Geotechnical Engineering Evaluation

Proposed McKinstry Warehouse Development
Lots 3 through 5
Spokane, Washington

for
McKinstry

June 19, 2020

GeoEngineers
Earth Science + Technology

Geotechnical Engineering Evaluation

Proposed McKinstry Warehouse Development
Lots 3 through 5
Spokane, Washington

for
McKinstry

June 19, 2020

GeoEngineers

523 East Second Avenue
Spokane, Washington 99202
509.363.3125

Geotechnical Engineering Evaluation

Proposed McKinstry Warehouse Development
Lots 3 through 5
Spokane, Washington

File No. 18324-005-01

June 19, 2020

Prepared for:

McKinstry
850 East Spokane Falls Blvd, Suite 100
Spokane, Washington 99202

Attention: Allie Teplicky, Real Estate Development Manager

Prepared by:

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

DAVID RYAN LAUDER
STATE OF WASHINGTON
42304
REGISTERED
PROFESSIONAL ENGINEER
6/19/20

David R. Lauder, PE
Senior Engineer

Teresa A. Dugger, PE
Associate

DRL:TAD:tt

GEOENGINEERS

Table of Contents

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 Warehouse development located west of Flint Road in Spokane, Washington. The approximate location of the project site is shown in the Vicinity Map, Figure 1. GeoEngineers previously conducted a preliminary geotechnical evaluation of the site to support due diligence efforts. The results of our previous evaluation are provided in our report titled "Preliminary Geotechnical Engineering Evaluation, Proposed Warehouse Development – Flint Road Site (Lots 3 through 5), Spokane, Washington," dated February 28, 2020.

We understand a portion of parcel 25305.9043 is under consideration for purchase and development, specifically Lots 2 through 5 of a proposed subdivision development, which total about 16 acres. Your partner, Puget Sound Pipe, is interested in developing Lot 2, while you are interested in developing Lots 3 through 5. The approximate locations of the respective lots are shown in the Site Plan, Figure 2.

We understand the proposed Warehouse will measure about 450 feet by 150 feet in plan dimension, encompassing a footprint of about 67,500 square feet. Finished floor grade for the proposed Warehouse will be at approximately Elevation 2,340.5. Existing site grades within the proposed Warehouse footprint range from about Elevation 2,341 near the south end of the proposed building to about Elevation 2,338 near the north end of the proposed building. Therefore, most of the building footprint will be at or above existing site grades, with up to about 2 feet of fill required to establish finished floor subgrade elevation along the northern portions of the proposed building footprint. Based on the grading plans, the southern finished exterior site grades within the southern portions of the site will be near existing site grades and fills in the range of less than 1 foot to about 2 feet will be required to establish final exterior site grades across the site.

Foundation loads were provided by the project structural engineer (KSI Structural Engineers), and will be moderate, less than about 200 kips for individual (column) loads and less than about 10 kips per lineal foot for continuous (wall) loads. We assume allowable total foundation settlement will be about 1 inch and allowable differential settlement between individual column foundations and along about 50 feet of continuous (wall) foundations will be about ½ inch.

Proposed improvements also likely will include constructing paved access and parking/storage areas, installation of underground utilities and stormwater facilities. Locations of proposed improvements relative to existing site features is shown in Figure 2.

2.0 SCOPE OF SERVICES

The purpose of our services was to provide geotechnical engineering recommendations for design and construction of the proposed warehouse development. Our recommendations are based on subsurface exploration, laboratory testing and engineering analysis. We performed our services in accordance with our proposal dated March 19, 2020. Authorization of our services via electronic communication was provided on March 25, 2020. Our specific scope of geotechnical services included:

1. Reviewing our files and previous preliminary report for applicable subsurface information.

2. 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. Lateral earth pressure criteria for design of retaining walls.
    e. Recommendations for design and construction of slab-on-grade floors.
    f. Recommendations for thickness of hot-mix asphalt (HMA) and Portland cement concrete (PCC) pavement.
    g. An evaluation of the feasibility of on-site infiltration of post-development stormwater.

3.0 SITE SURFACE CONDITIONS

The approximate 10-acre site encompassed by Lots 3 through 5 is located within an existing agricultural field. Lots 3 through 5 are bounded on the south, east and west by the agricultural field, and on the north by a stormwater ditch and undeveloped land associated with the Triumph Composite manufacturing facility. Flint Road is located about 800 feet to the east of the lots.

The site slopes gently down from southwest to northeast, with an elevation difference of about 6 feet between the southwest corner of Lot 5 and the northeast corner of Lot 3. At the time of our explorations, the field was covered with stubble. The approximate locations of site features are shown in Figure 2.

4.0 SITE SUBSURFACE CONDITIONS

4.1. Literature Review

4.1.1. Geology

Our interpretation of geologic/hydrogeologic conditions in the site vicinity is based on our review of the Washington State Department of Natural Resources "Geologic Map of the Airway Heights 7.5-Minute Quadrangle, Spokane County, Washington." The map indicates that surface geologic materials at the site consist of glacial flood deposits, predominantly gravel (Qfg). This material is described as a mixture of boulders, cobbles, sand and silt deposited during repeated catastrophic outburst flood events associated with glacial Lake Missoula. The site is located near the mapped contact between the glacial flood deposits and basalt rock of the Priest Rapids Member of the Wanapum Basalt. This geologic unit consists of fine-grained basalt, which is part of the larger Columbia River Basalt Group (CRBG). Within the west plains area of Spokane County, flood-deposited gravels are often situated on top of the lower basalt rock.

4.1.2. Soil Survey Map

We reviewed the Natural Resource Conservation Service (NRCS) Web Soil Survey, which provides general information on soil within the upper 5 feet of a site. The NRCS maps surficial soil at the site as Narcisse silt loam and Cheney ashy silt loam. The typical profile for Narcisse silt loam soil consists of silt with sand to a depth of about 2 to 3 feet, grading to silty sand to a depth of at least 5 feet. The typical profile for Cheney ashy silt loam consists of silt with sand to a depth of about 2 to 3 feet, grading to gravel with silt, sand cobbles and boulders. These soils exhibit moderate to high frost heave potential, moderate to high risk for corrosion of uncoated steel and low risk of corrosion of concrete. The NRCS indicates the potential limitations for development include possible occasional flooding and shallow depth to groundwater.

4.2. Field Activities

We explored subsurface conditions at the site on February 18, 2020 as part of our preliminary geotechnical evaluation by completing eight test pit excavations (TP-1 through TP-8). Note that additional test pits TP-9 through TP-12 were excavated within the adjacent Lot 2. Test pits TP-1 through TP-8 were advanced to depths in the range of about 5 to 10½ feet below ground surface (bgs). The locations of our test pits, including those completed for Lot 2, are shown in Figure 2.

Representative soil samples from the test pits 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.

4.3. Subsurface Conditions

In our opinion, subsurface conditions encountered in our explorations are generally consistent with the published geologic and soil survey literature. We encountered variable subsurface conditions at the locations of our test pits, and for the purposes of this report we characterized soil and rock into six general units: (1) topsoil; (2) silt; (3) silty sand; (4) gravel; (5) clay; and (6) basalt rock.

4.3.1. Topsoil

At each of the test pit locations we encountered about 8 to 12 inches of topsoil consisting of very soft silt with sand and organic matter. For the purposes of this report, we generally define topsoil as a fine-grained soil with an appreciable amount (generally more than about 15 percent by volume) of organic matter based on visual examination. It is unsuitable for direct support of the proposed improvements. However, the organic content, and other mineralogical and gradational characteristics used to evaluate the suitability of soil for use in landscaping and for agricultural purposes was not determined, nor considered in our analyses. Therefore, the information and recommendations in this report, and our logs and descriptions should not be used as a basis for concluding that topsoil from the subject site is suitable for use in landscaping or for agricultural purposes, nor for estimating the volume of topsoil that could be available for such purposes.

4.3.2. Silt

At the locations of each of our test pits, below the topsoil, we encountered dark brown to brown, soft sandy silt (Unified Soil Classification System [USCS] symbol "ML) as shown on the test pit logs). This soil unit extended to depths in the range of about 2 to 3½ feet bgs. We characterized the silt as having very low strength, moderate to high compressibility, low permeability and high susceptibility to changes in moisture content. Because of the generally non-plastic nature of the silt, and high moisture-sensitivity, the strength and stiffness of the silt unit is highly dependent on its in-situ moisture content. When the moisture content of the soil is near or dry of optimum, the soil will exhibit greater strength and stiffness characteristics. When the moisture content of the soil is above optimum, it will exhibit significantly less strength and stiffness characteristics.

4.3.3. Silty Sand

At the locations of test pits TP-2, TP-3 and TP-4, below the silt, we encountered loose to medium dense silty fine to medium sand, grading to sand with silt in TP-4 (USCS symbol "SM" and "SP-SM" as shown on the test pit logs). This soil unit extended to depths of about 2½ to 5½ feet bgs. We characterized the silty sand as having low to moderate strength, low to moderate compressibility, moderate permeability and high susceptibility to changes in moisture content.

4.3.4. Gravel

At the locations of test pits TP-1 through TP-8, below the silt or silty sand units, we encountered dense to very dense gravel with variable silt, sand, cobble and boulder content. (USCS symbol "GP-GM" or "GM" as shown on the logs), which extended to depths in the range of about 5 to 10½ feet bgs. We characterized the gravel unit as having moderate to high strength, low compressibility, moderate to high permeability and moderate to high susceptibility to changes in moisture content.

4.3.5. Clay

At the locations of test pits TP-4 through TP-7, below the gravel unit, we encountered stiff to very stiff clay with sand and occasional gravel (USCS symbol "CL" as shown on the logs), which extended to depths in the range of about 9 to 10 feet bgs. We characterized the clay unit as having moderate strength, low compressibility, very low permeability and high susceptibility to changes in moisture content.

4.3.6. Basalt Rock

We encountered refusal on basalt rock at the locations of test pits TP-1, TP-2, TP-3, TP-5 and TP-8 at depths in the range of about 5 to 10½ feet bgs. At the locations of test pits TP-3 and TP-5, we encountered highly weathered basalt rock at depths of about 8 feet and 9 feet bgs, respectively, and the backhoe was able to advance about 1 to 2½ feet into the weathered basalt before encountering refusal. We characterized the basalt rock as having very high strength, very low compressibility, very low permeability and high susceptibility to changes in moisture content.

4.4. Groundwater Conditions

We encountered groundwater at the locations of test pits TP-1 through TP-8 at depths in the range of about 3 feet to 6 feet below site grade. Based on surface topography and observed groundwater elevations, the groundwater gradient at the time of our explorations appears to flow generally from the southwest towards the northeast. Groundwater elevations likely fluctuate seasonally (generally being highest in the spring), and from year to year depending on precipitation and other forms of natural and artificial groundwater recharge on and upgradient of the site.

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. However, subsurface soil, rock and groundwater conditions encountered in our explorations will present challenges to design and construction. The following presents a brief description of geotechnical considerations for this project:

- Given the relatively shallow depth to groundwater observed at the time of our explorations, and potential for higher groundwater elevations to be present below the site during wetter periods of the year, or during years with higher antecedent precipitation than at the time of our explorations, we recommend finished site grades be established higher than current site grades (to the extent practicable) to reduce potential detrimental impacts to the site from shallow groundwater. Careful consideration should be given during development of site grading plans where loading dock retaining walls will be used.
- Given the shallow depth to groundwater, the site is not suitable for on-site infiltration of post-development stormwater. While a small amount of post-development stormwater likely can be infiltrated through swale bottoms, the project civil engineer should assess alternative methods for managing stormwater.
- The presence of shallow groundwater also should be considered when designing and constructing underground utilities. Excavations that extend below the groundwater table might require dewatering. Given the rate of seepage observed during our test pit explorations, high pumping flow rates should be anticipated. Alternatively, construction could be completed in the wet.
- Conventional earthmoving equipment should be adequate for initial stripping, excavation and filling activities. However, basalt rock was encountered at several of the explorations in the range of about 5 to 10½ feet bgs. Rippers or hydraulic breakers might be necessary to sufficiently break and loosen rock for removal.
- Given the moisture sensitivity of the surficial silt unit, site preparation and earthwork activities could be difficult, particularly if earthwork occurs outside of the ideal earthwork construction season in eastern Washington (approximately June through September). The moisture content of the silt unit likely will be above optimum outside of this construction window and could become muddy and unstable when subjected to disturbance and construction traffic. We recommend earthwork activities take place during the normally warmer and drier months of the year, if feasible. Use of cement as a stabilizing agent could provide for improved strength, workability and durability of surficial site soil.
- Foundations should be supported directly on the gravel unit, or on structural fill. Silt should be removed from below foundations. Partial excavation of the silty sand unit also could be necessary.
- Based on anticipated corrosivity of site soil, corrosion resistant underground pipes should be considered for design and installation.

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

Based on the results of our explorations and review of available information including water well reports in the site vicinity available on the Washington State Department of Ecology on-line well database, and geologic mapping, it is our opinion that the site classifies as a Site Class C. The current building code references the 2015 IBC. However, adoption of the 2018 IBC by the local building department could occur within the next six months. Therefore, we have provided design seismic parameters based on both the 2015 IBC as shown in Table 1, and the 2018 IBC as shown in Table 2. Note that the 2018 IBC references the 2016 Minimum Design Loads for Buildings and Other Structures (American Society of Civil Engineers [ASCE] 7-16).

TABLE 1. MAPPED 2015 IBC SEISMIC DESIGN PARAMETERS

Seismic Design Parameters	Recommended Parameters
Site Class	C
Mapped Spectral Response Acceleration at Short Periods (S_s)	0.326
Mapped Spectral Response Acceleration at 1 Second Period (S_1)	0.115
Site Amplification Factor at 0.2 Second Period (F_a)	1.2
Site Amplification Factor at 1 Second Period (F_v)	1.685
Design Spectral Acceleration at 0.2 Second Period (S_{DS})	0.261
Design Spectral Acceleration at 1 Second Period (S_{D1})	0.129

Note:

Parameters developed based on Latitude 47.637216 and Longitude -117.541623 using the ATC Hazards online tool.

TABLE 2. MAPPED 2018 IBC SEISMIC DESIGN PARAMETERS

Seismic Design Parameters	Recommended Parameters
Site Class	C
Mapped Spectral Response Acceleration at Short Periods (S_s)	0.304
Mapped Spectral Response Acceleration at 1 Second Period (S_1)	0.112
Site Amplification Factor at 0.2 Second Period (F_a)	1.3
Site Amplification Factor at 1 Second Period (F_v)	1.5
Design Spectral Acceleration at 0.2 Second Period (S_{DS})	0.264
Design Spectral Acceleration at 1 Second Period (S_{D1})	0.112

Note:

Parameters developed based on Latitude 47.637216 and Longitude -117.541623 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 18 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 the "Site Preparation and Earthwork" and "Structural Fill" sections of this report. Specifically, foundations should bear on properly prepared on-site gravel or on structural fill overlying on-site silty sand or gravel soil.

Individual (columns) and continuous (wall) footings may be designed using the allowable bearing pressures presented in Table 3. 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 3. ALLOWABLE BEARING PRESSURES

Footing Width (ft)	Allowable Bearing Pressures (psf)
2 or Less	2,000
3 to 4	2,500
5 or greater	3,000

5.2.3. Settlement

Based on the maximum foundation loads of less than 200 kips, we estimate that total foundation settlement (between columns, or along approximately 50 feet of continuous foundations) should be less than about 1 inch. Differential settlement should be less than about ½ inch. If foundation loads exceed the assumed amount provided at the beginning of this report, 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.

5.2.4. 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 0.40. 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 150 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 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). This also assumes potential saturated conditions within the passive zone. 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. Retaining Walls

Perimeter stem walls at loading dock locations will act as retaining walls, with retained soil and lateral earth pressures acting on the inside of the walls. Retaining walls that are allowed to yield at least 0.002H (where

H is the height of the wall) during backfilling (active soil pressure) should be designed for lateral pressures based on an equivalent fluid weight of 35 pcf (triangular earth pressure distribution) if the ground surface behind the wall is level for a distance equal to two times the wall height. This value applies to fill behind the walls that is placed and compacted as recommended in this report.

We recommend walls restrained from movement (rigid retaining walls) be designed for at-rest earth pressures using an equivalent fluid weight of 55 pcf.

These lateral earth pressures are based on drained conditions below the loading dock walls. If final site grades could result in portions of loading dock wall backfill being inundated with higher water levels on the retained side of the walls relative to exterior levels, additional lateral hydrostatic pressures should be included in the wall design. Alternatively, installation of wall drains and/or slab underdrains should be considered.

Surcharge loads are additive to lateral soil pressures. We should be consulted if surcharge loads are expected to impose additional lateral pressures on walls, or if walls will retain sloping backfill. Lateral surcharge loads from uniform loading may be estimated by multiplying the uniform vertical pressure by the applicable earth pressure coefficient (0.28 for active earth pressure conditions and 0.44 for at-rest earth pressure conditions). The resulting lateral pressure will be a uniform (rectangular) earth pressure.

Fill behind retaining walls should be placed as structural fill and conform to gradation specifications provided in the "Structural Fill" section of this report. Care must be taken by the contractor to avoid over compaction of fill placed behind walls. When placing and compacting fill within 5 feet of retaining walls, we recommend using hand-operated compaction equipment and a maximum 6-inch-thick lift thickness.

5.4. Floor Slab Support

The floor slab should be supported by at least 18 inches of structural fill overlying suitable on-site soil as recommended in the "Site Preparation and Earthwork" section of this report. We recommend the building floor slab be designed using a modulus of vertical subgrade reaction (k) of 200 pounds per cubic inch (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. The equation below may be used to estimate the modulus of vertical subgrade reaction for slab loads of various widths.

$$K = K_{S1} \frac{(B+1)^2}{4B^2}$$

Where:

- K is the modulus of vertical subgrade reaction for loaded area of width B;
- K_{S1} is the modulus of vertical subgrade reaction for a point load (200 pci); and
- B is the lateral dimension of the loaded area of the slab (in feet).

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 section of this report titled “Structural Fill.”

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. If a vapor retarder is required, we recommend using a heavy-duty plastic such as a 15- to 20-mil Stego® wrap due to the shallow groundwater conditions below the site. 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.

Provided finished floor is at about Elevation 2340.5, slab underdrains should not be necessary, in our opinion. At this elevation, the finished grades for the floor will be at or above existing and final surrounding site grades, which should allow for natural drainage away from the building floor slab. If modifications to the design results in a lower finished floor elevation, we should be consulted to review this recommendation.

5.5. Pavements

5.5.1. General

We recommend pavement materials at the site conform to applicable sections of the current Washington State Department of Transportation (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), 5-05 (Cement Concrete Pavement), 9-02 (Bituminous Materials), and applicable sections of 9-03 (Aggregates).

Determining suitable thicknesses for pavement sections should consider the following:

- Minimum thickness required to support the estimated traffic loads based on the characteristics of the supporting subgrade soil (structural design).
- Thickness required to resist frost heave (if the site is susceptible to frost heave).
- Constructability. A thicker subbase section or use of high strength geotextile and/or geogrid might be necessary in order to construct a suitable working pad during construction if the subgrade soil is soft and unable to support construction traffic during earthwork. Alternatively, stabilization of soft subgrade soil using a stabilizing agent (such as lime or cement) may be used to improve the engineering characteristics of subgrade soil.

Each of these items is discussed the following sections.

5.5.2. Structural (Thickness) Design

Pavement subgrade should be prepared as outlined in the "Site Preparation and Earthwork" section of this report. Soil placed as structural fill and gravel placed as crushed surfacing base course (CSBC) within proposed pavement and hardscape areas should be compacted as outlined in the "Structural Fill" section of this report. For structural (thickness) design of pavements, we used procedures outlines in the American Association of State and Highway Officials (AASHTO) 1993 Pavement Design Guide. We used the following input parameters in our analyses.

- We estimate the resilient modulus of subgrade soil should be on the order of about 3,000 pounds per square inch (psi). This is based on the condition that the earthwork contractor can adequately moisture-condition and prepare subgrade soil.
- We understand that traffic loading will consist predominantly of delivery trucks. The maximum truck weight will be about 75,000 pounds (including truck weight plus 45,000-pound load weight). We also assume that about 20 additional single panel delivery trucks will access the site on a weekly basis. On this basis, we estimated the 20-year design life equivalent single axle loads (ESALs) will be about 50,000 for HMA pavements and 100,000 for PCC pavements. (ESAL values for PCC pavements are higher for the same traffic volume than HMA pavements.)
- Reliability of 50 percent.
- Initial serviceability of 4.0 and terminal serviceability of 2.0.
- Structural coefficients for pavement materials as follows:
    - HMA – 0.44
    - CSBC – 0.14
    - Subbase – 0.10
    - Cement-modified soil – 0.10
- Drainage coefficient of 1.0 for HMA and 0.9 for CSBC and subbase assuming occasional inundation.
- Modulus of rupture for PCC pavement of 700 psi.
- Elastic modulus of PCC pavement of 4,000,000 psi.

5.5.3. Frost Heave Protection

For frost heave to occur, three elements must be present: (1) extended periods of freezing temperatures; (2) a source of water, particularly shallow groundwater situated within about 5 to 8 feet of ground surface; and (3) frost susceptible soil. Subgrade soils most susceptible to frost heave include silt, clay and silty sand and gravel. All three elements are present at the site. Therefore, it is our opinion that proposed pavements and exterior hardscape at the site are susceptible to frost heave.

A practical method to mitigate against frost heave is typically to remove at least a portion of the frost susceptible subgrade soil and replace with imported free-draining non-frost susceptible soil. Another option includes modifying the on-site subgrade soil to a specified depth or thickness with a stabilizing agent to reduce the frost susceptibility of the modified soil layer. The thickness of non-frost susceptible soil is a function of the estimated depth of seasonal frost penetration and risk associated with consequences of frost heave. For example, frost heave within a flexible (HMA) pavement could result in pavement cracking resulting in premature failure and/or increased maintenance, such as crack sealing, isolated full-depth repairs and/or overlays. Whereas risks associated with hardscape such as sidewalks could include heaved and disjointed sidewalk panels resulting in tripping and other hazards for pedestrians. Typical practices include providing a sufficient non-frost susceptible pavement/hardscape section for complete frost protection or partial frost protection as described below:

- Complete protection – construction of a non-frost susceptible pavement/hardscape section equal to the estimated thickness of seasonal frost penetration. For the Spokane area, estimated seasonal frost penetration is about 24 inches.
- Partial protection – construction of a non-frost susceptible pavement/hardscape section equal to about one-half to two-thirds of the estimated thickness of seasonal frost penetration. This method typically is more practical for flexible pavement sections as it allows for some frost heave to occur, with the intent of keeping seasonal frost heave levels low enough such that structural failure does not occur.
- No-frost protection – construction of the pavement/hardscape section without consideration for frost heave.

In our opinion, the use of partial frost protection for pavements and full frost protection for sidewalks and hardscape near building entrances is a feasible alternative. Ultimately, the owner should decide their level of risk tolerance relative to estimated construction costs for the various frost protection alternatives.

5.5.4. Constructability Considerations

Depending on the selected design pavement section and the condition (moisture content and related strength and stiffness) of the subgrade soil at the time earthwork is performed, additional measures might be required for constructability purposes. If the subgrade soil is soft, additional over excavation and use of stabilization fabric and possibly geogrid might be required in order to construct a suitable working pad for placement and compaction of overlying subbase, base course and pavement.

Risks associated with constructability issues can be at least partially mitigated by scheduling earthwork to occur during the normally warmer and drier months of the year. However, contingencies should be included in the project schedule and budget to account for the possibility of needing additional over excavation and replacement with imported subbase and stabilizing geotextiles or geogrids to provide a suitable working pad.

As an alternative to over excavation, stabilizing agents can be added to the on-site surficial silt soil to increase its workability and strength. Section “Soil Stabilization Using Admixture” provides a more detailed discussion on the types of admixtures available and placement of admixtures at the site.

Cement modification and cement treated base (CTB) are similar in that both involve adding and mixing cement into a soil matrix using the same types of equipment, followed by compacting the cement-soil mix to a dense condition.

- Cement modification generally involves adding cement to the soil at a lower percentage compared with CTB, with the goal of improving workability and strength, and reducing moisture sensitivity of the soil to something resembling a granular subbase material.

- CTB generally involves adding cement to either subgrade soil or imported aggregate base material to produce a high strength base to directly support a pavement layer. CTB using natural silty soil deposits typically requires a substantially higher percentage of cement compared to cement modification. Additionally, because CTB produces a stronger, more brittle product, the CTB layer may be prone to tension and shrinkage cracking, which can propagate up through the pavement layer.

For this project, we anticipate use of cement modified soil will provide a more cost-effective option than CTB. The amount of cement required to achieve suitable workability for a cement modified soil depends on the grain-size distribution of the soil and moisture content at the time of earthwork.

5.5.5. Recommended Pavement Sections

Based on the results of our pavement thickness analyses for structural support, and frost heave and constructability considerations, our recommended pavement thicknesses are presented in Tables 4 through 7. The tables include pavement section thicknesses for full frost protection, partial frost protection and no frost protection alternatives; as well as contingencies in the event subgrade soil is soft and cannot adequately support construction traffic during earthwork activities. A summary of the tables is presented below:

- Table 4 – HMA Thickness Recommendations using conventional imported granular subbase material to support the CSBC and HMA layers. Note that two optional pavement sections are provided for the' no frost protection' alternative. Either option should provide an equivalent level of support.
- Table 5 – HMA thickness recommendations using cement-modified soil to support the CSBC and HMA layers.
- Table 6 – PCC pavement thickness recommendations using imported granular subbase material to support CSBC and PCC layers.
- Table 7 – PCC pavement thickness recommendations using cement modified soil to support CSBC and PCC layers.

TABLE 4. HMA PAVEMENT THICKNESS RECOMMENDATIONS - CONVENTIONAL GRANULAR SUBBASE

Frost Protection Level	HMA Thickness (inches)	CSBC Thickness (inches)	Non-Frost Susceptible Subbase Thickness (inches)	Geotextile Separation Fabric Between Subgrade and Subbase (See Notes for Constructability Considerations)
Full Frost Protection	3	5	16	Mirafi 160N or equivalent [1]
Partial Frost Protection	3	4	8	Mirafi 160N or equivalent [2]
No Frost Protection Option 1 Option 2	 3 3	 8 4	 - 8	Mirafi 160N or equivalent [3]

Notes:

[1] Mirafi 160N will be suitable on top of firm subgrade soil. If subgrade soil is soft, subgrade should be excavated to firm bearing or 8 inches, whichever is less. If subgrade soil at working subgrade following overexcavation is still soft, geotextile should consist of stabilization fabric (Mirafi 180N or equivalent). As an alternative to additional overexcavation of soft soil, a punched and extruded biaxial or triaxial geogrid (Tensar BX 1200 or equivalent) may be placed directly over a separation fabric.

[2] Mirafi 160N will be suitable on top of firm subgrade soil. If subgrade soil is soft, subgrade soil should be excavated to firm bearing or 16 inches, whichever is less. If soil at working subgrade following overexcavation is still soft, geotextile should consist of a stabilization fabric (Mirafi 180N or equivalent). As an alternative to 16 inches of overexcavation, overexcavation can be limited to a depth of 8 inches, followed by placement of a punched and extruded biaxial or triaxial geogrid (Tensar BX 1200 or equivalent) over a geotextile separation fabric (Mirafi 160N or equivalent).

[3] Mirafi 160N will be suitable on top of firm subgrade soil. If subgrade soil is soft, subgrade soil should be excavated to firm bearing or 24 inches, whichever is less. If soil at working subgrade following overexcavation is still soft, geotextile should consist of a stabilization fabric (Mirafi 180N or equivalent). As an alternative to 24 inches of overexcavation, overexcavation may be limited to 16 inches, followed by placement of a punched and extruded biaxial or triaxial geogrid (Tensar BX 1200 or equivalent) over a geotextile separation fabric (Mirafi 160N or equivalent).

TABLE 5. HMA PAVEMENT THICKNESS RECOMMENDATIONS – CEMENT MODIFIED SOIL SUBBASE

Frost Protection Level	HMA Thickness (inches)	CSBC Thickness (inches)	Cement-Modified Soil Thickness (inches)	Geotextile Separation Fabric Between Subgrade and Subbase (See Notes for Constructability Considerations)
Full Frost Protection	3	9	12	Separation fabric not required [1]
Partial Frost Protection	3	6	8	Separation fabric not required [1]
No Frost Protection	3	4	8	Separation fabric not required [1]

Note:

[1] Use of cement-modified soil is based on the condition the earthwork contractor and specialty modification contractor can access the site with necessary equipment to complete grading and soil treatment.

TABLE 6. PCC PAVEMENT THICKNESS RECOMMENDATIONS - CONVENTIONAL GRANULAR SUBBASE

Frost Protection Level	PCC Thickness (inches)	CSBC Thickness (inches)	Non-Frost Susceptible Subbase Thickness (inches)	Geotextile Separation Fabric Between Subgrade and Subbase (See Notes for Constructability Considerations)
Full Frost Protection	8	4	12	Mirafi 160N or equivalent [1]
Partial Frost Protection	8	8	–	Mirafi 160N or equivalent [2]
No Frost Protection	8	4	–	Mirafi 160N or equivalent [3]

Notes:

[1] Mirafi 160N will be suitable on top of firm subgrade soil. If subgrade soil is soft, subgrade should be excavated to firm bearing or 12 inches, whichever is less. If subgrade soil at working subgrade following overexcavation is still soft, geotextile should consist of stabilization fabric (Mirafi 180N or equivalent). As an alternative to additional overexcavation of soft soil, a punched and extruded biaxial or triaxial geogrid (Tensar BX 1200 or equivalent) may be placed directly over a separation fabric.

[2] Mirafi 160N will be suitable on top of firm subgrade soil. If subgrade soil is soft, subgrade soil should be excavated to firm bearing or 20 inches, whichever is less. If soil at working subgrade following overexcavation is still soft, geotextile should consist of a stabilization fabric (Mirafi 180N or equivalent). As an alternative to 20 inches of overexcavation, overexcavation can be limited to a depth of 12 inches, followed by placement of a punched and extruded biaxial or triaxial geogrid (Tensar BX 1200 or equivalent) over a geotextile separation fabric (Mirafi 160N or equivalent).

[3] Mirafi 160N will be suitable on top of firm subgrade soil. If subgrade soil is soft, subgrade soil should be excavated to firm bearing or 24 inches, whichever is less. If soil at working subgrade following overexcavation is still soft, geotextile should consist of a stabilization fabric (Mirafi 180N or equivalent). As an alternative to 24 inches of overexcavation, overexcavation may be limited to 16 inches, followed by placement of a punched and extruded biaxial or triaxial geogrid (Tensar BX 1200 or equivalent) over a geotextile separation fabric (Mirafi 160N or equivalent).

TABLE 7. PCC PAVEMENT THICKNESS RECOMMENDATIONS – CEMENT MODIFIED SOIL SUBBASE

Frost Protection Level	PCC Thickness (inches)	CSBC Thickness (inches)	Cement-Modified Soil Thickness (inches)	Geotextile Separation Fabric Between Subgrade and Subbase (See Notes for Constructability Considerations)
Full Frost Protection	8	4	12	Separation fabric not required [1]
Partial Frost Protection	8	4	6	Separation fabric not required [1]
No Frost Protection	8	4	6	Separation fabric not required [1]

Notes:

[1] Use of cement-modified soil is based on the condition the earthwork contractor and specialty modification contractor can access the site with necessary equipment to complete grading and soil treatment.

We recommend that hardscape such as sidewalks which will be subjected to pedestrian traffic during winter months also be supported by a non-frost susceptible subbase layer. At a minimum, we recommend sidewalks within about 15 feet of building entrances and curb transitions be supported by a "Full Frost Protection" subbase section. The owner should evaluate risks vs. costs of frost protection for other hardscape section areas.

The upper 2 inches of CSBC may be replaced with crushed surfacing top course (CSTC) to aid the contractor in final grading and preparation for paving. Non-frost susceptible subgrade should consist of a free-draining sand or sand and gravel mixture with less than 10 percent fines.

The recommended pavement sections are based on an approximate 20-year design life. Additionally, a regular pavement maintenance program should be used, which includes periodic sealing of joints and cracks, and occasional repair or replacement of isolated damaged areas.

For PCC pavement, transverse and longitudinal joints should be spaced no greater than 14 feet on center. Use of dowels may be considered to provide additional load transfer support to concrete panels.

5.6. Site Drainage

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

5.6.1. Temporary Drainage

The presence of shallow groundwater should be considered when constructing underground utilities. Excavations that extend below the groundwater table might require dewatering. Given the rate of seepage observed during our test pit explorations, high pumping flow rates should be anticipated. Alternatively, construction could be completed in the wet.

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

5.6.2. 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.

Given the shallow depth to groundwater, the site is not suitable for on-site infiltration of post-development stormwater. While a small amount of post-development stormwater likely can be infiltrated through swale bottoms, the project civil engineer should assess alternative methods for managing stormwater. We understand off-site discharge to an existing regional drainage ditch is the planned method for stormwater disposal for this site.

If an evaporation pond is considered or required for on-site stormwater disposal, based our recent experience with similar projects near the airport, avian deterrence measures, such as netting, might be required. Additionally, use of a lined evaporation pond should take into consideration potential fluctuations in the groundwater table below the site and potential hydrostatic uplift pressures that could be exerted on the pond liner from groundwater. At this time, there is insufficient information to estimate the potential high groundwater table through the life span of the proposed development. If this option is pursued, additional consultation and coordination with the project civil engineer will be required to assess requirements, including selection of a design groundwater level, for a pond liner.

5.7. Site Preparation and Earthwork

We anticipate site preparation and earthwork operations could include: (1) clearing, stripping and grubbing; (2) site grading to establish final subgrade elevations for exterior pavement, hardscape and stormwater facilities; and (3) site grading to establish foundation grade for interior footings and subgrade for slab-on-grade floors. Additionally, cement modification of surficial site soil possibly could be used to improve its engineering characteristics for reuse as structural fill, provide a more weather-resistant working platform and act as a subbase layer for pavements and hardscape. Our specific recommendations for site preparation and earthwork are presented in the following sections.

5.7.1. Initial Site Preparation

We recommend all proposed pavement and hardscape areas be stripped of topsoil and deleterious and organic matter. Based on our explorations, we estimate the stripping depth to remove topsoil within existing swale and undeveloped areas could be in the range of about 8 to 12 inches. Greater stripping depths might be encountered during construction in other areas of the site not explored.

5.7.2. General Grading and Excavation

In our opinion, conventional earthmoving equipment such as excavators, backhoes and dozers should be adequate for initial stripping, excavation and filling activities. However, basalt rock was encountered at several of the explorations in the range of about 5 to 10½ feet bgs. While we do not anticipate basalt rock will be encountered during general site grading, depending on planned depths of underground utilities and top of rock elevations, it could be possible that rock is encountered within utility trench excavations. If this occurs, large track-mounted equipment with rippers or hydraulic breakers might be necessary to sufficiently break and loosen rock for removal.

As stated previously, the existing silt and silty sand units are highly 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 using these moisture sensitive soils 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. Additionally, the natural moisture content of these soil units could be more than about 3 percent above optimum at the time earthwork is performed. Therefore, proper moisture conditioning might be required, such as scarifying and aerating, to achieve suitable moisture contents before the soil can be reused on site.

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 or outside of the typical warm weather construction window in eastern Washington, we recommend that the project specifications and budget include provisions for removal of unsuitable material and importing and compacting additional structural fill.

5.7.3. Excavations for Subgrade and Foundation Grade

At the time of our explorations, the upper approximately 2 to 4 feet of soil across the site consisted of soft, highly moisture sensitive silt soil. This soil unit is not suitable for direct support of building foundations, floor slabs, pavement and hardscape. Therefore, we recommend the following considerations for site preparation:

- The silt unit should be completely removed from below foundations to expose the underlying silty sand or gravel unit. Additionally, if silty sand is exposed following initial foundation excavation, we recommend additional excavation to expose the underlying gravel unit or provide a minimum 3-foot-thick structural fill pad below footings, whichever occurs first. We anticipate excavations in the range of about 2 to 4 feet below current site grade will be required to remove the silt and possibly portions of the silty sand unit from below foundations. Excavations also should extend laterally beyond the edges of footings a distance equal to the depth of excavation below foundation grade.
- Floor slabs should be supported by at least 18 inches of imported granular structural fill overlying the silt unit. This fill section should consist of a combination of free-draining capillary break material and general imported granular structural fill.
- Exterior pavement and hardscape should be supported as recommended in the "Pavements" section of this report.

5.7.4. 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 92 percent of the MDD for soil within the proposed building footprint (below foundations and floor slab).

These minimum density requirements are based on subgrade soil being within about 3 percentage points of optimum at the time of earthwork and compaction. A representative of GeoEngineers should evaluate soil conditions at working subgrade and within foundation excavations before and during soil compaction and before placing structural fill, formwork or reinforcing steel. Depending on soil conditions at the time earthwork is performed, evaluation of subgrade preparation should be accomplished through in-place density testing of the prepared areas, or alternatively probing and proof-rolling may be used to assess the stability of undisturbed soil. The most appropriate method for evaluating subgrade conditions 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 should be mitigated as described below:

- Pavement areas should be overexcavated to firm bearing, or a suitable depth as indicated in the notes in Tables 4 through 7 based on the selected pavement type and section thickness. Use of geotextiles and geogrid below pavements also should be in accordance with Tables 4 through 7.
- Soft subgrade below hardscape and floor slabs should be overexcavated to firm bearing or a depth of 2 feet, whichever is less, and replaced with suitable structural fill. If soil is still soft following overexcavation, a geotextile stabilization fabric (Mirafi 180N or equivalent) should be placed on top of working subgrade before placing structural fill to re-establish subgrade elevations. Alternatively, a

punched and extruded biaxial or triaxial geogrid (Tensar BX 1200 or equivalent) may be placed on top of a geotextile separation fabric overlying the soft subgrade.

- Soft or unstable areas below foundations should be excavated to firm bearing and replaced with suitable structural fill.

5.7.5. Soil Stabilization Using Admixture

Stabilization by using admixtures in the in-situ soils may be an alternative to overexcavation within the slab-on-grade floor, pavement and hardscape areas. Additionally, if mass grading and earthwork activities are planned or occur during wet weather, the contractor likely will be unable to adequately protect the near-surface soil from becoming wet and unstable and using an admixture is a mitigation option. The owner or general contractor may consider retaining a specialty earthwork contractor to stabilize the on-site soil using an admixture. Lime, fly ash, asphalt and cement may be used to improve the physical and engineering properties of soil by reducing the plasticity, and enhancing the compaction characteristics and strength of the materials. Principal benefits of stabilization using an admixture include:

- Improved constructability of marginal on-site soil.
- Reduced plasticity and improved strength.
- Less susceptible to disturbance when exposed to water.
- Use of on-site soil rather than removal and replacement with imported granular structural fill.

The type and quantity of admixture used for stabilization will vary based on the in-situ engineering properties of the soil. Generally, lime is best suited for soil with a clay content greater than 10 percent, while fly-ash and asphalt admixtures are best suited for granular soil. While a wide range of soil types may be stabilized using Portland cement, most researchers agree that it is the most effective method (in comparison to other stabilizers) for sand, sandy and silty soil, and clayey soil of low to medium plasticity. Based on the high fines content and general non-plastic nature of the on-site surficial soil, it is our opinion that Portland cement is the preferred agent for subgrade stabilization at the site.

Four factors control the properties and characteristics of cement-stabilized soil including: (1) the gradation of the soil material, whether it's clay, silt, sand, coarse aggregate or a combination; (2) the percentage (by weight) cement in the mix; (3) moisture content of the soil before mixing and at the time of compaction and curing (moisture, temperature and time); and (4) the degree of compaction. It is possible simply by varying the cement content, to produce mixes ranging from those which result in modification of the compacted soil to those which result in hard soil-cement that will meet durability and strength requirements for a cement-treated base. Because of the varied reactions resulting from a mixture of cement with different soil, even those within a specific soil group, it will be necessary to conduct laboratory tests on soil samples to estimate the optimum cement and moisture contents, appropriate for the on-site surficial soil for the target compaction criteria specific to the proposed project. The preparation of test specimens in the laboratory should represent, on a small scale, the steps and processes actually employed during construction. Such testing was beyond the scope of services for this geotechnical evaluation.

During construction, the objective should be a thorough mixture of a pulverized soil with the correct quantity of stabilizer (cement) and sufficient moisture to permit maximum compaction. The following construction steps are typically employed for subgrade stabilization operations: (1) soil preparation; (2) admixture (cement) application; (3) pulverization and mixing; (4) compaction; and (5) curing. To achieve uniform mixing and compaction during construction, equipment must be selected, operated and sequenced to provide the following:

1. The proper water content (uniformly mixed);
2. The proper cement content (uniformly mixed);
3. The attainment of some minimum specified density;
4. Favorable temperature and moisture conditions for strength development during the curing period; and
5. Protection of the stabilized surface from traffic to prevent abrasion and to ensure adequate time for strength development.

In order to develop a final product that will be adequate for its intended use, it is essential that an experienced contractor be retained who is able to perform the work in accordance with the project plans and specifications. We recommend issuing a Contractor Prequalification form to allow only qualified bidders to submit a price for the project. It is equally important that the quality of the cement modification mixture, as produced and placed, be subjected to a quality control review in the field to evaluate uniformity of the product. Therefore, if cement modification is selected for this project, we recommend that full-scale laboratory testing, detailed project specifications and a quality control plan be completed prior to advertising for a qualified specialty contractor. We are available to provide additional assistance in this area, as necessary.

For cost estimating purposes we recommend a cement content of around 3 to 6 percent by weight, and an assumed compacted unit weight of about 115 pcf; more cement may be required depending on the soil treated and the soil moisture content during treatment. Additionally, excessively wet or soft pumping areas generated during backfill and fill placement may require additional cement and mixing. The curing process usually takes 1 to 3 days. Once mixed and placed, the cement-treated soil should be compacted to at least 90 to 95 percent of the MDD based on ASTM D 1557 and it should be visually in a uniformly firm condition prior to placement of additional structural fill. Our geotechnical report provides recommendations for placement of structural fill and a discussion on the use of on-site soils. It should be noted that if the treated soil is left exposed to excessive construction traffic and/or wet weather, the structural fill and roadway subgrades could become disturbed. In addition, it should be noted that once “set-up” has occurred, excavation through the soil and cement mixture may be difficult.

5.8. 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.

5.8.1. Use of On-Site Soil as Structural Fill

In our opinion, the silt and silty sand soil units are highly moisture sensitive. These soil units will only be suitable for reuse as structural fill in an untreated condition during extended periods of warm dry weather. Additionally, moisture conditioning such as scarifying and aerating might be required to achieve suitable moisture contents before these soil units are suitable for reuse. We recommend the silt and silty sand units only be used as structural fill below exterior areas. These units should not be reused as structural fill within the building footprint.

In our opinion, the gravel unit should be suitable as an all-weather general structural fill, although we anticipate limited amounts of this material will be available for reuse. This material likely will be significantly wet of optimum and require aeration to achieve suitable moisture contents before being suitable for reuse. Additionally, screening might be required to remove oversized particles.

The clay unit is not suitable for reuse as structural fill, in our opinion unless is it modified using an admixture, as discussed previously.

For these reasons, we recommend contingencies be included in the project schedule and budget to account for the possibility that the moisture content of the moisture-sensitive soil is not near optimum at the time of earthwork, requiring either additional handling by the earthwork contractor and time for proper moisture-conditioning, or off-site disposal of over-optimum soil and importing suitable structural fill, particularly if earthwork activities will occur during the late fall through late spring.

5.8.2. 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 (as general structural fill or subbase) 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.

5.8.3. 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, regardless of depth below floor subgrade or foundation grade, should be compacted to at least 92 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, unless a higher degree of compaction is required as outlined in this report.
- 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 floor slabs. 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.

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.

5.8.1. Cut and Fill 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. 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.

Permanent cut and fill slopes should be sloped at an inclination no steeper than 2H:1V. Permanent cut and fill slopes should be seeded or sodded as soon as practical following construction to reduce the potential for erosion. Some sloughing and erosion should be expected until vegetation matures.

5.9. Weather Considerations

As stated previously, the on-site silt and silty sand soil is 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 October 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.

Alternatively, cement modification can be used to construct a more weather-resistant working platform to support construction traffic. Details regarding design and construction of a cement-modified layer are presented in other portions of this report.

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 McKinstry for the proposed Warehouse development project near Flint Road in Spokane, Washington. McKinstry 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 B, titled “Report Limitations and Guidelines for Use,” for additional information pertaining to use of this report.

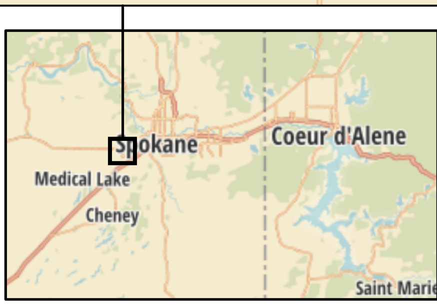
8.0 REFERENCES

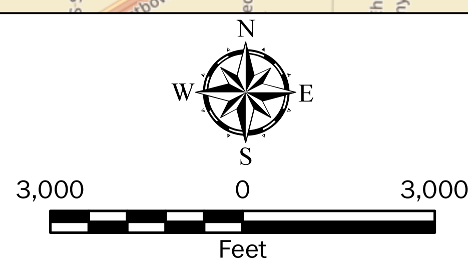
Derkey, R.E., et al. “Geologic Map of the Airway Heights 7.5-Minute Quadrangle, Spokane County, Washington”. Washington State Department of Natural Resources, 2004.

Natural Resource Conservation Service (NRCS), On-line Web Soil Survey. https://websoilsurvey.nrcs.usda.gov/app/. Accessed February 2020.

Spokane Regional Stormwater Manual. April 2008.

Washington State Department of Ecology, on-line well report viewer. https/fortress.wa.gov/ecy/wellconstruction/map/WCLSWebMap/default.aspx. Accessed March 2020.





Vicinity Map

Proposed Warehouse Development
Spokane, Washington

GEOENGINEERS

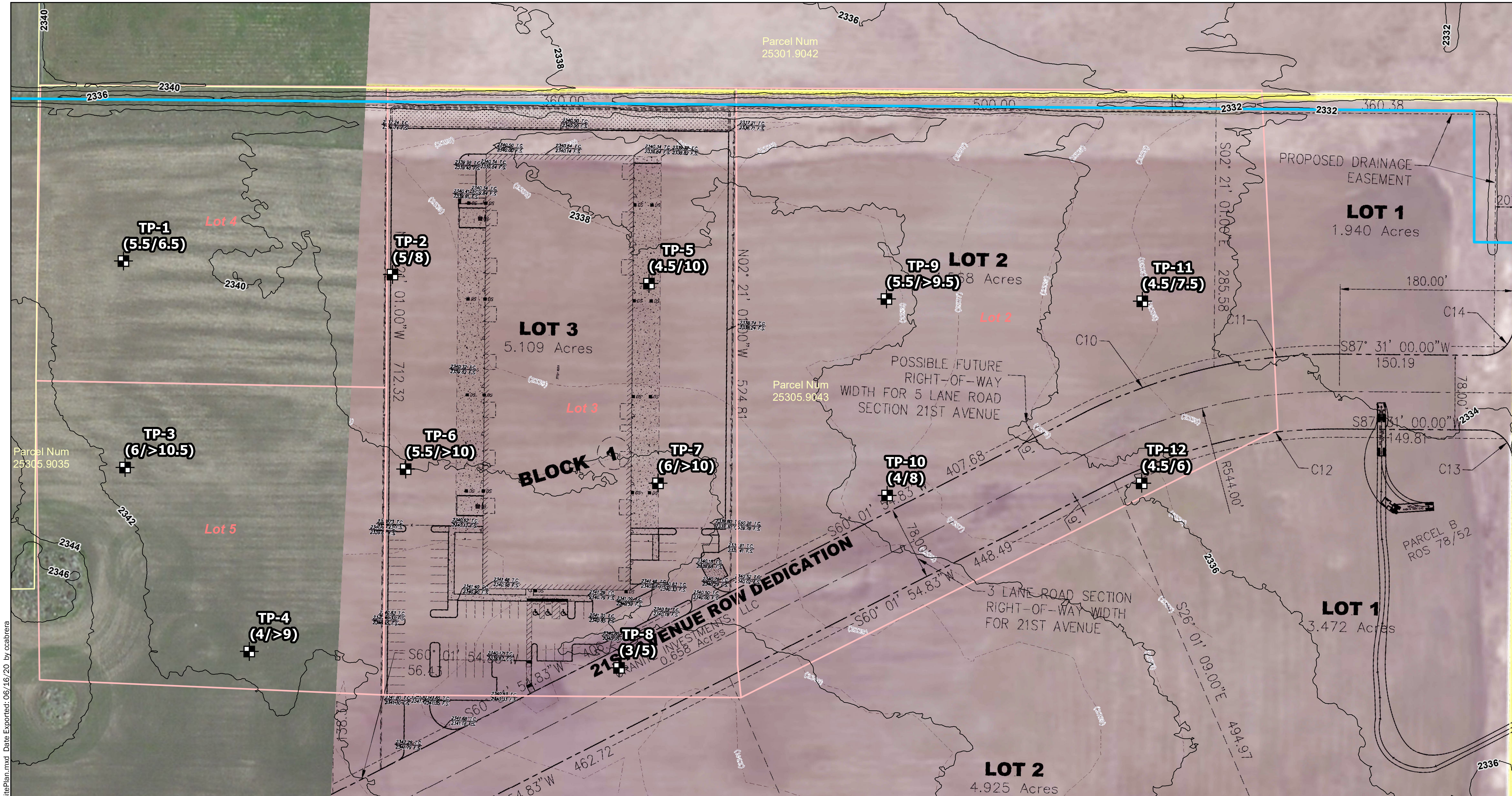
Figure 1

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

P:\18\18324005\GIS\MXD\1832400501_F01_VicinityMap_GT.mxd Date Exported: 03/27/20 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: Tax lots from Spokane County GIS. Aerial image from ESRI. Contours generated from 2015 Spokane Lidar, Washington State DNR. Horizontal Datum: NAD83 (HARN) Vertical Datum: NAVD88 (GEOID03). Overlay from DCI Engineers.

Projection: NAD 1983 UTM Zone 11N

Legend

- Test Pit Number and Approximate Location
- Regional Drainage Ditch
- Contours (2 Ft Interval)
- (3/5) (Approximate depth to groundwater at time of exploration (ft)/ Approximate depth to basalt rock (ft))
- Approximate Tax Parcel Boundary (Spokane County)
- Approximate Proposed Lot Boundary

N W E S

100 0 100
Feet

Site Plan

Proposed Warehouse Development
Spokane, Washington

GeoEngineers

Figure 2

P:\18\18324005\GIS\MXD\1832400501_F02_SitePlan.mxd Date Exported: 06/16/20 by ccabrera

APPENDIX A
Field Methods, Exploration Logs and Geotechnical Laboratory Testing

APPENDIX A
FIELD METHODS, EXPLORATION LOGS AND GEOTECHNICAL LABORATORY TESTING

General

We explored soil, rock and groundwater conditions at the site on February 18, 2020, by observing excavation of 12 test pits (TP-1 through TP-12) at the approximate locations shown on the Site Plan, Figure 2. The test pits were excavated by Vietzke Excavating under subcontract to GeoEngineers using a rubber-tired Caterpillar 308 track-mounted excavator.

General Soil Sampling Procedures

The explorations were continuously monitored by a representative from GeoEngineers who classified the soil encountered, maintained detailed logs of the test pits showing stratigraphic changes and other pertinent information, obtained representative soil samples and observed groundwater conditions. Soil encountered in the test pits was classified in the field in general accordance with ASTM International (ASTM) D 2488, the Standard Practice for the Classification of Soils (Visual-Manual Procedure), which is described in Key to Exploration Logs, Figure A-1. Logs of the explorations are presented in Logs of Test Pits, Figures A-2 through A-13. 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.

Exploration locations were selected base on review of the proposed lot dimensions and established in the field using a hand-held global positioning (GPS) device. Elevations at test pit locations was estimated based on interpolation of exploration locations relative to elevation contours shown on Figure 2. Ground surface contours were estimated based on publicly available LiDAR data completed by others for the Spokane region. Elevations are based on the North American Vertical Datum of 1988 (NAVD88). The locations and elevations shown on the logs should be considered accurate to the degree implied by the method used.

Geotechnical Laboratory Testing

Soil samples obtained from the test pits were returned to our laboratory for further examination. Representative samples were selected for percent passing the 200 sieve tests in accordance with ASTM test method D 1140 and moisture content determinations in accordance with ASTM D 2216. Results are presented on the test pit logs at the 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

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.

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

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 and dry density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PL	Point lead test
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

Rev 07/2019

Date Excavated	2/18/2020	Total Depth (ft)	6.5	Logged By BKH Checked By DRL	Excavator Vietzke Excavating Equipment Caterpillar 308 Excavator	See "Remarks" section for groundwater observed Caving not observed

Surface Elevation (ft) Vertical Datum	2341 NAVD88	Latitude Longitude	47.6385 -117.5458	Coordinate System Horizontal Datum	WA State Plane North NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
					TS	Approximately 9 to 12 inches of dark brown silt with sand and organic matter (very soft, moist) (topsoil)			
2340	1		1		ML	Dark brown silt with sand (soft, moist)			
2339	2				GP-GM	Brown fine to coarse gravel with silt, sand, cobbles and boulders (medium dense to dense, moist)			
2338	3		2 / SA				8	9	
2337	4								
2336	5								
2335	6					Becomes very dense			Moderate to rapid groundwater seepage observed at 5½ feet below ground surface
					RX	Test pit terminated at approximately 6½ feet below ground surface because of excavator refusal in basalt rock			

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on USGS Topo.

Date:6/19/20 Path:P:\18\18324005\GINT\1832400500.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_TESTPIT_1P_GEOTEC_%F

Log of Test Pit TP-1

GEOENGINEERS

Project: Proposed Warehouse Development
Project Location: Spokane, Washington
Project Number: 18324-005-01

Date Excavated	2/18/2020	Total Depth (ft)	8	Logged By	BKH	Excavator	Vietzke Excavating	See "Remarks" section for groundwater observed
				Checked By	DRL	Equipment	Caterpillar 308 Excavator	Caving not observed

Surface Elevation (ft)	2339.5	Latitude	47.6385	Coordinate System	WA State Plane North
Vertical Datum	NAVD88	Longitude	-117.5446	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
2339	0		1		TS	Approximately 9 to 12 inches of dark brown silt with sand and organic matter (very soft, moist) (topsoil)			
2338	1				ML	Dark brown silt with fine sand (soft, moist)			
2337	2				SM	Brown silty fine to medium sand (loose, moist)			
2336	3		2 %F		GP-GM	Brown fine to coarse gravel with silt, sand, cobbles and boulders (medium dense to dense, moist)	8	9	
2335	4								
2334	5					Becomes wet			Moderate to rapid groundwater seepage observed at 5 feet below ground surface
2333	6								
2332	7					Grades with less silt			
	8				RX	Test pit terminated at approximately 8 feet below ground surface because of excavator refusal in basalt rock			

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on USGS Topo.

Log of Test Pit TP-2

GEOENGINEERS

Project: Proposed Warehouse Development
Project Location: Spokane, Washington
Project Number: 18324-005-01

Date Excavated	2/18/2020	Total Depth (ft)	10.5	Logged By BKH / Checked By DRL	Excavator: Vietzke Excavating / Equipment: Caterpillar 308 Excavator	See "Remarks" section for groundwater observed / Caving not observed

Surface Elevation (ft)	2342	Latitude	47.638	Coordinate System	WA State Plane North
Vertical Datum	NAVD88	Longitude	-117.5458	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				TS	Approximately 8 to 10 inches of dark brown silt with sand and organic matter (very soft, moist) (topsoil)			
2341	1				ML	Dark brown sandy silt (soft, moist)			
2340	2								
2339	3		1 / MC		SM	Brown silty fine to medium sand with occasional gravel (loose, moist)	16		
2338	4				GM	Light brown silty fine to coarse gravel with sand, cobbles and boulders (medium dense to dense, moist)			
2337	5		2						
2336	6					Becomes very dense			Slight groundwater seepage observed at 6 feet below ground surface
2335	7								
2334	8				GC	Reddish brown clayey gravel (very dense, wet) (decomposed basalt)			
2333	9								
2332	10								

Test pit completed at 10½ feet below ground surface

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on USGS Topo.

Log of Test Pit TP-3

GeoEngineers

Project: Proposed Warehouse Development
Project Location: Spokane, Washington
Project Number: 18324-005-01

Date Excavated 2/18/2020	Total Depth (ft) 9	Logged By BKH Checked By DRL	Excavator Vietzke Excavating Equipment Caterpillar 308 Excavator	See "Remarks" section for groundwater observed Caving not observed

Surface Elevation (ft) 2342 Vertical Datum NAVD88	Latitude 47.6374 Longitude -117.5452	Coordinate System WA State Plane North Horizontal Datum NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
2342–2341	0–1			TS	Approximately 9 to 12 inches of dark brown silt with sand and organic matter (very soft, moist) (topsoil)			
2341–2339	1–3			ML	Dark brown silt with fine sand and occasional boulders (soft, moist)			
2339–2337.5	3–4.5	3–4	1 %F	SM	Light brown silty fine to medium sand (loose, wet)	20	40	Rapid groundwater seepage observed at 4 feet below ground surface
2337.5–2336.5	4.5–5.5	4.5–5	2	SP-SM	Brown fine to medium sand with silt and occasional boulders (loose to medium dense, wet)			
2336.5–2335	5.5–7	6–7	3 %F	GM	Brown silty fine to coarse gravel with sand, cobbles and occasional boulders (medium dense to dense, wet)	16	13	
2335–2333	7–9	7.5–8.5	4	CL	Light gray-tan clay with trace sand and occasional gravel (very stiff, moist)			PP: UCS = 2.25 tsf

Test pit completed at 9 feet below ground surface

UCS = unconfined compressive strength

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on USGS Topo.

Log of Test Pit TP-4

GeoEngineers

Project: Proposed Warehouse Development
Project Location: Spokane, Washington
Project Number: 18324-005-01

Figure A-5
Sheet 1 of 1

Date:6/19/20 Path:P:\18\18324005\GINT\1832400500.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_TESTPIT_1P_GEOTEC_%F

Date Excavated 2/18/2020	Total Depth (ft) 10	Logged By BKH Checked By DRL	Excavator Vietzke Excavating Equipment Caterpillar 308 Excavator
See "Remarks" section for groundwater observed See "Remarks" section for caving observed			

Surface Elevation (ft) 2338 Vertical Datum NAVD88	Latitude 47.6385 Longitude -117.5436	Coordinate System WA State Plane North Horizontal Datum NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
2338	0				TS	Approximately 8 to 10 inches of dark brown silt with sand and organic matter (very soft, moist) (topsoil)			
2337	1				ML	Dark brown sandy silt (soft, moist to wet)			
2336	2								
2335	3		1		GP-GM	Brown fine to coarse gravel with silt, sand and cobbles (medium dense to dense, moist to wet)			Minor caving observed at 3 to 6 feet below ground surface
2334	4								Rapid groundwater seepage observed at 4½ feet below ground surface
2333	5								
2332	6								
2331	7		2		CL	Reddish brown clay with sand (very stiff, moist)			PP: UCS = 3.50 tsf
2330	8								
2329	9		3		GC	Brown clayey fine to coarse gravel with sand (very dense, wet) (decomposed basalt)			
2328	10				RX	Test pit terminated at approximately 10 feet below ground surface because of excavator refusal in basalt rock			

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on USGS Topo.

Log of Test Pit TP-5

GeoEngineers

Project: Proposed Warehouse Development
Project Location: Spokane, Washington
Project Number: 18324-005-01

Date Excavated 2/18/2020	Total Depth (ft) 10	Logged By BKH Checked By DRL	Excavator Vietzke Excavating Equipment Caterpillar 308 Excavator	See "Remarks" section for groundwater observed See "Remarks" section for caving observed

Surface Elevation (ft) 2340 Vertical Datum NAVD88	Latitude 47.638 Longitude -117.5446	Coordinate System WA State Plane North Horizontal Datum NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
2340	0				TS	Approximately 10 to 12 inches of dark brown silt with sand and organic matter (very soft, moist) (topsoil)			
2339	1				ML	Dark brown silt with sand (soft, moist to wet)			
2336.5	3.5		1		GP-GM	Brown fine to coarse gravel with silt, sand, cobbles and boulders (medium dense to dense, wet)			Moderate caving observed at 4 to 7 feet below ground surface Rapid groundwater seepage observed at 5½ feet below ground surface
2332	8		2		CL	Tan clay with trace sand (stiff, moist to wet)			PP: UCS = 2.00 tsf
2330	10					Test pit completed at 10 feet below ground surface			

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on USGS Topo.

Log of Test Pit TP-6

GEOENGINEERS

Project: Proposed Warehouse Development
Project Location: Spokane, Washington
Project Number: 18324-005-01

Date Excavated	2/18/2020	Total Depth (ft)	10	Logged By	BKH	Excavator	Vietzke Excavating	See "Remarks" section for groundwater observed
				Checked By	DRL	Equipment	Caterpillar 308 Excavator	See "Remarks" section for caving observed

Surface Elevation (ft)	2340	Latitude	47.6379	Coordinate System	WA State Plane North
Vertical Datum	NAVD88	Longitude	-117.5435	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
2340	0				TS	Approximately 8 to 10 inches of dark brown silt with sand and organic matter (very soft, moist) (topsoil)			
2339	1				ML	Dark brown silt with sand (soft, moist to wet)			
2338	2								
2337	3		1		GP-GM	Brown fine to coarse gravel with silt, sand and cobbles (medium dense to dense, moist to wet)			Minor caving observed at approximately 3 to 5 feet below ground surface
2336	4								
2335	5								
2334	6		2						Moderate groundwater seepage observed at 6 feet below ground surface
2333	7								
2332	8				CL	Brown clay with trace sand (very stiff, moist to wet)			
2331	9								
2330	10					Test pit completed at 10 feet below ground surface			

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on USGS Topo.

Log of Test Pit TP-7

GeoEngineers

Project: Proposed Warehouse Development
Project Location: Spokane, Washington
Project Number: 18324-005-01

Date Excavated	2/18/2020	Total Depth (ft)	5	Logged By	BKH	Excavator	Vietzke Excavating	See "Remarks" section for groundwater observed
				Checked By	DRL	Equipment	Caterpillar 308 Excavator	Caving not observed

Surface Elevation (ft)	2340	Latitude	47.6374	Coordinate System	WA State Plane North
Vertical Datum	NAVD88	Longitude	-117.5437	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0				TS	Approximately 10 to 12 inches of dark brown silt with sand and organic matter (very soft, moist to wet) (topsoil)			
2339	1				ML	Dark brown silt with trace sand (soft, moist to wet)			
2338	2								
2337	3		1 / SA		GM	Brown silty fine to coarse gravel with sand, cobbles and occasional boulders (dense, wet)	22	15	Rapid groundwater seepage observed at 3 feet below ground surface
2336	4								
2335	5				RX	Test pit terminated at approximately 5 feet below ground surface because of excavator refusal in basalt rock			

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on USGS Topo.

Log of Test Pit TP-8

GEOENGINEERS

Project: Proposed Warehouse Development
Project Location: Spokane, Washington
Project Number: 18324-005-01

Date Excavated	2/18/2020	Total Depth (ft)	9.5	Logged By	BKH	Excavator	Vietzke Excavating	See "Remarks" section for groundwater observed
				Checked By	DRL	Equipment	Caterpillar 308 Excavator	See "Remarks" section for caving observed

Surface Elevation (ft)	2338	Latitude	47.6384	Coordinate System	WA State Plane North
Vertical Datum	NAVD88	Longitude	-117.5426	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0		1		TS	Approximately 9 to 12 inches of silt with sand organic matter (very soft, moist) (topsoil)			
2337	1		2		ML	Dark brown sandy silt (soft, moist to wet)			
2336	2								
2335	3				SM	Brown silty fine to medium sand (loose, moist to wet)			
2334	4		3						
2333	5								
2332	6		4 %F		GP-GM	Brown fine to coarse gravel with silt, sand, cobbles and boulders (medium dense to dense, wet)	15	7	Rapid groundwater seepage observed at 5½ feet below ground surface Minor caving observed at approximately 6 feet below ground surface
2331	7								
2330	8		5		CL	Brown clay with sand (very stiff, moist to wet)			
2329	9								

Test pit completed at 9½ feet below ground surface

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on USGS Topo.

Log of Test Pit TP-9

GeoEngineers

Project: Proposed Warehouse Development
Project Location: Spokane, Washington
Project Number: 18324-005-01

Date Excavated	2/18/2020	Total Depth (ft)	8	Logged By	BKH	Excavator	Vietzke Excavating	See "Remarks" section for groundwater observed
				Checked By	DRL	Equipment	Caterpillar 308 Excavator	Caving not observed

Surface Elevation (ft)	2338	Latitude	47.6379	Coordinate System	WA State Plane North
Vertical Datum	NAVD88	Longitude	-117.5426	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
2338–2337	0				TS	Approximately 9 to 12 inches of dark brown silt with sand and organic matter (very soft, moist) (topsoil)			
2337	1		1 MC		ML	Dark brown silt with sand (soft, moist to wet)	43		
2336	2								
2335	3				GM	Brown silty fine to coarse gravel with sand and cobbles (medium dense to dense, moist to wet)			
2334	4		2						Slight groundwater seepage observed at 4 feet below ground surface
2333	5				CL	Reddish tan clay with trace sand (very stiff to hard, moist)			
2332	6		3						PP: UCS = 3.5 tsf
2331	7								
2330	7.5				GC	Red clayey fine to coarse gravel (very dense, moist) (decomposed basalt)			
	8				RX	Test pit terminated at approximately 8 feet below ground surface because of excavator refusal in basalt rock			

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on USGS Topo.

Log of Test Pit TP-10

GeoEngineers

Project: Proposed Warehouse Development
Project Location: Spokane, Washington
Project Number: 18324-005-01

Date Excavated	2/18/2020	Total Depth (ft)	7.5	Logged By	BKH	Excavator	Vietzke Excavating	See "Remarks" section for groundwater observed
				Checked By	DRL	Equipment	Caterpillar 308 Excavator	Caving not observed

Surface Elevation (ft)	2335	Latitude	47.6384	Coordinate System	WA State Plane North
Vertical Datum	NAVD88	Longitude	-117.5415	Horizontal Datum	NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
2335–2334	0–1				TS	Approximately 10 to 12 inches of dark brown silt with sand and organic matter (very soft, moist) (topsoil)			
2334–2332	1–3	▯	1		ML	Dark brown silt with sand (soft, moist to wet)			
2332–2331	3–4	▯	2 MC		SM	Brown silty fine to medium sand (loose, moist to wet)	18		
2331–2328	4–7	▯	3		GM	Brown silty fine to coarse gravel with sand, cobbles and occasional boulders (medium dense to dense, wet)			Rapid groundwater seepage observed at 4½ feet below ground surface
2328–2327.5	7–7.5	▯	4		SC	Reddish brown clayey fine to coarse sand with gravel (dense, wet) (decomposed basalt)			
					RX	Test pit terminated at approximately 7½ feet below ground surface because of excavator refusal in basalt rock			

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on USGS Topo.

Log of Test Pit TP-11

GeoEngineers

Project: Proposed Warehouse Development
Project Location: Spokane, Washington
Project Number: 18324-005-01

Figure A-12
Sheet 1 of 1

Date Excavated	2/18/2020	Total Depth (ft)	6	Logged By BKH Checked By DRL	Excavator Vietzke Excavating Equipment Caterpillar 308 Excavator

See "Remarks" section for groundwater observed
Caving not observed

Surface Elevation (ft) Vertical Datum	2336 NAVD88	Latitude Longitude	47.6379 -117.5415	Coordinate System Horizontal Datum	WA State Plane North NAD83 (feet)

Elevation (feet)	Depth (feet)	SAMPLE: Testing Sample	SAMPLE: Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
2336–2335	0–1				TS	Approximately 8 to 10 inches of dark brown silt with sand and organic matter (very soft, moist) (topsoil)			
2335–2333	1–3				ML	Dark brown silt with fine sand and occasional cobbles (soft, moist to wet)			
2333	3		1		CL	Brown sandy clay (soft, wet)			
2332–2330	4–6		2 %F		GP	Reddish brown fine to coarse gravel with sand, cobbles and trace silt (dense, wet)	13	3	Rapid groundwater seepage observed at 4½ feet below ground surface
	6				RX	Test pit terminated at approximately 6 feet below ground surface because of excavator refusal in basalt rock			

Notes: See Figure A-1 for explanation of symbols.
The depths on the test pit logs are based on an average of measurements across the test pit and should be considered accurate to ½ foot.
Coordinates Data Source: Horizontal approximated based on USGS Topo. Vertical approximated based on USGS Topo.

Log of Test Pit TP-12

GeoEngineers

Project: Proposed Warehouse Development
Project Location: Spokane, Washington
Project Number: 18324-005-01

APPENDIX B
Report Limitations and Guidelines for Use

APPENDIX B
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 McKinstry 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 March 19, 2020, 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 Warehouse Development located west of Flint Road 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.