Geotechnical Engineering Evaluation

Proposed 840 Building
Spokane, Washington

for
Spokane Portland and Seattle, LLC

December 13, 2019

GeoEngineers
Earth Science + Technology

Geotechnical Engineering Evaluation

Proposed 840 Building
Spokane, Washington

for

Spokane Portland and Seattle, LLC

December 13, 2019

GeoEngineers

523 East Second Avenue
Spokane, Washington 99202
509.363.3125

Geotechnical Engineering Evaluation

Proposed 840 building
Spokane, Washington

File No. 18324-004-00

December 13, 2019

Prepared for:

Spokane Portland and Seattle, LLC
850 East Spokane Falls Boulevard, Suite 100
Spokane, Washington 99202

Attention: Patrick Farley

Prepared by:

GeoEngineers, Inc.
523 East Second Avenue
Spokane, Washington 99202
509.363.3125

David R. Lauder, PE
Senior Engineer

Teresa A. Dugger, PE
Associate

DAVID RYAN LAUDER
STATE OF WASHINGTON
42304
REGISTERED
PROFESSIONAL ENGINEER
12/13/19

DRL:TAD:tjh

Disclaimer: Any electronic form, facsimile or hard copy of the original document (email, text, table and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

GEOENGINEERS

Table of Contents

LIST OF TABLES

LIST OF FIGURES

APPENDICES

1.0 INTRODUCTION

This report presents the results of GeoEngineers, Inc.'s (GeoEngineers') geotechnical engineering evaluation during design for the proposed 840 Building located at 840 East Spokane Falls Boulevard in Spokane, Washington. The approximate location of the project site is shown in the Vicinity Map, Figure 1.

We understand McKinstry plans to construct and operate a Regional Health Building, which will be used by the joint Medical School operated by the University of Washington and Gonzaga University. The proposed four-story building will encompass a footprint of about 35,000 square feet, with a slab-on-grade floor. The site of the proposed building is situated at the southwest corner of the intersection of Spokane Falls Boulevard and Hamilton Street, north of the existing McKinstry Building. The site is currently occupied by several buildings used by McKinstry. We understand the businesses at this location will be relocated, the buildings demolished, and the proposed 840 Building constructed over the same general footprint.

Typical column loads for the proposed 840 Building are estimated to range between about 330 and 430 kips, although two columns will carry loads on the order of about 600 kips. The column loads are an unfactored combination of dead plus live loads. Finished floor grade for the proposed building will be in the range of about Elevation 1,885 to 1,886, which is near existing exterior site grade and finished floor grade for the existing buildings. Elevations in this report are based on the North American Vertical Datum of 1988 (NAVD 88), unless otherwise indicated. Additional site improvements likely will include installation of new utilities, exterior hardscape and landscaping, and stormwater disposal facilities.

Redevelopment of the existing McKinstry Building (also referred to as the SIERR Building and the Great Northern Building) located south of the project was the subject of previous environmental, geotechnical and hydrogeologic services provided by GeoEngineers and others. Redevelopment of the site to the south included conducting environmental assessments of soil and groundwater, and remediation and environmental permitting of contaminated soil associated with historic railroad activities on the site. We understand Stantec will be providing environmental consultation services to McKinstry for this project.

Note that the recommendations provided in this report do not include provisions for the handling of contaminated soil. We should be contacted to re-evaluate our recommendations if the results of environmental testing conducted by others indicate the presence of contaminated soil at the site.

2.0 SCOPE OF SERVICES

The purpose of our services was to provide geotechnical engineering recommendations for design and construction of the proposed 840 Building. Our recommendations are based on review of existing information, subsurface exploration, laboratory testing and engineering analysis. We performed our services in accordance with our proposal dated November 5, 2019. Written authorization of our services was provided on November 11, 2019. Our specific scope of geotechnical services included:

1. Reviewing our files for applicable subsurface information.
2. Exploring subsurface conditions at the site by drilling borings.
3. Conducting geotechnical laboratory testing of select soil samples collected from the borings.

4. Developing geotechnical engineering recommendations for the project including:
    a. Recommendations for site preparation and fill placement.
    b. Recommendations for design and construction of shallow spread foundations.
    c. Evaluation of potential seismic hazards and recommendations for seismic design criteria based on the International Building Code (IBC).
    d. Recommendations for design and construction of slab-on-grade floors.
    e. Recommendations for thickness of hot-mix asphalt (HMA) and portland cement concrete (PCC) pavement.
    f. An evaluation of the feasibility of on-site infiltration of post-development stormwater. We provide recommendations for design infiltration rates for bio-infiltration swales and other shallow infiltration facilities, as well as drywell outflow rates. We also provide recommendations for construction and testing of stormwater infiltration facilities.

3.0 SITE SURFACE CONDITIONS

The project site encompasses about 1.5 acres and is located southwest of the intersection of Spokane Falls Boulevard and North Hamilton Street. The existing McKinstry building is situated south of the site and a paved parking area is situated to the west of the site. The site also is situated near a bend in the Spokane River, with the river located within several hundred feet of the site to the east, west and south.

Most of the site is encompassed by existing single-story warehouse buildings. The remainder of the site is generally paved with asphalt concrete pavement. Site grades across the site are relatively level with elevations ranging from about 1,884 to 1,886. The approximate locations of existing site features in the vicinity of the project area are shown in the Site Plan, Figure 2.

4.0 SITE SUBSURFACE CONDITIONS

4.1. Field Activities

We explored subsurface conditions at the site on November 26 and 27, 2019 by drilling six borings (B-1 through B-6). The borings were advanced to depths in the range of about 20 to 30 feet below ground surface (bgs). Locations of our borings relative to existing site features are shown in Figure 2.

Representative soil samples from the borings were returned to our laboratory for examination. Detailed descriptions of our site exploration and laboratory testing programs along with exploration logs are presented in Appendix A.

4.2. Literature Review

4.2.1. Geologic Conditions

The Washington State Department of Natural Resources maps the site as Quaternary Alluvium. This geologic unit consists predominantly of silt, sand and gravel deposits in present-day stream channels and on flood plains. It was formed from reworked glacial flood deposits and is underlain by deeper glacial flood deposits, which extend to depths greater than 100 feet below ground surface in the site vicinity.

4.2.2. Previous Explorations

Previous explorations have been conducted on and to the south of the site. The previous explorations were associated with the existing Great Northern Building, and included groundwater monitoring wells (MW-4 and MW-5) installed by Hart Crowser in 1991, boring B-100 drilled by GeoEngineers in 2010 and a pilot test borehole drilled for the SIERR Building ground source heat pump (GSHP) system drilled in 2010. We also reviewed boring logs from the Hamilton Street Bridge located about 200 feet to the south of the project site, which were drilled by the Washington State Department of Transportation (WSDOT) in 1981. The approximate locations of the previous explorations are shown in Figure 2 (note that the WSDOT borings were drilled beyond the limits of Figure 2). Logs of the previous explorations are presented in Appendix B.

4.3. Subsurface Conditions

4.3.1. General

We characterized the soil encountered in our borings into two general units based on engineering properties: (1) Upper Sand and Gravel; and (2) Lower Gravel.

4.3.2. Upper Sand and Gravel

At the locations of each of our borings, we encountered an upper layer consisting of loose to dense sand and gravel with variable silt and cobble content. The Upper Sand and Gravel unit extended to depths in the range of about 6 to 10 feet bgs. Standard penetration test (SPT) N-values ranged from 5 to 52, with an average of about 10. Results of grain-size analyses on representative samples indicate the fines (silt- and clay-sized soil particles passing the U.S. No. 200 sieve) content is in the range of about 6 to 13 percent. While we did not observe debris in the samples collected in the upper 6 to 10 feet, the variability of the material and SPT N-values suggests this soil is fill, consisting of either imported granular soil or reworked native soil. We characterized the Upper Sand and Gravel unit as having variable strength, compressibility and moisture sensitivity, and moderate to high permeability. The approximate elevation of the bottom of the Upper Sand and Gravel unit is shown at the boring locations in Figure 2.

4.3.3. Lower Gravel

Below the Upper Sand and Gravel unit, we encountered a natural alluvial deposit of medium dense to very dense fine to coarse gravel with sand, cobbles and occasional boulders, which extended to the depths explored. Based on review of the boring log for the GSHP Pilot Test borehole, the Lower Gravel unit extends to a depth of at least 186 feet bgs. SPT N-values of samples collected in the Lower Gravel unit ranged from 14 to 80, with an average of about 45. Results of grain-size analyses on representative samples indicate the fines content of the Lower Gravel unit is in the range of about 4 to 9 percent. We characterized the lower gravel unit as having moderate strength, low compressibility, high permeability and low susceptibility to changes in moisture content.

4.4. Groundwater Conditions

We encountered groundwater in the borings at depths in the range of about 15½ to 17 feet bgs at the time of drilling. Groundwater elevations below the site are influenced by the adjacent Spokane River, Groundwater elevations vary seasonally, and from year to year depending on the elevation of the river. Peak flows in the Spokane River typically occur during late winter through spring.

We also reviewed results of previous groundwater elevation measurements associated with existing monitoring well MW-4, which was installed in 1991 by Hart Crowser. Groundwater monitoring records dating back to 1994 indicate the highest groundwater elevation measured at MW-4 was 1,876.49 (7.62 feet below top of well casing), and the lowest groundwater elevation measured was 1,867.09 (17.22 feet below top of well casing). The highest measured groundwater elevation was recorded during a flood event where the seasonal peak streamflow recurrence interval was about 20 years. Based on review of groundwater elevation data collected during the spring quarter (March through May), groundwater levels during years with near average precipitation and runoff ranged from about 9.9 feet to 13.81 feet bgs (Elevation 1,874.41 to 1,870.50).

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our geotechnical engineering evaluation, we believe subsurface conditions are suitable for support of the proposed improvements, provided recommendations in this report are followed during design and construction. The following presents a brief description of geotechnical considerations for this project:

- The site is suitable for support of foundation loads using shallow spread footings. However, the Upper Sand and Gravel unit exhibits variable strength and compressibility characteristics. We estimate that total and differential foundation settlements in excess of 1 inch could occur if foundations are supported on the unimproved Upper Sand and Gravel unit. Therefore, in order to provide more uniform bearing conditions and reduce the potential for unacceptable foundation settlement, we recommend overexcavating the Upper Sand and Gravel unit from below foundation grade to expose the Lower Gravel unit, and either recompacting the excavated soil or replacing with imported structural fill to meet minimum density requirements. The elevation of the top of the Lower Gravel Unit varies across the site. For preliminary estimating purpose, we recommend assuming excavations below foundations could extend to Elevation 1,876 to remove the Upper Sand and Gravel unit.
- Site soil encountered in our borings is generally suitable for reuse as structural fill. However, portions of the Upper Sand and Gravel unit are moisture sensitive and will be difficult to properly work or compact if the moisture content at the time of earthwork is more than about 3 percentage points wet or dry of optimum. Accordingly, the reuse of portions of the Upper Sand and Gravel unit during the winter and early Springs months might not be feasible.
- The site is suitable for infiltration of post-development stormwater. Given the relatively shallow seasonal high groundwater elevations below the site, stormwater infiltration facilities should be limited to shallow systems, such as infiltration trenches or galleries, low-profile drywells, or single-depth (City of Spokane Type I) drywells (if feasible).
- As stated previously, recommendations in this report do not include provisions for the handling of contaminated soil. We should be contracted to re-evaluate our recommendations if the results of environmental testing conducted by others indicate the presence of contaminated soil at the site.

These and other considerations are discussed in the following sections of this report. This report should be read in its entirety to fully understand geotechnical design and construction considerations and recommendations.

5.1. Seismic Considerations

5.1.1. Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and/or sand boils may result from liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to the structures. Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level. Dense soils/bedrock or soils that exhibit cohesion are generally considered not to be susceptible to liquefaction.

The natural soil deposits encountered in our borings within the upper 30 feet below site grade generally consist of medium dense to dense gravel. While information regarding the relative density of the lower gravel unit below the groundwater table from our recent borings is limited, results of the previous borings for the Great Northern Building GSHP system as well as borings drilled by WSDOT for the Hamilton Street Bridge indicate the lower gravel unit extends to a depth of at least 180 feet bgs at the site and the relative density of the deposit is generally medium dense to dense. On this basis, it is our opinion that the risk of liquefaction at the site is low.

5.1.2. Fault Rupture

We reviewed the United States Geologic Survey (USGS) online database of mapped quaternary faults. There are no mapped active faults in the vicinity of the site. Based on the location of mapped faults, it is our opinion that damage to the proposed building due to fault rupture is low.

5.1.3. IBC Seismic Design Information

Based on the results of our explorations and review of available information including water well reports, geologic mapping, and a previous geophysical survey conducted nearby on the Gonzaga University campus in the same geologic unit, it is our opinion that the site classifies as a Site Class D. Based on discussions with the project structural engineer (DCI Engineers), we understand the 2015 edition of the IBC will be used for design. Therefore, we have provided design seismic parameters based on the 2015 IBC, as shown in Table 1.

TABLE 1. MAPPED 2015 IBC SEISMIC DESIGN PARAMETERS

Seismic Design Parameters	Recommended Parameters
Site Class	D
Mapped Spectral Response Acceleration at Short Periods (S_s)	0.333
Mapped Spectral Response Acceleration at 1 Second Period (S_1)	0.115
Site Amplification Factor at 0.2 Second Period (F_a)	1.534
Site Amplification Factor at 1 Second Period (F_v)	2.34
Design Spectral Acceleration at 0.2 Second Period (S_{DS})	0.340
Design Spectral Acceleration at 1 Second Period (S_{D1})	0.179

Notes: Parameters developed based on Latitude 47.661400 and Longitude -117.397332 using the ATC Hazards online tool.

5.2. Foundation Support

5.2.1. Minimum Width and Embedment

Individual (column) and continuous (wall) footings should be designed with minimum dimensions of 24 inches and 18 inches, respectively. Exterior footings should be embedded at least 24 inches below exterior finished grade for frost protection. Interior footings within heated areas should be embedded at least 12 inches below finished floor grades to provide sufficient bearing resistance.

5.2.2. Allowable Bearing Pressures

Individual and continuous footings should bear on soil prepared as recommended in Sections 5.6 and 5.7 of this report. Additionally, these recommendations also are contingent upon final foundation grade situated between about Elevation 1,880 and 1,884, and final interior and exterior grades (within a distance of 2B of footings) between about Elevation 1,885 and 1,886. Because of the relatively shallow groundwater table and the influence of groundwater elevation on our bearing capacity calculations, we should be contacted to review our recommended allowable bearing pressures if final foundation grades will be higher or lower than assumed in our analyses.

Isolated (columns) and continuous (wall) footings may be designed using the allowable bearing pressures presented in Table 2. The weight of overlying fill may be neglected when estimating foundation loads. The allowable bearing pressures include a safety factor of about 3 and may be increased by one-third for short-term live loads such as wind and seismic events.

TABLE 2. ALLOWABLE BEARING PRESSURES

Type	Footing Width (ft)	Allowable Bearing Pressures (psf)
Isolated Footings	2 to 4 Greater than 4	4,000 5,000
Continuous Footings	2 or less Greater than 2	3,000 4,000

5.2.3. Modulus of Vertical Subgrade Reaction

Mats and large footings may be designed using beam-on-elastic-foundation, soil-structure interaction analysis. For this analysis procedure, estimates of the modulus of vertical subgrade reaction (Ks) are required (also commonly referred to as spring constants). It should be noted that Ks is not an inherent soil property. Instead, the value of Ks depends on a number of factors including: the dimensions of the loaded area, the depth of the loaded area below ground surface, and the position of the soil spring below the mat or foundation. We have provided estimates of Ks values that can be used to estimate soil spring constants in soil-structure interaction analyses in Tables 3 and 4 below.

TABLE 3. MODULUS OF VERTICAL SUBGRADE REACTION – SQUARE OR RECTANGULAR FOUNDATIONS

Foundation Width (ft)	Modulus, Ks (pci)
2	300
4	125
6	70
8	50
10	35
12	30

Notes: ft = feet; pci = pounds per cubic inch

TABLE 4. MODULUS OF VERTICAL SUBGRADE REACTION – CONTINUOUS FOUNDATIONS

Foundation Width (ft)	Modulus, Ks (pci)
1.5	75
2	60
3	50
4	40
5	30
6	25

Beam-on-elastic-foundation analysis is a simplification of actual soil-structure interactions. One of the primary simplifying assumptions is that each spring acts independently of other springs. In reality, loads imparted through a foundation into the soil influence both the soil directly below the load as well as the surrounding soil. Additionally, because of the number of variables involved, there is considerable uncertainty associated with estimating Ks. Ways to account for these uncertainties with the values and simplifications inherent in the beam-on-elastic foundation analyses include:

- Conduct a parametric study to evaluate the effect of varying Ks values on mat or foundation design. The American Concrete Institute (ACI) suggests varying Ks from one-half the estimated value to 5 or 10 times the estimated value.
- Use different Ks values at different locations below the mat or foundation. This can be accomplished by doubling the Ks values along the perimeter of the mat or foundation. The intent of this approach is to model the real-life stress interaction that is not otherwise incorporated into a beam-on-elastic-foundation model.

5.2.4. Settlement

Based on the estimated foundation loads provided, we estimate that total and differential foundation settlement (between columns, or along approximately 50 feet of continuous foundations) should be less than about 1 inch. If foundation loads exceed the estimated amount, it will be necessary for us to re-evaluate foundation settlement for the proposed building.

Settlement should occur relatively rapidly, essentially as loads are applied. On this basis, post-construction total and differential settlement should be small, and will be a function of the magnitude of live load. Loose soil not removed from footing excavations, or disturbance of soil at foundation grade during construction could result in larger settlements than estimated.

5.2.5. Lateral Resistance

The ability of shallow foundations to resist lateral foundation loads is a function of the frictional resistance against the foundation base and the passive resistance which can develop on the face of below-grade elements of the structure as those elements move horizontally into the soil. For foundation grade prepared as recommended herein, the allowable frictional resistance may be computed using a coefficient of friction of 0.35. This value should be applied to vertical dead load forces for the contact between the bottom of the footing and supporting material.

The allowable passive resistance on the face of footings may be computed using an equivalent fluid density of 300 pounds per cubic foot (pcf), triangular distribution, for on-site soil or imported structural fill. This is based on the condition that backfill placed against embedded elements is compacted to at least 90 percent of the maximum dry density (MDD) for a distance of at least 2.5D beyond the edge of the foundation element (where D is the depth from ground surface to the bottom of the foundation element). Note that lateral movement on the order of about 0.01D will be required to mobilize the design passive resistance.

Both the frictional coefficient value and the equivalent fluid density value presented above include a safety factor of about 1.5.

5.3. Floor Slab Support

The floor slab may be supported on-grade, provided it is underlain by prepared subgrade as recommended in the "Site Preparation and Earthwork" Section 5.6 of this report. We recommend the building floor slab be designed using a modulus of vertical subgrade reaction (k) of 300 pci. Please note that this value is valid for floor slabs designed to resist point loads. The modulus of vertical subgrade reaction varies as a function of the size of the loaded area. Refer to Tables 3 and 4 for recommended modulus values for larger loaded areas. The structural engineer should design the thickness and required reinforcement of the floor slab based on the anticipated structural floor loads.

To retard the upward wicking of moisture beneath the floor slab, we recommend that a capillary break be placed over the subgrade. To that end, we recommend that floor slabs be underlain by at least 4 inches of free-draining crushed rock. The crushed rock should meet the criteria outlined in the previous section of this report titled "Structural Fill" Section 5.7.

A vapor retarder consisting of durable plastic sheeting also may be used in areas where the prevention of moisture migration through the building floor slab could adversely influence performance of adhesives, which might be used to anchor carpet, tile or other floor finishes to the slab. Because of selection of flooring material, and the associated manufacturer warranties of various flooring material, is generally not available during the geotechnical evaluation, we believe the architect is in a better position make the final determinations regarding use of a vapor retarder. Currently, the ACI does not recommend placing a moisture break layer of sand or crushed rock above plastic vapor retarders unless the building roof is in-place at the time of slab construction. If a moisture break layer is not used, appropriate consideration should be given to the cement type used for the slab concrete, jointing layout and curing operations to reduce the potential for curling of the slab.

5.4. Pavements

We recommend pavement materials at the site conform to applicable sections of the 2018 WSDOT Standard Specifications. Specifically, asphalt surfacing should consist of plant-mixed HMA placed and compacted in general accordance with Sections 5-04 (Hot-Mix Asphalt), 9-02 (Bituminous Materials) and applicable sections of 9-03 (Aggregates).

Pavement subgrade should be prepared as outlined in the “Site Preparation and Earthwork” Section 5.6 of this report. Soil placed as structural fill and gravel placed as crushed surfacing base course (CSBC) within proposed pavement areas should be compacted as outlined in the “Structural Fill” Section 5.7 of this report. We estimate the resilient modulus of properly prepared subgrade should be at least 10,000 pounds per square inch (psi).

Traffic loading information was not available at the time we prepared this report. For design purposes, we assume that traffic will consist predominantly of automobiles with occasional delivery trucks, buses or garbage trucks. On this basis, we recommend the following preliminary pavement thicknesses for site pavements. These thicknesses should be reviewed once design-level information is available.

TABLE 5. HMA PAVEMENT THICKNESS RECOMMENDATIONS

Pavement Areas	HMA Thickness (inches)	CSBC Thickness (inches)
Light-duty (automobile parking areas)	2.5	6
Heavy-duty (access and truck loading/unloading)	3	8

The upper 2 inches of CSBC may be replaced with crushed surfacing top course (CSTC) to aid the contractor in finished grading and preparation for paving. The recommended pavement sections are based on the assumption that a regular maintenance program will be used, which includes periodic sealing of joints and cracks, and occasional repair or replacement of isolated damaged areas.

We recommend PCC pavements in heavy-duty areas consist of at least 8 inches of plain jointed PCC over 4 inches of CSBC. Transverse and longitudinal joints should be spaced no greater than 14 feet on center.

5.5. Site Drainage

The following sections provide information on temporary drainage and stormwater considerations.

5.5.1. Temporary Drainage

As indicated previously, the groundwater table below the site is influenced by the water level of the Spokane River. Groundwater elevations below the site likely will be highest during the spring. We recommend earthwork activities that include excavation below possible seasonal high groundwater elevations be conducted during the summer and fall months when groundwater elevations will be lower to reduce the potential for encountering groundwater. Given the high permeability of the natural gravel deposit, temporarily dewatering of excavations that extend below the groundwater table will be very difficult, to impossible without supplemental improvements, such as shoring or cut-off walls.

Some local ponding of water from precipitation could occur in excavations during construction. Site excavations should be provided with appropriate ditches and sumps to keep exposed areas as dry as possible.

5.5.2. Stormwater Considerations

We recommend that all surfaces be sloped to drain away from proposed structures. Pavement surfaces and open spaces should be sloped such that surface runoff is collected and routed to suitable discharge points. Roof drains should be tight lined to suitable discharge points located at least 15 feet from building perimeters.

Based on the results of our site exploration, laboratory testing and engineering analyses, it is our opinion that the site is suitable for infiltration of post-development stormwater. In our opinion, the potential for negative on-site or down-gradient impacts (such as surface flooding or subsurface flooding into crawl spaces or basements) due to increased post-development infiltration is low. This opinion is based on the relatively high permeability of site soils and estimated depth to groundwater. Additionally, much of the existing site is currently paved or impervious. Therefore, we anticipate the net change in infiltration of stormwater between pre- and post-development conditions should be relatively minor.

We recommend that a GeoEngineers' representative be on-site during infiltration facility installation to observe excavations to confirm that appropriate target soil units are exposed, or alternatively, provide guidance for modifications to the systems if unsuitable soil is encountered. Additionally, we recommend full-scale testing be conducted on installed infiltration facilities promptly upon completion, but before final grading and paving is complete to confirm compliance of the system to design requirements. If results of testing indicate modifications are required, such as increasing the size or depth of shallow facilities, or installation of additional drywells, those modifications can be made more expediently before final site work is complete.

Infiltration facilities should be situated such that applicable setback and other site suitability criteria as outlined in the *Spokane Regional Stormwater Manual* (SRSM) are met. The SRSM requires a minimum 4 feet of vertical separation between the bottom of infiltration facilities and a limiting layer (in this case typical seasonal high groundwater). Results of our recent borings indicate the groundwater gradient is generally from the southeast to northwest across the site (away from the river). Therefore, elevations of infiltration facility bottoms required to maintain a minimum 4 feet of vertical separation will depend on their location. Using a typical seasonal high groundwater elevation of 1,874 at the location of MW-4 as a baseline, and groundwater elevation measurements from our recent borings, we recommend the following minimum elevations for infiltration facilities as presented in Table 6 below (i.e. the infiltration gallery bottom, drywell bottom, etc. should be placed at or above the elevations listed in Table 6).

TABLE 6. MINIMUM INFILTRATION FACILITY BOTTOM ELEVATIONS

Location	Minimum Elevation (ft)
B-1	1,879
B-2	1,878
B-3	1,876.5
B-4	1,877.5

Location	Minimum Elevation (ft)
B-5	1,878.5
B-6	1,878

Linear interpolation may be used for proposed infiltration facilities located between boring locations.

5.5.2.1. Bio-infiltration Swales

Based on the results of our explorations and laboratory testing, most of the soil encountered in our borings and anticipated to be within the upper 4 feet below bio-infiltration swale bottoms should contain less than 12 percent fines. Therefore, in our opinion, equations 6-1a and 6-1c may be used to size bio-infiltration swales. If soil containing more than 12 percent fines is encountered at planned swale subgrade, it should be removed and replaced with suitable sand and gravel containing less than 12 percent fines.

We anticipate that imported topsoil will be required for the treatment zone. Imported topsoil should meet cation exchange capacity (CEC), organic matter content and infiltration rate criteria as outlined in the SRSM.

5.5.2.2. Drywells

Drywells should be situated at least 20 feet from existing and proposed buildings. Drywells also should be spaced at least 30 feet apart. We estimated drywell outflow rates using the Spokane 200 Method as outlined in the SRSM. We estimated drywell outflow rates for standard City of Spokane Type 1 single-depth drywells, as well as alternative low-profile drywells including 72-inch-diameter, 4-foot-tall drywells with flat lids, and 500-gallon low-profile drywells, which are manufactured locally. Our recommendations for design drywell outflow rates are presented in Table 7. Results of Spokane 200 Method analysis are presented in Table 8.

TABLE 7. RECOMMENDED DESIGN DRYWELL OUTFLOW RATES

Drywell Type	Recommended Design Outflow Rate Option 1 (cfs)	Recommended Design Outflow Rate Option 2 (cfs)
Type 1	0.11	0.30
72-Inch-Diameter Low Profile	0.07	0.30
500-Gallon Low-Profile	0.04	0.15

Notes: cfs = cubic feet per second

Option 1 consists of installing drywells to their planned elevation, provided bottom elevations meet criteria as outlined in Table 6, and discharging stormwater into the Upper Sand and Gravel unit. Additionally, Option 1 assumes excavations encounter sand and gravel soil with less than 9 percent passing the U.S. No. 200 sieve. If a layer of silty sand or silty gravel is encountered at planned drywell bottom elevation, the silty sand or gravel layer should be overexcavated to expose underlying sand or gravel with less than 9 percent passing the U.S. No. 200 sieve, and the excavation backfilled with drain rock to re-establish the planned drywell bottom elevation.

Option 2 consists of overexcavating to Elevation 1,874 and replacing the excavation with washed drain rock to re-establish planned facility bottom.

5.5.2.3. Shallow Infiltration Facilities

If shallow infiltration facilities, such as infiltration trenches or infiltration galleries will be used, they should be situated at least 20 feet from existing and proposed buildings. We estimated the saturated hydraulic conductivity and infiltration rates of site soil using procedures outlined in Section 6.B.4 of the Washington State Department of Ecology 2019 *Stormwater Management Manual for Eastern Washington* (SWMMEW). Based on the results of our analyses, we estimated the unfactored saturated hydraulic conductivity (Ksat) of site soil within the zone of infiltration facilities is about 100 inches per hour (in/hr). Correction factors should be applied to the unfactored Ksat value to determine the design value (Ksat design). Results of our estimates of saturated hydraulic conductivity based on grain-size analyses are presented in Table 8.

Based on the results of our subsurface explorations and laboratory testing, we recommend using a total correction factor of 0.2 in accordance with Table 6.4 of the SWMMEW (equivalent safety factor of 5). On this basis, our recommended Ksat design value is 20 inches per hour. Note that hydraulic conductivity and infiltration rate (I) are not equivalent. Infiltration rate is a function of both the hydraulic conductivity of the soil and the hydraulic gradient (i).

$$I = K_{sat} \times i$$

Where:

I = infiltration rate of water through a unit cross section of the infiltration facility

Ksat = saturated hydraulic conductivity of the soil below the infiltration facility

i = steady state hydraulic gradient ($\Delta h/\Delta z$)

Given the potential for seasonal high groundwater conditions to be about 4 feet below facility bottoms, we estimated the potential hydraulic gradient that could develop during the design storm event using the Green-Ampt equation. We understand the peak flow for the 10-year design storm event is 3.2 cfs with a total volume of about 2,200 cubic feet. Our analyses indicate the hydraulic gradient that could develop below infiltration facilities during the design storm event when the groundwater table is 4 feet below the bottom of the infiltration facility is about 1 foot per foot. **Therefore, we recommend using a long-term design infiltration rate of 20 in/hr to design shallow infiltration facilities.** This applies to infiltration through the bottom of shallow infiltration systems and assumes that shallow infiltration facilities will be located within the Upper Sand and Gravel unit. If a layer of silty sand or gravel is encountered at planned facility bottom, the material should be overexcavated to expose sand and gravel with less than 9 percent fines and the excavation should be backfilled with washed drain rock to re-establish planned facility bottom. We recommend GeoEngineers be present during earthwork activities, prior to installation of the infiltration facilities, to observe soil conditions and recommend adjustments to earthwork activities, as necessary.

5.6. Site Preparation and Earthwork

We anticipate initial site preparation and earthwork operations could include: (1) demolition and removal of existing structures; (2) clearing, stripping and grubbing; (3) excavation and removal or relocation of existing underground utilities; (4) site grading to establish pavement, hardscape and slab-on-grade floor subgrades; and (5) excavation and filling to establish proposed foundation grades, floor slab and exterior site grades. Our specific recommendations for site preparation and earthwork are presented in the following sections.

5.6.1. Initial Site Preparation

Existing structures and pavements should be demolished. Active underground utilities should be excavated and relocated outside of improvement areas. Abandoned underground utilities should be excavated and removed or abandoned in place with lean concrete or grout. The resulting excavations and voids should be backfilled with structural fill, as defined in the following section of this report. Demolition debris should be removed and disposed of off-site in accordance with local, state and federal regulations.

5.6.2. General Grading and Excavation

Because upper portions of site soil exhibit variable strength and compressibility characteristics, we recommend excavating at least a portion of the Upper Sand and Gravel unit and either recompacting the excavated soil to meet minimum density requirements or replacing the excavated soil with properly compacted imported structural fill. In order to provide uniform bearing conditions and reduce the potential for unacceptable differential settlement, we recommend the following:

- Below foundations, overexcavate to expose the Lower Gravel Unit. Because of the variable thickness of the Upper Sand and Gravel unit, it will be critical for the geotechnical engineer-of-record to be on site to observe foundation excavations and confirm the target soil unit is encountered. For preliminary planning and estimating purpose, we recommend assuming overexcavation below foundations will extend to Elevation 1,876. Additional overexcavation might be required if unsuitable soil is encountered at working subgrade.
- Below floor slabs, overexcavate at least 2 feet below slab subgrade elevation.

Depending on a number of factors such as foundation layout, it might be more economical to excavate the entire building footprint to expose the Lower Gravel unit and reuse on-site soil or replace with imported structural fill. The decision as to whether overexcavation is limited to individual footings or to the entire building footprint is a means and methods consideration, assuming other mitigating factors such as the presence of contaminated soil and the handling of contaminated soil affects project earthwork costs.

In our opinion, site soil can be excavated using conventional excavating equipment such as backhoes, trackhoes or dozers.

As stated previously, portions of the Upper Sand and Gravel unit are moisture sensitive and will be difficult to work or compact if moisture contents are greater or less than the optimum moisture content by about 3 percentage points. Accordingly, earthwork during or after periods of wet weather should be avoided, if possible. If earthwork activities cause excessive subgrade disturbance, replacement with structural fill might be necessary.

Disturbance to a greater depth also should be expected when site preparation work is conducted during periods of wet weather, or if the soil moisture content is near saturation. Accordingly, if earthwork activities are performed during wet weather, we recommend that the project specifications and budget include provisions for removal of unsuitable material and importing and compacting additional structural fill.

5.6.3. Subgrade Preparation

Soil exposed at working subgrade following stripping and excavation should be compacted to a dense condition before placing structural fill. To that end, the upper 12 inches of soil present at working subgrade following excavation should be compacted to the following criteria:

- At least 90 percent of MDD based on the ASTM International (ASTM) D 1557 laboratory test procedure for soil more than 2 feet below finished pavement or hardscape subgrade.
- At least 95 percent of MDD for soil less than 2 feet below finished pavement and hardscape subgrades.
- At least 95 percent of the MDD for soil within the proposed building footprint (below foundations and floor slab).

A representative of GeoEngineers should evaluate soil conditions at working subgrade and within foundation excavations before placing structural fill, formwork or reinforcing steel. Evaluation of subgrade preparation should be accomplished through in-place density testing of the prepared areas. Alternatively, probing and proof-rolling may be used. The most appropriate method for evaluating subgrade preparation should be determined by the geotechnical engineer-of-record at the time earthwork is performed.

Areas identified as soft or unstable during subgrade preparation observations within hardscape, pavement and floor slab areas should be overexcavated to firm bearing, or a depth of at least 2 feet whichever is less, and replaced with suitable structural fill. Soft or unstable areas below foundations should be excavated to firm bearing and replaced with suitable structural fill.

If soil is still unstable at working subgrade following overexcavation within hardscape, pavement and floor slab areas, a stabilization fabric such as Mirafi 180N or equivalent should be placed on top of working subgrade before placing structural fill to establish final subgrade elevations.

5.6.4. Temporary Cut Slopes

In our opinion, excavations in the on-site soil are highly susceptible to sloughing and caving. Excavations deeper than 4 feet should be shored or sloped at stable inclinations if workers are required to enter such excavations. Shoring for excavations must conform to provisions of Title 296 Washington Administrative Code (WAC), Part N, “Excavation, Trenching and Shoring.”

In our opinion, site soil classifies as Type C for excavation purposes (Chapter 296-155-664 WAC). The maximum allowable temporary slope for Type C soil is 1.5H:1V (horizontal:vertical) for simple excavations less than 20 feet deep located above the groundwater table or seepage zone.

Temporary cut slope guidance assumes that all surface loads are kept a minimum distance of at least one-half the depth of the cut away from the top of the slope. Flatter slopes will be necessary if surface loads are imposed above the cuts a distance equal to or less than one-half the depth of the cut, or if seepage is present within cuts. It is the contractor’s responsibility to monitor and adjust the inclination of temporary excavated slopes and assure site safety during the proposed construction.

Alternatively, temporary shoring should be installed if space constraints limit the depth and/or inclination of cut slopes. Regardless of the soil type encountered in the excavation, shoring, trench boxes or sloped sidewalls will be required under Washington Industrial Safety and Health Administration (WISHA) regulations, as applicable.

While this report describes certain approaches to excavation, the contract documents should specify that the contractor is responsible for selecting excavation methods, monitoring the excavations for safety, reducing temporary slope inclinations to improve stability and providing shoring, as required, to protect personnel.

5.7. Structural Fill

Soil used as fill to support foundations, slab-on-grade floors, hardscape and paved areas is classified as structural fill for the purposes of this report. Structural fill material requirements vary depending upon its use as described below. Structural fill, whether on-site soil or imported, should be free of debris, organic material, frozen soil and particles larger than 4 inches in maximum dimension.

5.7.1. Use of On-Site Soil as Structural Fill

In our opinion, most of the on-site soil has the characteristics to be suitable for re-use as general structural fill. Portions of the on-site soil are moisture sensitive and will be difficult to properly work or compact during extended periods of wet weather.

5.7.2. Imported Structural Fill

Imported structural fill, where required, should meet the following criteria:

- General Structural Fill – Imported general structural fill placed below foundations, floor slabs (except for the capillary break layer), pavements and hardscape should consist of a well-graded sand or sand and gravel mixture with less than about 10 percent fines. The following gradations generally meet these criteria as described in the 2018 WSDOT *Standard Specifications for Road, Bridge and Municipal Construction* (Standard Specifications):
    - "Gravel Borrow" in Section 9-03.14(1).
    - "Select Borrow" in Section 9-03.14(2), with the added criteria of being well-graded.
    - "Foundation Material Class A and B" in Section 9-03.17.

  "Gravel Borrow" and "Select Borrow" will be suitable for use as structural fill during dry weather conditions only. If structural fill is placed during wet weather, the fines content of the structural fill should be less than 5 percent. Other gradations may be used if they meet the general criteria stated above and are approved by the Geotechnical Engineer-of-Record.
- Imported structural fill used as base course for pavements should consist of CSBC and CSTC meeting criteria in section 9-03.9(3) of the current WSDOT Standard Specifications.
- Imported structural fill placed as capillary break material below floor slabs should consist of 1½-inch-minus free-draining crushed gravel with negligible sand or silt. Material in conformance with "Section 9-03.1(4) C, Grading No. 57" of the WSDOT Standard Specifications generally meets these criteria. Alternative guidelines may be used if approved by the Geotechnical Engineer-of-Record.

5.7.3. Fill Placement and Compaction Criteria

Structural fill should be placed in loose lifts not exceeding 8 inches in thickness (or a thickness compatible with the compaction equipment used, not to exceed 12 inches) and mechanically compacted to a firm condition. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. We recommend structural fill be compacted to the following criteria based on the ASTM D 1557 laboratory test procedure:

- Soil used as structural fill placed within the proposed building areas, regardless of depth below floor subgrade or foundation grade, should be compacted to at least 95 percent of the previously mentioned MDD.
- Structural fill placed adjacent to and within a distance of 2.5D of foundation elements (where D is the embedded depth of the foundation element), which are designed to resist lateral loads should be compacted to at least 90 percent of the MDD.
- Structural fill placed adjacent to and within a distance of H of retaining walls (where H is the height of soil retained behind the wall), should be compacted in the range of 90 to 92 percent of the MDD, unless retained soil will support pavement or structures. Then structural fill should be compacted to meet criteria as outlined in this report. Care should be taken by the contractor not to overstress the walls during compaction. Compaction within 5 feet of the back of the walls should be limited to light-weight compaction equipment. This likely will require the lift thickness be reduced in order to achieve compaction criteria.
- Structural fill in roadway, parking areas and below exterior hardscapes, including utility trench backfill, should be compacted to at least 90 percent of the MDD, except the upper 2 feet of fill below final subgrade should be compacted to a minimum 95 percent of the MDD.
- Structural fill placed as capillary break for floor slabs and crushed rock base course for pavements should be compacted to at least 95 percent of the MDD.
- Non-structural fill, such as fill placed in landscaped areas, should be compacted to at least 85 percent of the MDD, with the exception that compaction should not exceed 85 percent for fill placed within stormwater swales. In areas intended for future development, a higher degree of compaction should be considered to reduce the settlement potential of the fill soil.
- Structural fill that consists of material too granular to test should be compacted using method or performance specifications, as determined by the geotechnical engineer of record. At a minimum, structural fill that is too granular to test should be compacted using at least 5 passes of a minimum 10-ton vibratory roller with a dynamic force of at least 30,000 pounds per impact vibration and at least 1,000 vibrations per minute.

We recommend a representative of GeoEngineers be on site during earthwork operations to observe site preparation and structural fill placement. Soil conditions should be evaluated by in-place density tests, visual evaluation, probing and proof-rolling of the structural fill and recompacted on-site soil, as it is prepared, to check for compliance with contract documents and recommendations in this report.

5.8. Weather Considerations

As stated previously, portions of the on-site soil are moisture sensitive. As the moisture content of the soil increases, the strength decreases. During wet weather, as the soil approaches saturation, it becomes soft and muddy. Performing earthwork in these conditions will lead to disturbance of near-surface soil. During dry weather, the on-site soil should be less susceptible to disturbance and provide better support for construction equipment. In addition, drying of soil that is above its optimum moisture content is most effective during extended periods of warm, dry weather.

The wet weather season generally begins in November and continues through May in eastern Washington. However, periods of wet weather may occur during any time of year. If wet weather earthwork is unavoidable, we recommend that the following steps be taken if surficial soil conditions begin to deteriorate:

- Stop earthwork activities during and immediately after periods of heavy precipitation.
- Grade the ground surface in and around the work area so that areas of ponded water do not develop, and water does not enter and collect in excavations and trenches.
- Accumulated water should be removed from the work area in accordance with the project Stormwater Pollution Prevention Plan (SWPPP).
- Areas of uncompacted soil should be sealed by rolling with a smooth-drum roller before precipitation occurs.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are not susceptible to disturbance.
- Construction activities should be scheduled so that the length of time that soil is exposed to moisture is reduced to the extent practical.

6.0 DESIGN REVIEW AND CONSTRUCTION SERVICES

The recommendations in this report are based on the previously stated assumptions and design information provided to us. We welcome the opportunity to discuss construction plans and specifications for this project as they are being developed. We believe GeoEngineers should be retained to review the geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in this report. Through our service to you on this project, we understand your project goals, objectives and preferences; the various assumptions that may have been made; and the many technical interrelationships involved. Consequently, we are more likely to recognize a problem for what it is, and to recommend the most effective solution.

GeoEngineers also maintains an accredited soil and material testing laboratory which allows us to provide special inspection and testing services in general accordance with the IBC and local building department requirements. Our services include inspection and/or testing of subgrade soil and structural fill placement and compaction.

7.0 LIMITATIONS

We have prepared this report for Spokane Portland and Seattle, LLC for the proposed 840 Building project in Spokane, Washington. Spokane Portland and Seattle, LLC may distribute copies of this report to their designated design and construction team members and their authorized agents and regulatory agencies as may be required for the project.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering and environmental science practices in this area at the time this report was prepared. The conclusions, recommendations, and opinions presented in this report are based on our professional knowledge, judgment and experience. No warranty or other conditions, express or implied, should be understood.

Please refer to Appendix C, titled "Report Limitations and Guidelines for Use," for additional information pertaining to use of this report.

8.0 REFERENCES

Derkey, R.E, et al. "Geologic Map of Spokane Northwest 7.5-minute Quadrangle, Spokane County, Washington". Washington State Department of Natural Resources, 2004.

GeoEngineers, Inc., "Revised Geotechnical Engineering Evaluation, Great Northern Spokane, 802 East Spokane Falls Boulevard, Spokane, Washington," GEI File No. 19614-001-00, November 10, 2010.

Green W.H and G.A. Ampt. 1911. "Studies on Soil Physics: I. Flow of Air and Water Through Soils." J. Argic Sci., Vol. 4, pgs 1-24.

Norton, Andy. "Spokane, Washington LiDAR, Technical Data Report". QSI Environmental report to Puget Sound Regional Council. 2015.

TABLES

Table 8

Hydraulic Conductivity and Drywell Outflow Rate Estimates

Proposed 840 Building

Spokane, Washington

								2019 Ecology SWMMEW Method								
								Equation 6.16			Equation 6.17 -Coase Grained Soil			Equation 6.18 - Fined Grained Soil		
Exploration ID	Sample Depth (ft)	Approximate Elevation (ft)	Soil Type	D10 (mm)	D60 (mm)	D90 (mm)	% fines	Log10(Ksat)	Ksat Initial (cm/sec)	Ksat Initial (in/hr)	Log10(Ksat)	Ksat Initial (cm/sec)	Ksat Initial (in/hr)	Log10(Ksat)	Ksat Initial (cm/sec)	Ksat Initial (in/hr)
B-2	3.5-5	1882.0	SW-SM	0.25	3.0	8	6.8	-1.29544	0.05	72	-1.0	*0.09*	*129*	-2.328844	0.00	6.65
B-5	3.5-5	1881.0	GW-GM	0.20	16.0	40	7.9	-1.63432	0.02	33	-1.1	*0.08*	*111*	-8.596782	0.00	0.00
B-4	3.5-5	1880.5	GW-GM	0.10	7.0	20	9.2	-1.72636	0.02	27	-1.2	*0.06*	*83*	-5.208936	0.00	0.01
B-3	6-7.5	1878.5	SP-SM	-	-	-	8.5									
B-2	8.5-10	1877.0	SW-SM	0.10	7.0	25	9.1	-1.78928	0.02	23	-1.2	*0.06*	*83*	-5.059078	0.00	0.01
B-4	8.5-10	1875.5	SW-SM	0.15	6.5	20	7	-1.5931	0.03	36	-1.2	*0.07*	*97*	-4.57006	0.00	0.04
B-5	11-12.5	1873.5	GW	0.35	8.0	25	4.3	-1.19944	0.06	90	-0.9	*0.12*	*175*	-3.700394	0.00	0.28

								Spokane Regional Stormwater Manual - Spokane 200 Method										
											Single-Depth			72-Inch Diameter Drywell			Low-Profile Drywell		
Exploration ID	Sample Depth (ft)	Approximate Elevation (ft)	Soil Type	D10 (mm)	D60 (mm)	D90 (mm)	% fines	Ksat (cm/sec)	Ksat (in/hr)	Normalized Q (cfs/ft)	SF Dim.	Q_a (cfs/ft)	Q_D[1] (cfs)	SF Dim.	Q_a (cfs)	Q_D[2] (cfs)	SF Dim.	Q_a (cfs)	Q_D[3] (cfs)
B-2	3.5-5	1882.0	SW-SM	0.25	3.0	8	6.8	0.017	25	0.048	2	0.024	0.14	1.8	0.03	0.096	1.8	0.027	0.048
B-5	3.5-5	1881.0	GW-GM	0.20	16.0	40	7.9	0.013	19	0.038	2	0.019	0.11	1.8	0.02	0.075	1.8	0.021	0.038
B-4	3.5-5	1880.5	GW-GM	0.10	7.0	20	9.2	0.010	14	0.029	2.3	0.013	0.08	2.1	0.01	0.051	2.1	0.014	0.026
B-3	6-7.5	1878.5	SP-SM	-	-	-	8.5	0.011	16	0.033	2.3	0.014	0.09	2.1	0.02	0.058	2.1	0.016	0.029
B-2	8.5-10	1877.0	SW-SM	0.10	7.0	25	9.1	0.010	14	0.030	2.3	0.013	0.08	2.1	0.01	0.052	2.1	0.014	0.026
B-4	8.5-10	1875.5	SW-SM	0.15	6.5	20	7	0.016	23	0.046	2	0.023	0.14	1.8	0.03	0.091	1.8	0.025	0.046
B-5	11-12.5	1873.5	GW	0.35	8.0	25	4.3	0.041	58	0.100	1.3	0.077	0.46	1.1	0.09	0.309	1.1	0.091	0.154

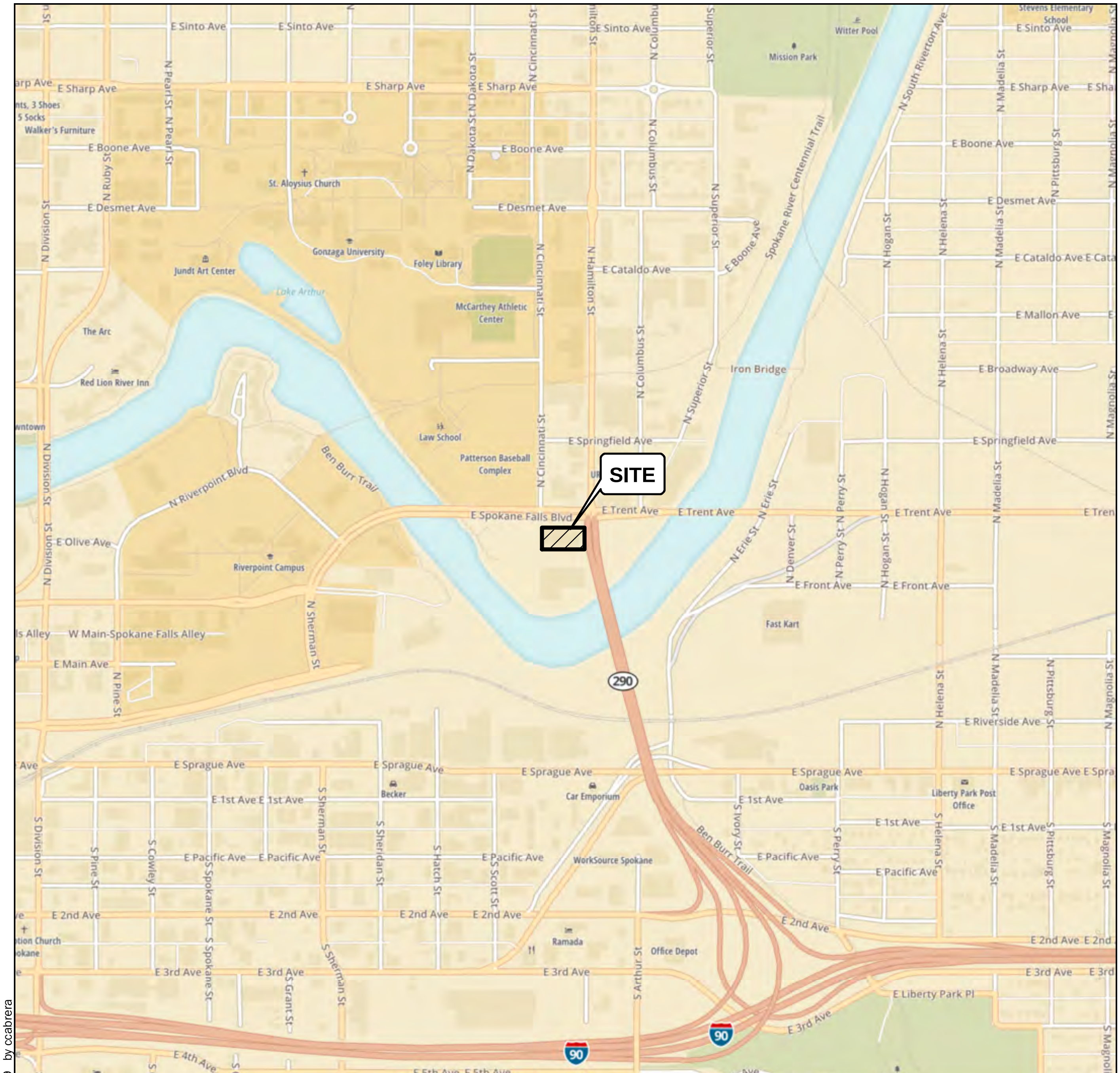
Notes:

[1]City of Spokane Standard Type 1 Drywell or Spokane County Standard Type A Drywell

[2]72-Inch-Diameter Drywell with Flat Lid (Wilbert Precast Product No. 720DW)

[3]500-Gallon Low-Profile Drywell (Wilbert Precast Product No. 1652)

FIGURES



N
W E
S

1,000 0 1,000
Feet

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 11N

Vicinity Map

Proposed 840 Building
Spokane, Washington

GeoEngineers

Figure 1

P:\18\18324004\GIS\MXD\1832400400_F01_VM.mxd Date Exported: 12/06/19 by ccabrera

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: ESRI

Projection: NAD 1983 UTM Zone 11N

Legend

- Boring Number and Approximate Location (GeoEngineers, 2019) (Approximate elevation of bottom of upper sand and gravel unit)
- Monitoring Well Number and Approximate Location (Hart Crowser, 1991)
- Pilot Test Borehole Number and Approximate Location (GeoEngineers, 2010)
- Boring Number and Approximate Location (GeoEngineers, 2010)
- Proposed Building Approximate Location

N
W E
S

40 0 40
Feet

Site Plan

Proposed 840 Building
Spokane, Washington

GeoEngineers

Figure 2

P:\18\18324004\GIS\MXD\1832400400_F02_SP.mxd Date Exported: 12/09/19 by ccabrera

APPENDICES

APPENDIX A
Field Methods, Boring Logs and Geotechnical Laboratory Testing

APPENDIX A
FIELD METHODS, BORING LOGS AND GEOTECHNICAL LABORATORY TESTING

General

We explored soil and groundwater conditions at the site on November 26 and 27, 2019 by drilling six borings (B-1 through B-6) at the approximate locations shown in the Site Plan, Figure 2. The borings were advanced using a truck-mounted CME 75 hollow-stem auger drill rig owned and operated by GeoEngineers.

General Soil Sampling Procedures

Soil samples were obtained from the borings at approximate 2½- to 5-foot-depth intervals using 2-inch, outside-diameter standard split-spoon samplers and 2.4-inch, inside-diameter California-style split-barrel samplers. The samplers were driven into the ground using a 140-pound automatic hammer, falling 30 inches on each blow. The number of blows required to drive the samplers each of three, 6-inch increments of penetration were recorded in the field. The sum of the blow counts for the last two, 6-inch increments of penetration is reported on the boring logs, unless otherwise indicated. The blow counts for the 2-inch, outside-diameter split-spoon sampler are reported as the standard penetration test (SPT) N-value, unless otherwise noted. The approximate N-values for the California-style samplers also are reported on the boring logs under the “Remarks” section at the respective sample depths. The conversion of California-style sampler penetration resistant to approximate SPT N-values was made using the Lacroix-Horn equation (ASTM SPT-523, 1973). Sampling equipment was decontaminated between each sampling event using a combination of Liquinox and distilled water. Portions of select soil samples also were placed in 4-ounce jars and returned to GeoEngineers office for temporary refrigerated storage.

The explorations were continuously monitored by a representative from GeoEngineers who classified the soil encountered, maintained detailed logs of the borings showing stratigraphic changes and other pertinent information, obtained representative soil samples, and observed groundwater conditions. Soil encountered in the borings was classified in the field in general accordance with ASTM D 2488, the Standard Practice for the Classification of Soils (Visual-Manual Procedure), which is described in Figure A-1, Key to Exploration Logs. Logs of the borings are presented in Figures A-2 through A-7, Logs of Borings. The logs are based on interpretation of the field and laboratory data and indicate the depth at which subsurface materials or their characteristics change, although these changes might actually be gradual.

The boring locations were established in the field using a hand-held global positioning (GPS) device and by taping from existing site features. Ground surface elevations at boring locations were obtained from publicly available LiDAR data. Elevations are based on the North American Vertical Datum of 1988 (NAVD 88). The locations and elevations shown on the logs should be considered accurate to the degree implied by the method used.

Field Screening of Soil Samples

The GeoEngineers’ representative performed field screening of soil samples obtained during drilling activities. Field screening results are used as a general guideline to delineate depths with possible petroleum-related contamination. The screening methods used include: (1) visual screening; (2) water sheen screening; and (3) headspace vapor screening using a MiniRae photoionization detector (PID) calibrated to isobutylene.

Visual screening consists of inspecting the soil for stains indicative of contamination. Visual screening is generally more effective when contamination is related to heavy petroleum hydrocarbons such as motor oil, or when hydrocarbon concentrations are high.

Water sheen screening is a more sensitive method that has been effective in evaluating whether hydrocarbon concentrations are less than regulatory cleanup guidelines. Water sheen screening involves placing soil in water and observing the water surface for signs of sheen. Sheen screening might detect both volatile and nonvolatile petroleum hydrocarbons. Sheen classifications are as follows:

No Sheen	No visible sheen on water surface.
Slight Sheen	Light, colorless, dull sheen; spread is irregular, not rapid; sheen dissipates rapidly. Natural organic matter in the soil might produce a slight sheen.
Moderate Sheen	Light to heavy sheen; might have some color/iridescence; spread is irregular to flowing, might be rapid; few remaining areas of no sheen on water surface.
Heavy Sheen	Heavy sheen with color/iridescence; spread is rapid; entire water surface might be covered with sheen.

Headspace vapor screening involved placing a soil sample in a plastic sample bag. Air was captured in the bag, and the bag was shaken to expose the soil to the air trapped in the bag. The probe of the PID was then inserted into the bag to measure volatile organic compounds (VOCs) in the air within the bag. In this application, the PID measured concentration of organic vapors ionizable by a 10.6 electron volt (eV) lamp in the range between 1.0 and 2,000 parts per million [ppm]), with a resolution of +/- 2 ppm.

Field screening results are site-specific. The effectiveness of field screening results will vary with temperature, moisture content, organic content, soil type and type and age of contaminant. The presence or absence of a sheen or headspace vapors does not necessarily indicate the presence or absence of petroleum hydrocarbons.

Results of the field screening are shown on the boring logs as the respective screening depths. Results of the field screening did not indicate the presence of petroleum contamination.

Geotechnical Laboratory Testing

Soil samples obtained from the borings were returned to our laboratory for further examination and testing. Representative soil samples were selected for geotechnical laboratory tests to evaluate geotechnical engineering characteristics of the site soil and to confirm or revise our field classifications. The laboratory testing program was completed in general accordance with applicable ASTM standards and is summarized in Table A-1, Summary of Laboratory Testing.

TABLE A-1. SUMMARY OF LABORATORY TESTING

Standard Test Method for:	Test Method Designation	Total Tests Performed	Results Location
Minus 200 Washes	ASTM D 1140	2	Percent fines presented on boring logs at respective sample depths.
Grain size analyses	ASTM C 136	7	Presented on Figures A-8 and A-9. Percent fines also presented on boring logs at respective sample depths.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	SOD	Sod/Forest Duff
	TS	Topsoil

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

- 2.4-inch I.D. split barrel
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- Direct-Push
- Bulk or grab
- Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

Key to Exploration Logs

GeoEngineers

Figure A-1

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	11/26/2019	11/26/2019	20	JML	DRL	GeoEngineers, Inc.	Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	1885.4 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Truck mounted CME 75
Latitude / Longitude	47.66122 / -117.39675	System Datum	WGS84	Groundwater: Date Measured 11/26/2019; Depth to Water (ft) 15.50; Elevation (ft) 1869.90	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Sheen	Headspace Vapor (ppm)	REMARKS
1885	0								GP-GM	Gray fine to coarse gravel with silt, sand and occasional cobbles (very dense, moist) (fill?)			
			8	52		S-1					NS	<1	
									GM	Dark brown silty fine to coarse gravel with sand (medium dense, moist) (fill?)			
			12	41		S-2					NS	<1	Approximate SPT N Value = 17
									SM	Brown silty fine to coarse sand with occasional gravel (medium dense, moist) (fill?)	NS	<1	
1880	5												
			6	11		S-3			SP-SM	Brown fine to coarse sand with silt and gravel (medium dense, moist) (fill?)	NS	<1	
									GP-GM	Brown fine to coarse gravel with silt, sand, cobbles and occasional boulders (medium dense to dense, moist)			
			18	72		S-4					NS	<1	Approximate SPT N Value = 30
1875	10												
			9	61		S-5					NS	<1	
1870	15						∇			Becomes wet			
			10	41		S-6					NS	<1	
	20												

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Date:12/11/19 Path:\\GEOENGINEERS.COM\WAN\PROJECTS\18\18324004\GINT\1832400400.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_ENVIRONMENTAL_STANDARD

Log of Boring B-1

GeoEngineers

Project: Proposed 840 Building
Project Location: Spokane, Washington
Project Number: 18324-004-00

Drilled	Start 11/26/2019	End 11/26/2019	Total Depth (ft) 19.5	Logged By JML Checked By DRL	Driller GeoEngineers, Inc.
Drilling Method	Hollow-stem Auger				
Surface Elevation (ft) Vertical Datum	1885.8 NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Truck mounted CME 75
Latitude Longitude	47.661232 -117.397805	System Datum	WGS84	Groundwater Date Measured 11/26/2019	Depth to Water (ft) 16.60; Elevation (ft) 1869.20

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Sheen	Headspace Vapor (ppm)	REMARKS
1885	0								AC	Approximately 3 inches of asphalt concrete pavement			
			12	8		S-1			SW-SM	Brown fine to coarse sand with silt and gravel (loose, moist) (fill?)	NS	<1	
			18	13		S-2 SA					NS	<1	Approximate SPT N Value = 5 %F=6.8
1880	5												
			10	5		S-3							
			16	16		S-4 SA					NS	<1	Approximate SPT N Value = 7 %F=9.1
1875	10								GP	Gray-brown fine to coarse gravel with sand, cobbles, trace silt and occasional boulders (very dense, moist)			
			10	55		S-5					NS	<1	
1870	15												
							▽			Becomes wet			
			11	50/5"		S-6					NS	<1	

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Date:12/11/19 Path:\\GEOENGINEERS.COM\WAN\PROJECTS\18\18324004\GINT\1832400400.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_ENVIRONMENTAL_STANDARD

Log of Boring B-2

GEOENGINEERS

Project: Proposed 840 Building
Project Location: Spokane, Washington
Project Number: 18324-004-00

Drilled	Start 11/26/2019	End 11/26/2019	Total Depth (ft)	20	Logged By JML / Checked By DRL	Driller GeoEngineers, Inc.	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	1884.4 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Truck mounted CME 75
Latitude / Longitude	47.661664 / -117.397737	System Datum	WGS84	Groundwater Date Measured: 11/26/2019; Depth to Water (ft): 16.80; Elevation (ft): 1867.60	

Notes:

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Sheen	Headspace Vapor (ppm)	REMARKS
	0					AC	Approximately 2 inches asphalt concrete pavement			
		16	27	S-1 / SA		SM	Brown silty fine to coarse sand with gravel (loose to medium dense, moist) (fill?)	NS	<1	Approximate SPT N Value = 11 %F=13.0
1880		6	9	S-2				NS	<1	
	5									
		12	22	S-3 / %F		GP-GM	Brown fine to coarse gravel with silt and sand (loose, moist) (fill?)	NS	<1	Approximate SPT N Value = 9 %F=8.5
1875		10	10	S-4		SP-SM	Brown fine to medium sand with silt and occasional gravel (medium dense, moist) (fill?)	NS	<1	
	10					GP	Gray-brown fine to coarse gravel with sand, cobbles, trace silt and occasional boulders (dense to very dense, moist)			
1870		18	150	S-5				NS	<1	Approximate SPT N Value = 62
	15									
					▽		Becomes wet			
1865		12	43	S-6				NS	<1	
	20									

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Date:12/11/19 Path:\\GEOENGINEERS.COM\WAN\PROJECTS\18\18324004\GINT\1832400400.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_ENVIRONMENTAL_STANDARD

Log of Boring B-3

GeoEngineers

Project: Proposed 840 Building
Project Location: Spokane, Washington
Project Number: 18324-004-00

Drilled	Start 11/26/2019	End 11/26/2019	Total Depth (ft)	20	Logged By JML, Checked By DRL	Driller	GeoEngineers, Inc.	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	1884.4 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Truck mounted CME 75
Latitude / Longitude	47.66166 / -117.39729	System Datum	WGS84	Groundwater: Date Measured 11/26/2019; Depth to Water (ft) 15.55; Elevation (ft) 1868.85	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Sheen	Headspace Vapor (ppm)	REMARKS
	0								AC	Approximately 2 inches of asphalt concrete pavement			
			12	12		S-1			SP-SM	Brown fine to coarse sand with silt and occasional gravel (medium dense, moist) (fill?)	NS	<1	
1880			18	15		S-2 SA			GW-GM	Dark brown fine to coarse gravel with silt and sand (loose to medium dense, moist) (fill?)	NS	<1	Approximate SPT N Value = 6 %F=9.2
	5		3	11		S-3					NS	<1	
1875			18	34		S-4 SA			SW-SM	Brown fine to coarse sand with silt and gravel (medium dense, moist) (fill?)	NS	<1	Approximate SPT N Value = 14 %F=7.0
	10								GP	Gray-brown fine to coarse gravel with sand, cobbles, trace silt and occasional boulders (dense, moist)			
1870			12	56		S-5					NS	<1	
	15						▽			Becomes wet			
1865			12	71		S-6					NS	<1	
	20												

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Date:12/11/19 Path:\\GEOENGINEERS.COM\WAN\PROJECTS\18\18324004\GINT\1832400400.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_ENVIRONMENTAL_STANDARD

Log of Boring B-4

GEOENGINEERS

Project: Proposed 840 Building
Project Location: Spokane, Washington
Project Number: 18324-004-00

Figure A-5
Sheet 1 of 1

Drilled	Start 11/27/2019	End 11/27/2019	Total Depth (ft)	25	Logged By JML	Checked By DRL	Driller	GeoEngineers, Inc.	Drilling Method	Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	1885.8 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Truck mounted CME 75
Latitude / Longitude	47.66115 / -117.397177	System Datum	WGS84	Groundwater Date Measured	Depth to Water (ft) / Elevation (ft)

Notes: Unable to measure groundwater during drilling

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Sheen	Headspace Vapor (ppm)	REMARKS
1885	0								AC	Approximately 2 inches of asphalt concrete pavement			
			13	15		S-1			SP-SM	Dark brown fine to coarse sand with silt and gravel (medium dense, moist) (fill?)	NS	<1	
			18	38		S-2 SA			GW-GM	Brown-gray fine to coarse gravel with silt and sand (medium dense, moist) (fill?)	NS	<1	Approximate SPT N Value = 16 %F=7.9
1880	5									Grades to black			
			8	11		S-3			GP-GM	Brown fine to coarse gravel with silt and sand (medium dense, moist) (fill?)	NS	<1	
			16	90		S-4			GP	Gray-brown fine to coarse gravel with sand, cobbles, trace silt and occasional boulders (dense, moist)	NS	<1	Approximate SPT N Value = 37
1875	10												
			16	96		S-5 SA					NS	<1	Approximate SPT N Value = 40 %F=4.3
			16	115		S-6					NS	<1	Approximate SPT N Value = 47
1870	15												
										Becomes wet			
			10	39		S-7					NS	<1	
1865	20												
										Grades with more cobbles			
			18	80		S-8							
	25												

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

Date:12/11/19 Path:\\GEOENGINEERS.COM\WAN\PROJECTS\18\18324004\GINT\1832400400.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_ENVIRONMENTAL_STANDARD

Log of Boring B-5

GEOENGINEERS

Project: Proposed 840 Building
Project Location: Spokane, Washington
Project Number: 18324-004-00

Figure A-6
Sheet 1 of 1

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	11/27/2019	11/27/2019	30	JML	DRL	GeoEngineers, Inc.	Hollow-stem Auger

Surface Elevation (ft) / Vertical Datum	1885.5 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	Truck mounted CME 75
Latitude / Longitude	47.661604 / -117.396817	System Datum	WGS84	Groundwater Date Measured: 11/27/2019	Depth to Water (ft): 16.25; Elevation (ft): 1869.25

Notes:

FIELD DATA

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Sheen	Headspace Vapor (ppm)	REMARKS
1885	0								AC	Approximately 1½ inches of asphalt concrete pavement			
			16	45		S-1 / %F			SP-SM	Brown fine to coarse sand with silt, gravel and occasional cobbles (loose to medium dense, moist) (fill?)	NS	<1	Approximate SPT N Value = 19 %F=6.2
			12	8		S-2					NS	<1	
1880	5												
			1	33		S-3			GP-GM	Brown fine to coarse gravel with silt and sand (medium dense, moist) (fill?)	NS	<1	Approximate SPT N Value = 14
									GP-GM	Gray-brown fine to coarse gravel with silt and sand (medium dense, moist)			
			13	25		S-4					NS	<1	
1875	10								GP	Gray-brown fine to coarse gravel with sand, cobbles, trace silt and occasional boulders (medium dense to very dense, moist)			
			16	216/5"*		S-5							*Refusal on apparent cobble, blowcount overstated
			12	80**		S-6							**Rock stuck in sampler shoe, blow count overstated
1870	15												
							▽			Becomes wet			
			12	18		S-7							
1865	20												
			11	79		S-8					NS	<1	
1860	25												
			4	20		S-9							
	30												

Note: See Figure A-1 for explanation of symbols.
Coordinates Data Source: Horizontal approximated based on . Vertical approximated based on .

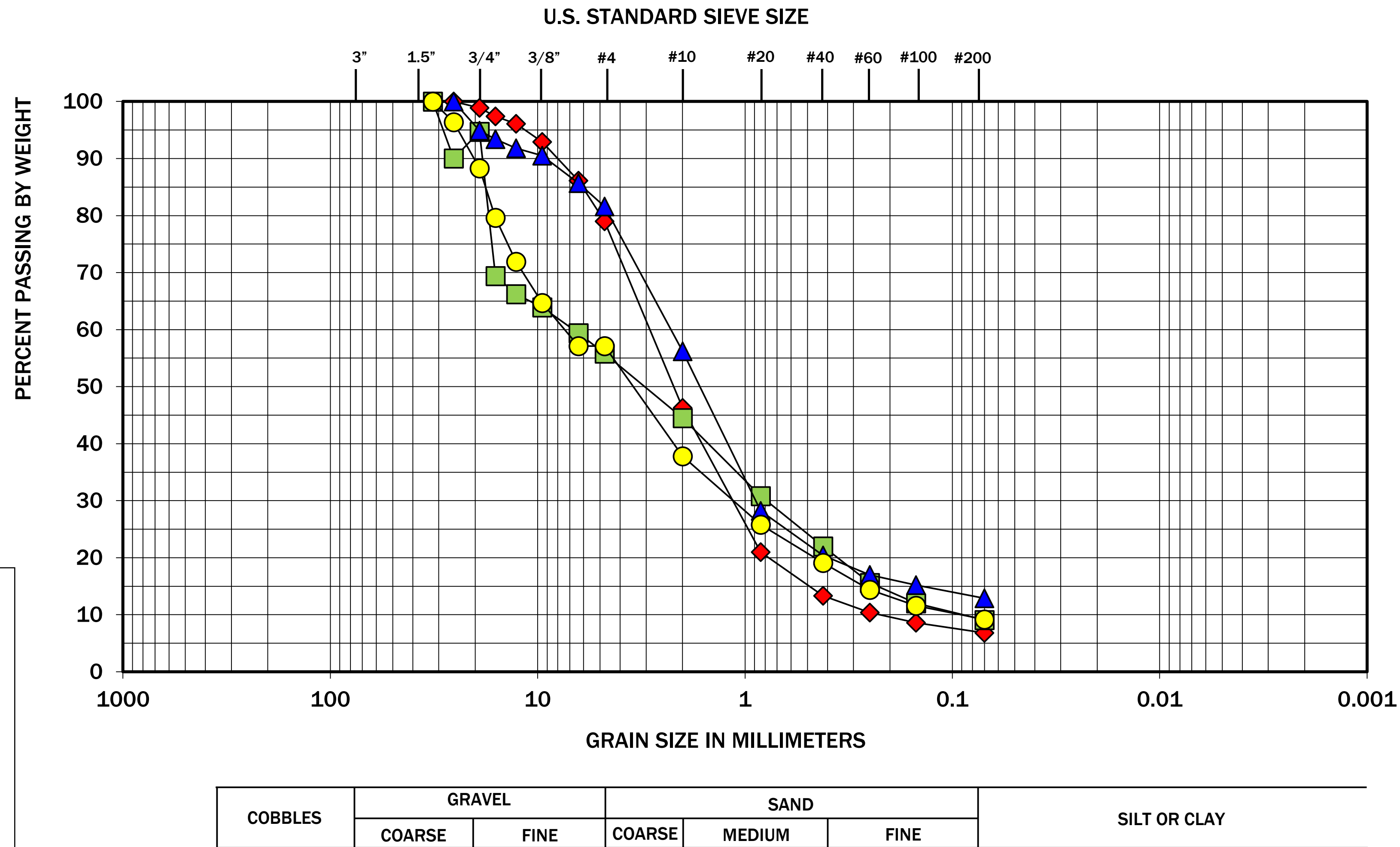
Date:12/11/19 Path:\\GEOENGINEERS.COM\WAN\PROJECTS\18\18324004\GINT\1832400400.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_ENVIRONMENTAL_STANDARD

Log of Boring B-6

GEOENGINEERS

Project: Proposed 840 Building
Project Location: Spokane, Washington
Project Number: 18324-004-00

Figure A-7
Sheet 1 of 1



Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆ (red diamond)	B-2	3½	5	Fine to coarse sand with silt and gravel
■ (green square)	B-2	8½	6	Fine to coarse sand with silt and gravel
▲ (blue triangle)	B-3	1	6	Silty sand with gravel
● (yellow circle)	B-4	3½	7	Fine to coarse gravel with silt and sand

Note: This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc. Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations, or generated by separate operations or processes.

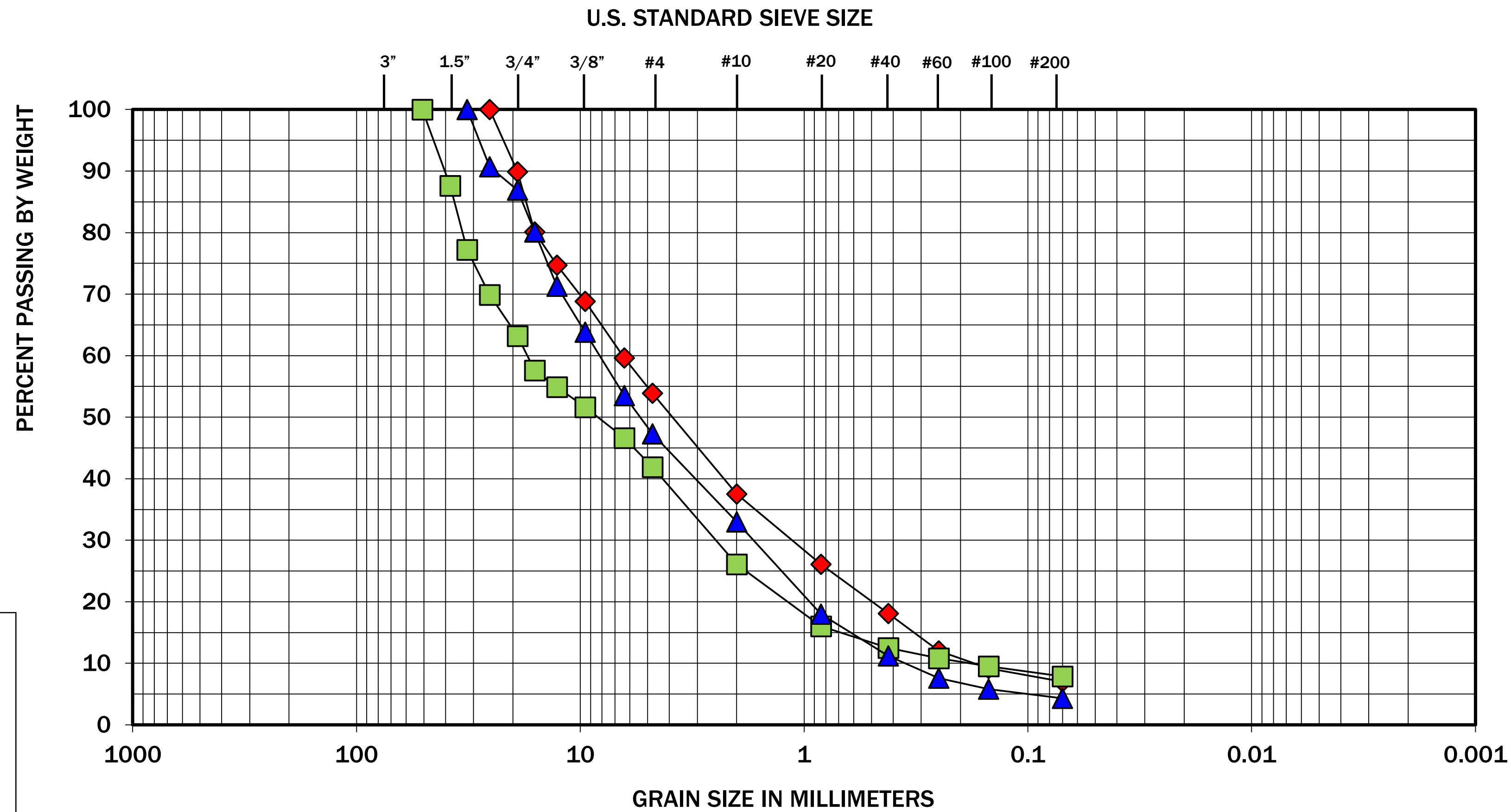
The grain size analysis results were obtained in general accordance with ASTM D 6913.

Sieve Analysis Results

Proposed 840 Building
Spokane, Washington

GEOENGINEERS

Figure A-8



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

Symbol	Boring Number	Depth (feet)	Moisture (%)	Soil Description
◆	B-4	8½	5	Fine to coarse sand with silt and gravel
■	B-5	3½	5	Fine to Coarse gravel with silt and sand
▲	B-5	11	3	Fine to coarse gravel with sand and trace silt

Note: This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc. Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations, or generated by separate operations or processes.

The grain size analysis results were obtained in general accordance with ASTM D 6913.

Sieve Analysis Results

Proposed 840 Building
Spokane, Washington

GeoEngineers

Figure A-9

APPENDIX B
Logs of Previous Explorations

Drilled	Start 7/29/2010	End	Total Depth (ft) 20	Logged By MP / Checked By TAD	Driller GeoEngineers, Inc.	Drilling Method Hollow-Stem Auger

Surface Elevation (ft) / Vertical Datum	1896.0 / City of Spokane	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop
Drilling Equipment	CME-75		
Easting (X) / Northing (Y)		System / Datum	
Notes:			

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
7/29/2010	13.5	

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
1895	0								AC	Approximately 1 inch asphalt concrete pavement			
									SP-SM	Dark brown medium to coarse sand with silt, fine gravel and trace organic matter (medium dense, moist) (upper sand)			
			8	11		1					12		SA; %F=11
1890	5												
									GW	Brown fine gravel with sand and trace silt (very dense, moist) (lower gravel)			
			15	50		2							SA; %F=4
1885	10												
			16	56		3	∇			Grades wet			
1880	15												
			18	73		4							SPT N-Value = 30
	20												

Notes: Please refer to Figure B-1 for an explanation of symbols.

Log of Boring B-100

GEOENGINEERS

Project: Great Northern Spokane

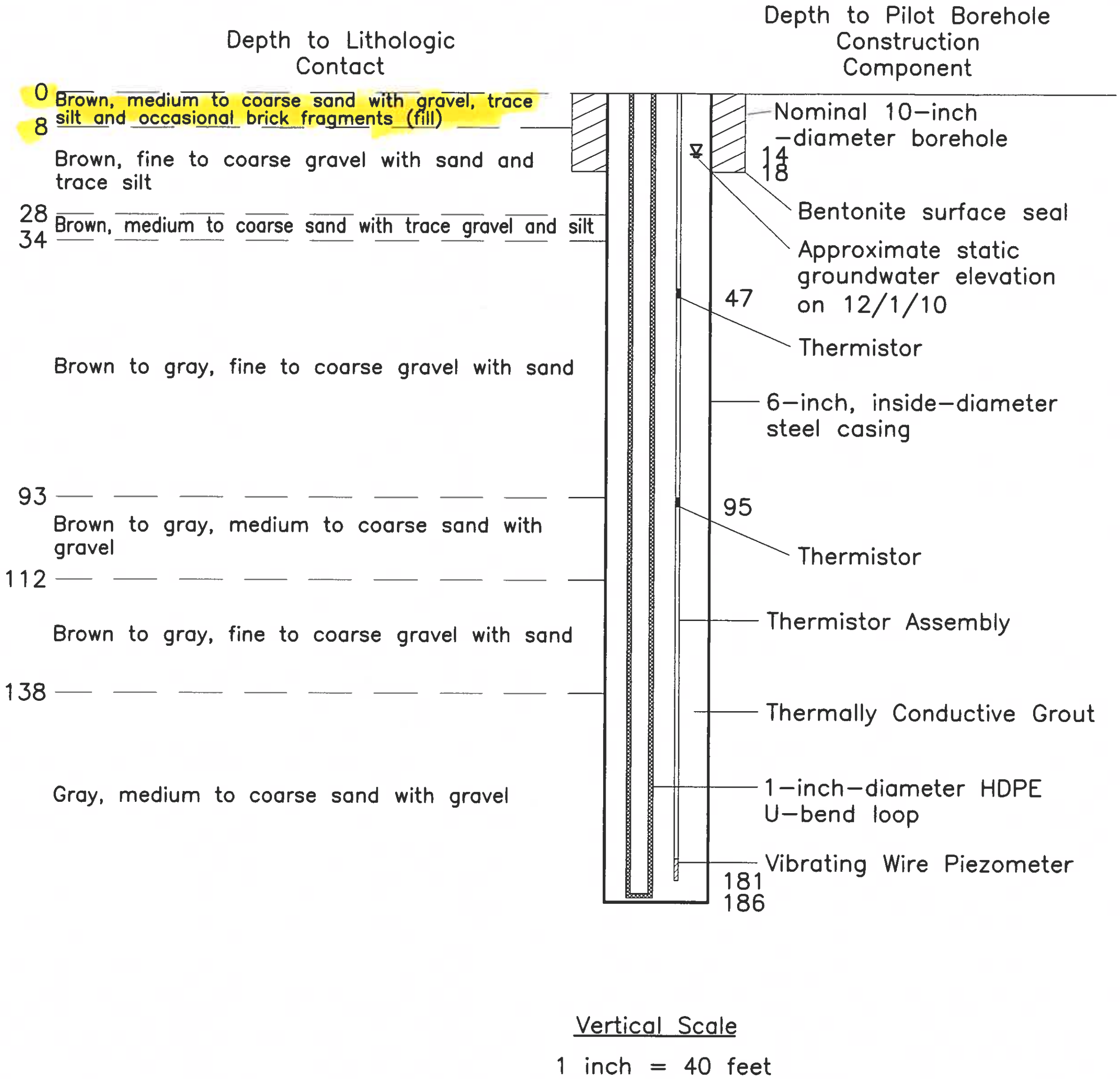
Project Location: Spokane, Washington

Project Number: 19614-001-00

Figure B-2

Sheet 1 of 1

Spokane: Date:12/12/19 Path:P:\19\19614001\GINT\1961400 1.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_STANDARD



Notes:
1. Depths are in feet and referenced to existing grade.

Pilot Test Borehole As-Built Diagram

Great Northern Project
Spokane, Washington

GEOENGINEERS

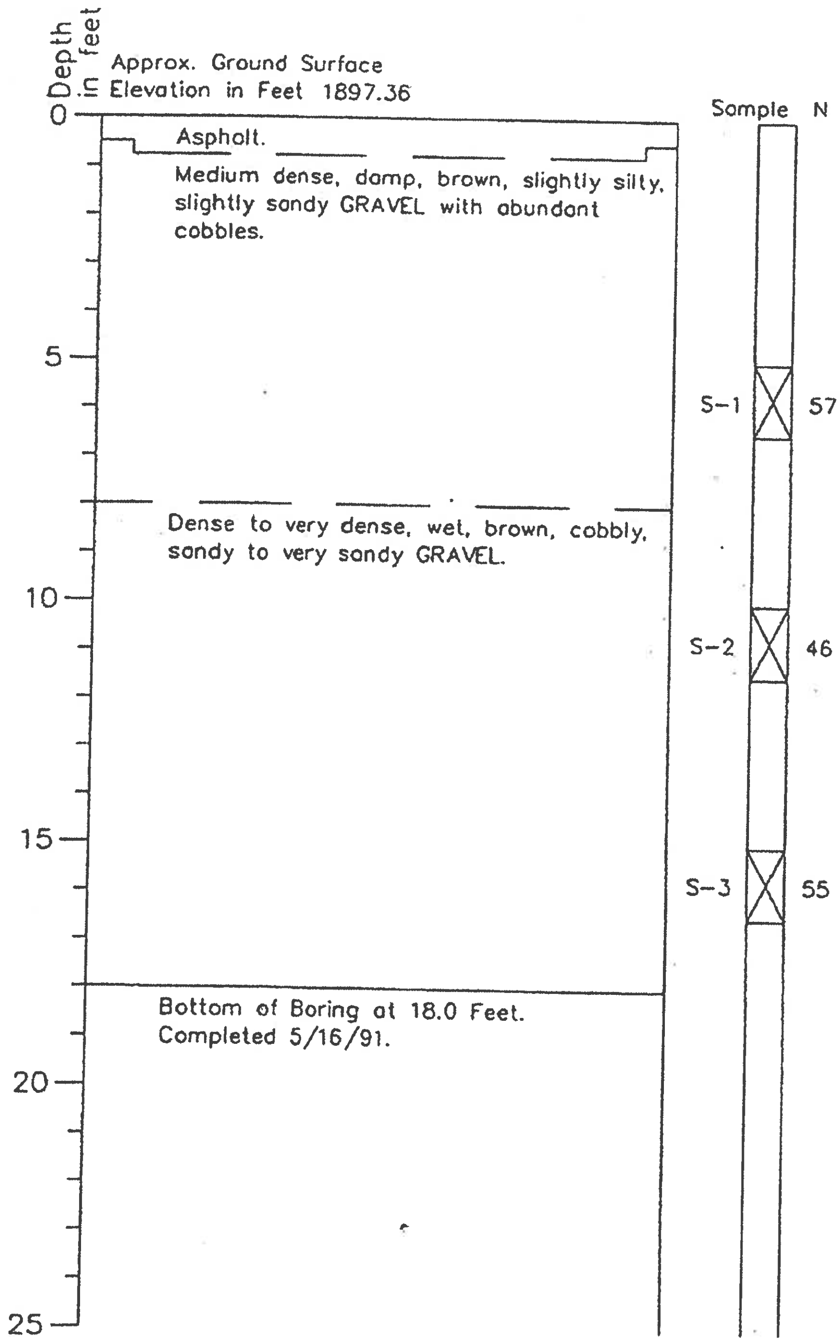
Figure X

OFFICE:SPOK PM JER \\spo\projects\19\19614001\Cadd\1961400100FX.dwg\TAB:Layout2 modified by jesmith on Dec 29, 2010 - 13:48

Boring Log and Construction Data for Monitoring Well MW-4

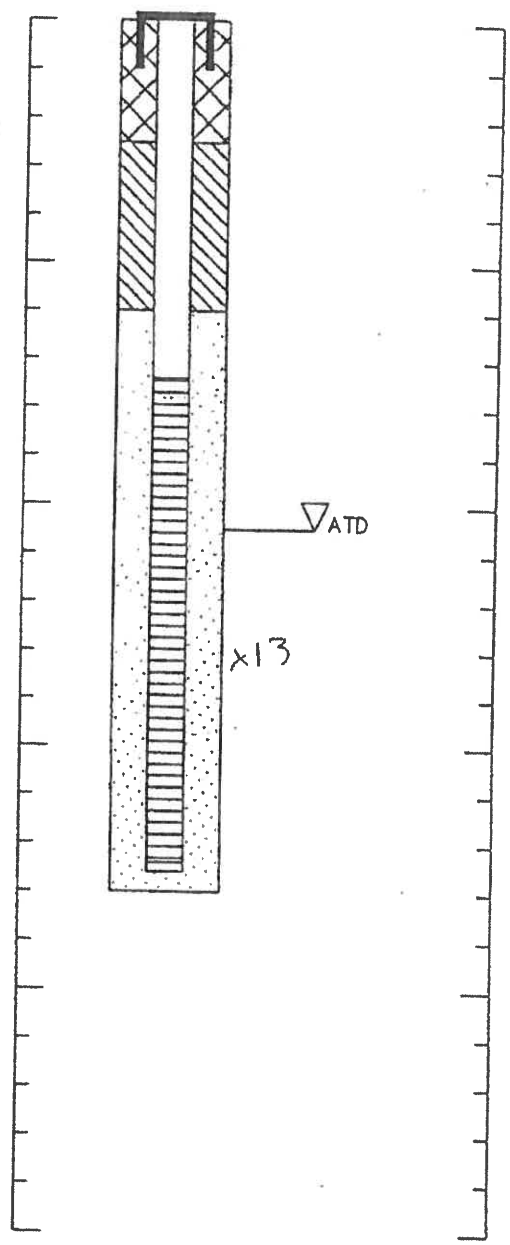
Geologic Log

Approx. Ground Surface Elevation in Feet 1897.36



Monitoring Well Design

Casing Stickup in Feet 0
Top of PVC in Feet 1897.36

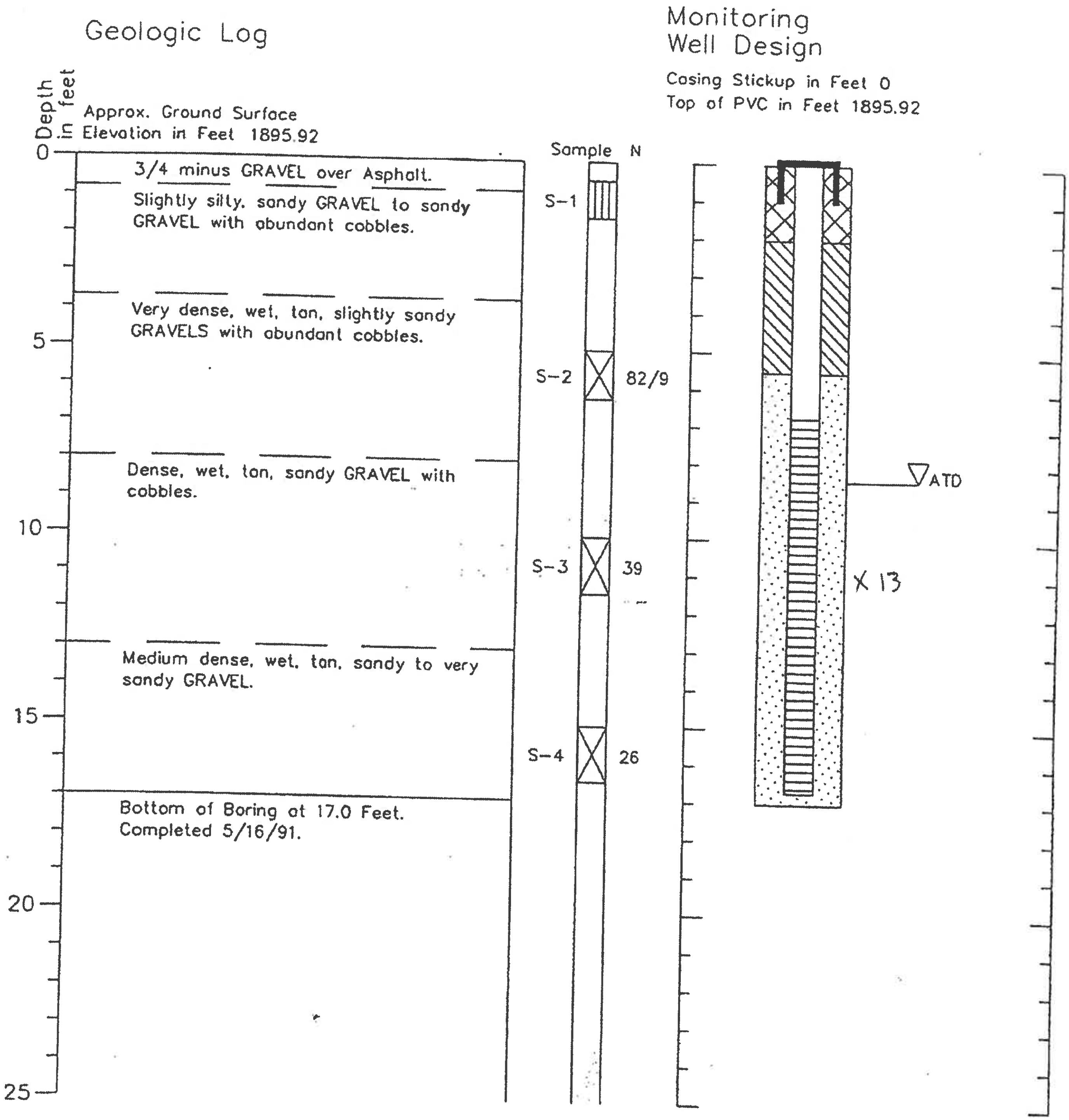


1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER
J-3148-01 5/91

Figure A-2

Boring Log and Construction Data for Monitoring Well MW-5



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Ground water level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

J-3148-01 5/91

Figure A-3

APPENDIX C
Report Limitations and Guidelines for Use

APPENDIX C
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory "limitations" provisions in its reports. Please confer with GeoEngineers if you need to know more how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical and Environmental Services are Performed for Specific Purposes, Persons and Projects

This report has been prepared for Spokane Portland and Seattle, LLC for the project specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the project, and its schedule and budget, GeoEngineers' services have been executed in accordance with our proposal dated November 5, 2019 and generally accepted geotechnical practices in this area at the time this report was prepared. GeoEngineers does not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

A Geotechnical Engineering or Geologic Report is based on a Unique Set of Project-Specific Factors

This report has been prepared for the proposed 840 Building located at 840 East Spokane Falls Boulevard in Spokane, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;

[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

- elevation, configuration, location, orientation or weight of the proposed structure;
- composition of the design team; or
- project ownership.

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, GeoEngineers can provide written modifications or confirmation, as appropriate.

Environmental Concerns are Not Covered

Unless environmental services were specifically included in GeoEngineers' scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Geotechnical, Geologic and Most Environmental Findings are Professional Opinions

GeoEngineers' interpretations of subsurface conditions are based on field observations and chemical analytical data from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. GeoEngineers' report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

Report Recommendations are Not Final

GeoEngineers has developed the following recommendations based on data gathered from subsurface investigation(s). These investigations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if GeoEngineers does not perform construction observation.

GeoEngineers recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions. If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by constructors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable, but separating logs from the report can create a risk of misinterpretation.

Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- encourages contractors to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer.

Contractors are Responsible for Site Safety on Their Own Construction Projects

GeoEngineers' geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialty.