic Design Criteria and Requirements

Due to the potential for liquefaction of some soils on portions of the site, the code-based site classification should be considered Site Class F. Consequently, a site-specific seismic hazard study, including spectral site response analysis, will be required to develop site-specific values for use in the structural design. Site class is based on the average shear wave velocities of soils within 100 feet of the base of the planned new structure. This was measured directly in the top of the CPTs, approximately 40 feet bgs at the site. Based on measured shear wave velocities and the expected velocities in the underlying gravel units, structural design would likely be based on Site Class D.

The general, code-based seismic design criteria, in accordance with the 2014 Oregon Structural Specialty Code (OSSC), would be based on S_s equal to 0.99 g and S_1 equal to 0.42 g.

Mat Foundations
Based on the stiffness of site soils and depending on the magnitude of the structure loads, it may be feasible to support the new courthouse on a mat foundation. A mat foundation consists of a 2- to 5-foot-thick reinforced slab that distributes the weight of the structure over the entire building footprint. The use of a mat in lieu of deep foundations is normally evaluated as part of the design-level geotechnical services and must consider site-specific soil properties, actual structure loads, and tolerable settlement (usually limited to about 1 inch).

Deep Foundation Considerations
Due to the thickness of silt and clay soils at the site and the anticipated loads associated with the new structure, it may be necessary to support the courthouse on deep foundations that derive their capacity from the underlying gravel interpreted to be present below depths of about 45 feet bgs at the site. Depending on the structural requirements, including uplift and lateral resistance, it may be possible to use either driven steel piles (pipe or H-piles) or drilled, cast-in-drilled-hole (CIDH) piles (augercast or drilled shafts). Regardless of the foundation type selected, only limited capacity will be derived from the silt and clay soils within the top 45 feet bgs, and penetration into the more competent gravel will be required. The depth of penetration into the gravel will be a function of required foundation loads and relative density of the bearing unit, which has not been determined.

Drilled Piles
Cast-in-drilled-hole (CIDH) concrete piles are one deep foundation alternative for support of the planned 10- to 12-story courthouse. Based on the estimated loads and possible presence of cobbles, we have preliminarily considered 12- to 30-inch-diameter CIDH concrete piles for foundation support.

Axial Capacity
Axial capacity of the CIDH piles will be derived primarily from shaft friction in the gravel unit present below depths of 43 to 47 feet bgs. The contribution from end bearing is significantly reduced due to the magnitude of displacement required to engage full end bearing in addition to the disturbance resulting from drilling operations. For an estimated pile length of 85 feet (40 feet of penetration into gravel), pile capacities would range from about 250 kips to 400 kips for 12-inch-diameter piles and 30-inch-diameter piles, respectively. Piles should be spaced a minimum of three pile diameters, center-to-center.

Construction Considerations
Caving in the sand, gravel, and cobbles and significant loss of drilling mud have been reported when penetrating the gravel and cobble for foundations in downtown Portland. Construction of deep foundations may require the use of casing if “open-hole” techniques are used. In addition, a contingency should be included in the project budget and schedule for increased grout/concrete volumes.

Use of deep foundations requires full-time observation during construction to confirm subsurface conditions and construction procedures are consistent with our recommendations.

Driven Piles
Driven piles, such as steel pipe or H-piles, are commonly used to support mid- to high-rise structures downtown Portland. Theses foundations derive their capacity almost entirely in end bearing in the

underlying gravels. Depending on the type and size of the piles, typical loads for 12- to 18-inch-diameter pipe or H-piles range from 200 to 400 kips per pile.

Construction Considerations
Driven piles can generally be installed more efficiently than drilled piles in conditions similar to those at the site. Driving piles does not require casing when penetrating the silt and clay. The capacity of driven piles can be evaluated during driving with dynamic testing using a pile driving analyzer (PDA). The PDA provides information regarding the resistance of the pile with depth which can be used to develop site-specific terminal driving criteria. An indicator pile program that includes PDA on at least four piles should be considered to help refine the required length of pile and driving criteria in advance of the installation of production piles.

Use of deep foundations requires full-time observation during construction to confirm subsurface conditions and construction procedures are consistent with our recommendations.

Temporary Shoring
Temporary construction excavation and site safety are the sole responsibility of the contractor who also is solely responsible for the means, methods, and sequencing of construction operations. We are providing the following information only as a service to our client for planning purposes by client's design team. Under no circumstances should the information provided herein be interpreted to mean that PBS is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

Conceptual planning includes construction of one level below grade at the site. We estimate the base elevation of this level will be at a depth 12 to 15 feet bgs. Due to the proximity of existing streets and structures, there is not sufficient room to safely slope the excavation without impacting them. As a result, we recommend only using shoring that provides continuous support; open cuts will not be allowed. Although permanent groundwater was measured at depths below about 40 feet bgs, zones of perched water may be present and may rise in response to wet weather.

A wide variety of shoring systems are available for temporary shoring and have been discussed in our previously prepared report[1]. 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 opinion, a soldier pile wall combined with braces and struts or tiebacks may be used for shoring.

Due to the presence of relatively weak silt and clay soils within the planned depth of the excavation, soil nails and tiebacks may need to be long in order to derive enough capacity for shoring support. However, it may be possible to cantilever shoring to the planned depth without the need for tiebacks.

Underpinning of Existing Structures
Due to the proximity of the existing parking ramp structure to the proposed excavation below grade, consideration should be given to the type of shoring used and staging of excavation and construction in order to provide continuous lateral support. Underpinning the existing structure foundations with pin piles or micropiles that derive their capacity below the depth of excavation may be used in addition to more rigid shoring systems for excavations in the vicinity of the structures.

Regardless of the selected foundation installation type, shoring, or underpinning alternatives, a comprehensive pre-construction survey of adjacent structures and surrounding site features should be completed. This could include an optical survey of “targets” and benchmarks established on building walls, foundations, sidewalks and other features, measuring vibration and/or photo or video documentation. Following the survey, thresholds should be established and include contingency plans with required actions if detrimental effects are observed.

It will be important to maintain a schedule of completing regular, detailed observation during construction. This could include regular vertical/horizontal survey in addition to regular observation and documentation. These conditions should be compared to the conditions observed during the pre-construction survey.

LIMITATIONS

Prior to design and construction, additional subsurface explorations, laboratory testing, and geotechnical engineering analyses will be required. This report has been prepared for the exclusive use of the addressee and their architects and engineers for aiding in the conceptual planning and construction feasibility considerations of the proposed new county court house building and is not to be relied upon by other parties. It is not to be photographed, photocopied, or similarly reproduced, in total or in part, without the expressed written consent of the Client and PBS. It is the addressee's responsibility to provide this report to the appropriate design professionals, building officials and contractors to assure correct implementation of the recommendations.

The opinions, comments, and conclusions presented in this report are based upon information derived from our literature review, field explorations, laboratory testing, and engineering analyses. Conditions between or beyond our explorations may vary from those encountered. It is possible that soil, rock, or groundwater conditions could vary between or beyond the points explored.

Unanticipated soil and rock conditions and seasonal soil moisture and groundwater variations are commonly encountered and cannot be fully determined by merely taking soil samples or soil borings. Such variations may result in changes to our recommendations and may require additional funds for expenses to attain a properly designed and constructed project. Therefore, we recommend a contingency fund to accommodate such potential extra costs.

The scope of these geotechnical services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluation of hazardous substances in the soil, surface water, or groundwater at this site.

If there is a substantial lapse of time between the submission of this report and the start of work at the site, if conditions have changed due to natural causes or construction operations at or adjacent to the site, or if the basic project scheme is significantly modified from that assumed, this report should be reviewed to determine the applicability of the conclusions and recommendations presented herein. Land use, site conditions (both on and off site), or other factors may change over time and could materially affect our findings. Therefore, this report should not be relied upon after three years from its issue, or in the event that the site conditions change.

CLOSING

We trust this report meets your current needs. If you have any questions or wish to further discuss our observations, conclusions, and recommendations, please contact us at 503.248.1939.

Sincerely,
PBS Engineering and Environmental Inc.

REGISTERED PROFESSIONAL ENGINEER
53860PE
OREGON
JANUARY 15, 2002
RYAN WHITE
RENEWAL DATE: 6/30/2016

Tony Rikli, PE
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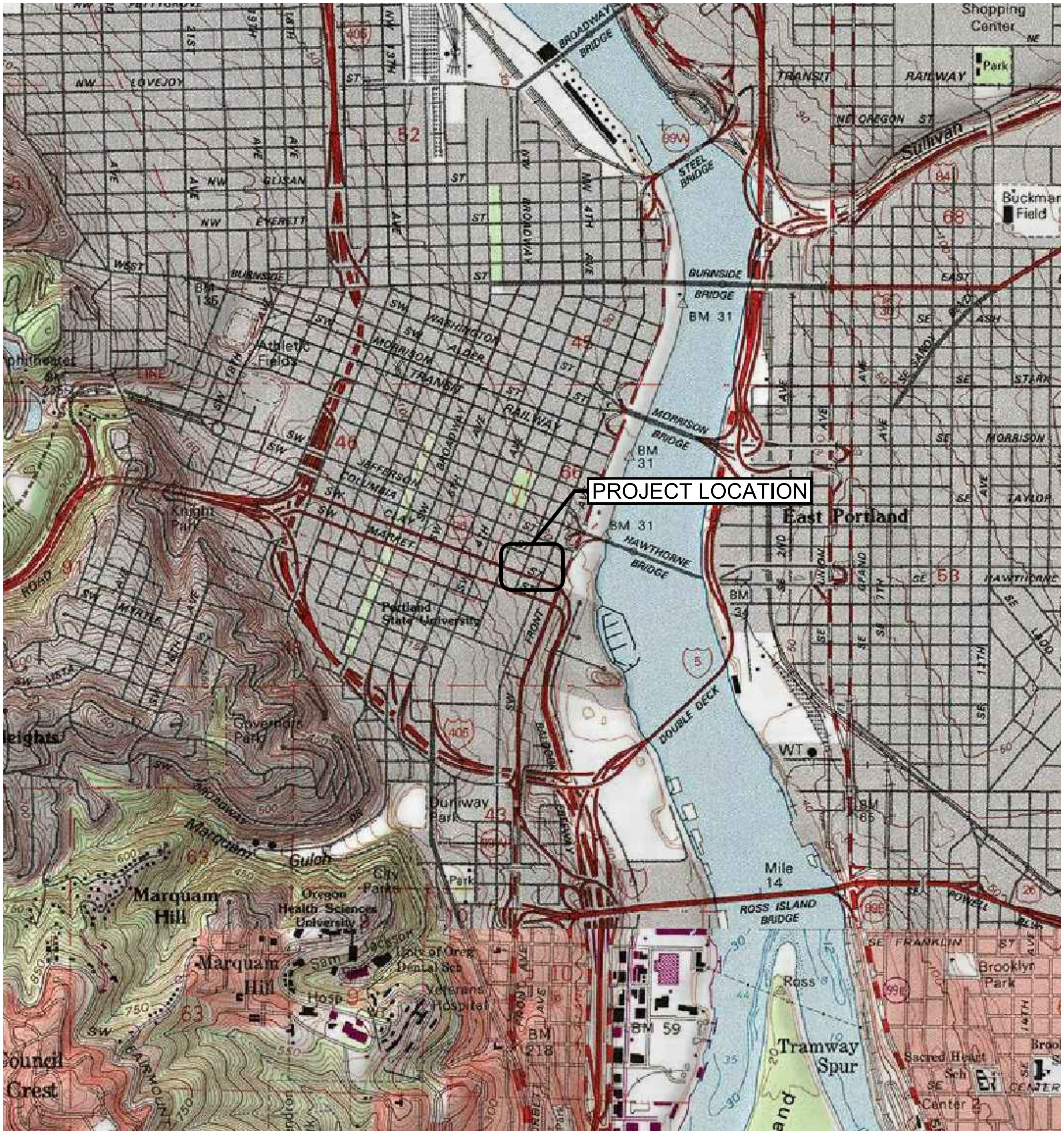
Senior Geotechnical Consultant

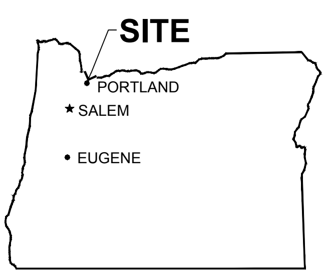
TR/RW/AR/rd

Attachments: Figure 1 – Vicinity Map
Figure 2 – Site Plan
Figures A1 through A2 – Logs for CPT-1 and CPT-2
Figure A3 – Shear Wave Velocity Profile

FIGURES



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: 2-TASK:4	VICINITY MAP	FIGURE
	DATE MAR 2015	BLOCK 128 PORTLAND, OREGON	1

SOURCE: © 2011 GOOGLE EARTH PRO, © 2012 GOOGLE

LEGEND

B-1	BORING NUMBER AND LOCATION FROM GEOCON REPORT DATED MAY 2001
CPT-1	CPT NUMBER AND LOCATION DATED FEBRUARY 2015

0' 30' 60' 120'

SCALE: 1" = 60'

PREPARED FOR: MULTNOMAH COUNTY

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PBS	PROJECT # 15194.869 PHASE: 2-TASK:4	SITE PLAN	FIGURE
	DATE MAR 2015	BLOCK 128 PORTLAND, OREGON	2

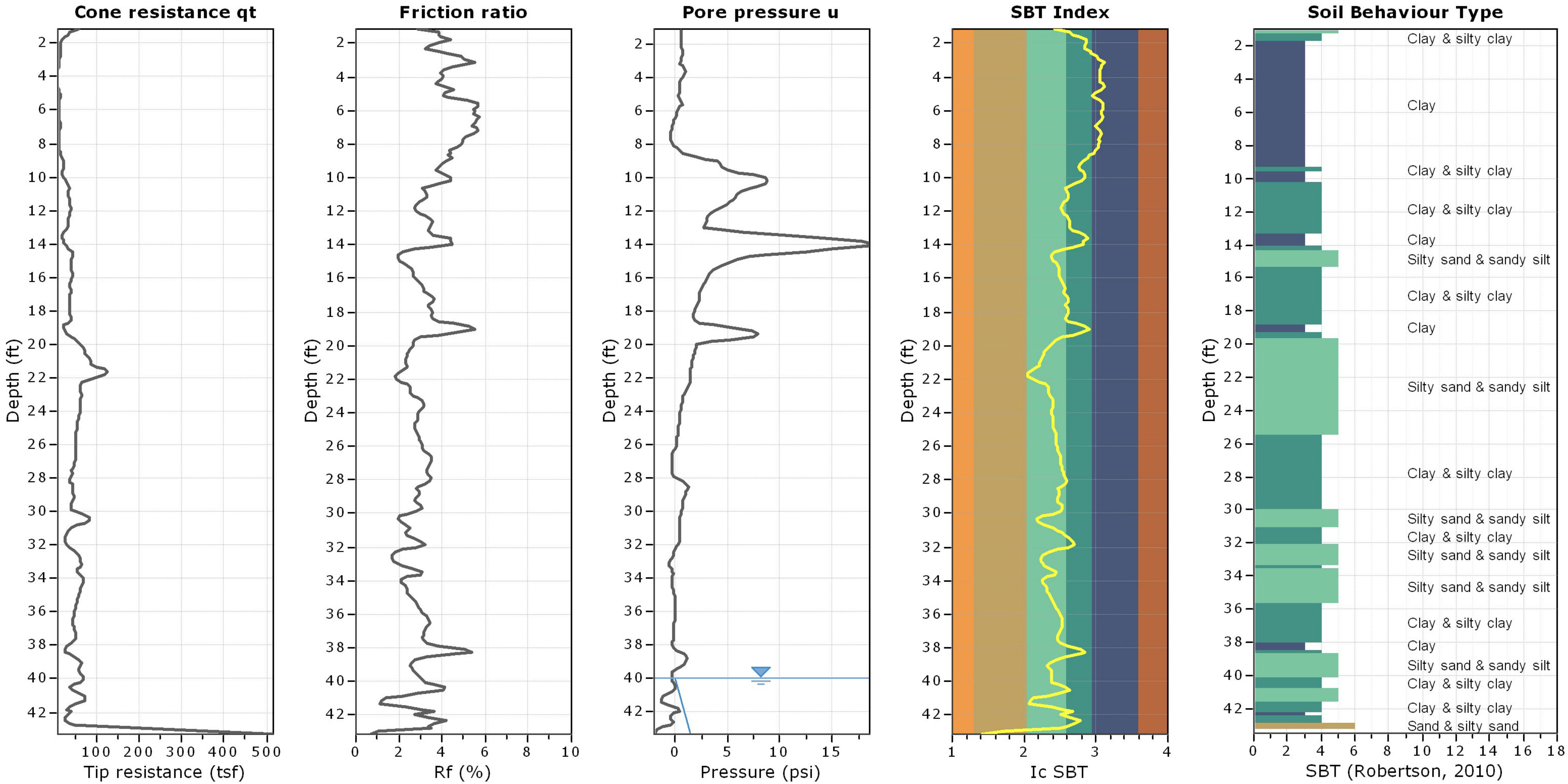
ATTACHMENT A

Field Explorations

PBS Engineering and Environmental, Inc.
4412 SW Corbett Avenue
Portland, Oregon 97239
www.pbsenv.com

Project: **15194.869 Multnomah County Courthouse**
Location: **Block 128**

CPT: 15015 CPT-1 Text File
Total depth: 43.31 ft, Date: 3/18/2015
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown



SBT legend

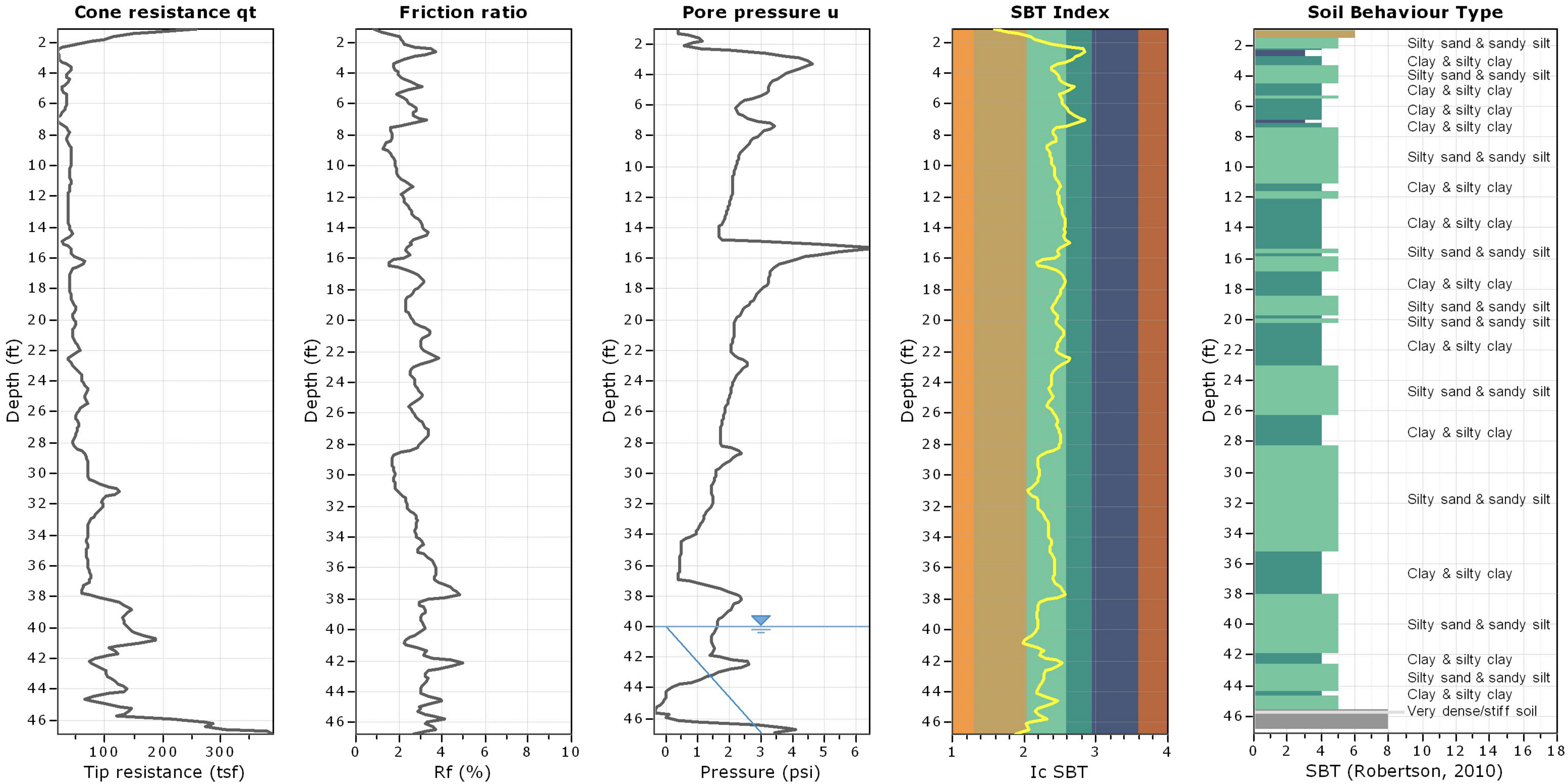
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

FIGURE A1

PBS Engineering and Environmental, Inc.
4412 SW Corbett Avenue
Portland, Oregon 97239
www.pbsenv.com

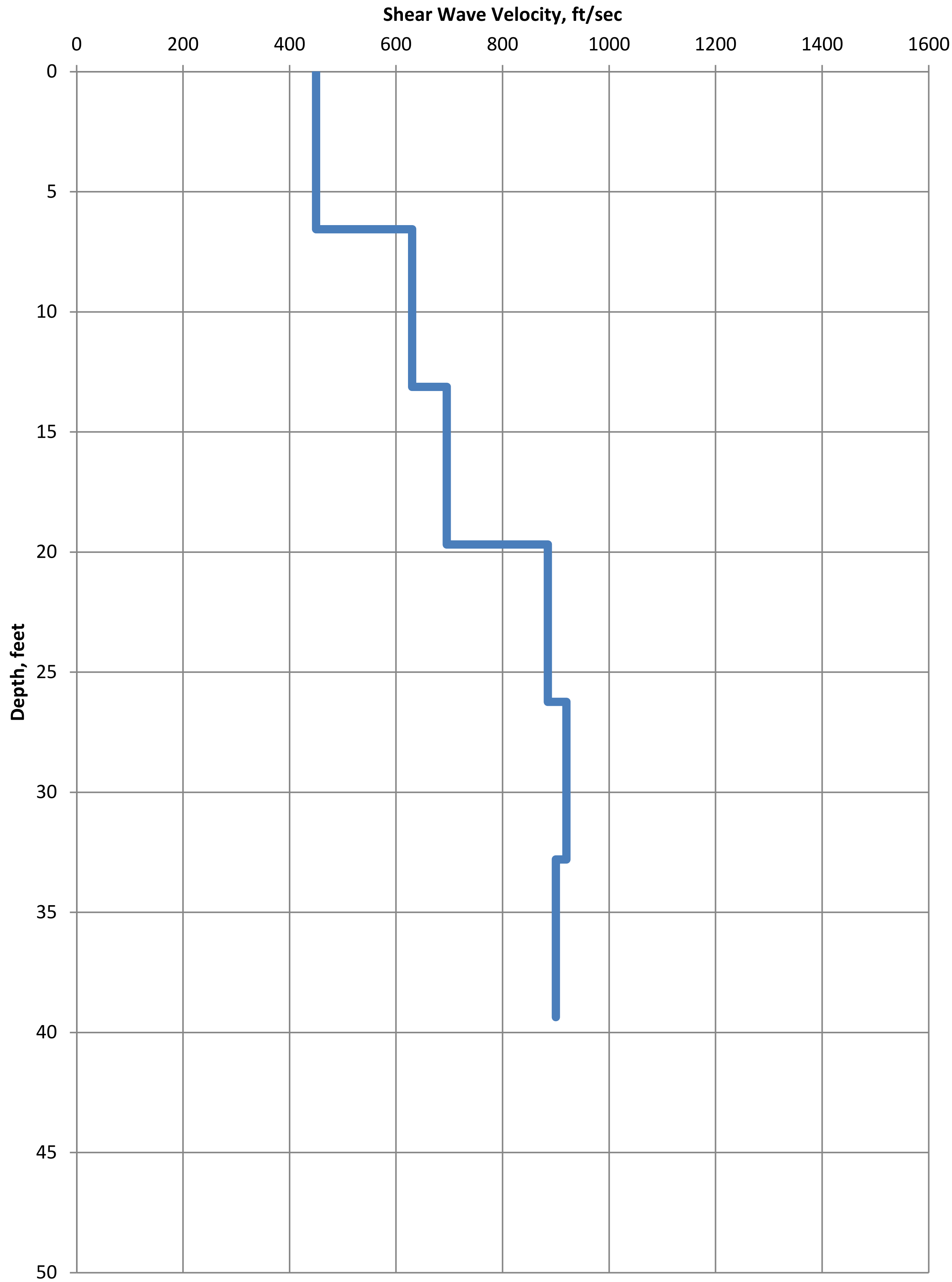
CPT: 15015 CPT-2 Text File
Total depth: 46.92 ft, Date: 3/18/2015
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

Project: **15194.869 Multnomah County Courthouse**
Location: **Block 128**



SBT legend
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

FIGURE A2



PREPARED FOR: MULTNOMAH COUNTY

PBS	PROJECT # 15194.869	SHEAR WAVE VELOCITY PROFILE	FIGURE
	DATE MAR 2015	BLOCK 128 PORTLAND, OREGON	A3