Draft Geotechnical Engineering Services Report

Puyallup AOB Site
Puyallup, Washington

for
MC Construction Consultants

March 28, 2022

GeoEngineers

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File No. 8947-005-00

March 28, 2022

Prepared for:

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1.0 INTRODUCTION AND PROJECT UNDERSTANIDNG

GeoEngineers, Inc. (GeoEngineers) is pleased to submit this geotechnical engineering study and report for the Puyallup AOB Site. The site is located at 330 3rd Street SW in Puyallup, Washington as shown on the Vicinity Map, Figure 1. Prior experience at this site includes subsurface explorations and a preliminary study prepared by GeoEngineers for the City of Puyallup to support potential improvements to the site. GeoEngineers advanced three borings which we reference to support this study. Our previous report is titled "AOB Site Preliminary Geoenvironmental Study" and is dated September 30, 2011 (September 2011 Report).

Our understanding of the proposed improvements is based on conversations with you and review of preliminary site plans. Proposed improvements include a four-story multifamily residential structure with at grade parking and with three stories of residential space above. Below grade parking is not currently envisioned. Based on our discussions with you, we understand that the preferred foundation support method is conventional shallow foundations underlain by ground improvement.

2.0 SCOPE OF SERVICES

The purpose of our services is to review existing geotechnical information at the site as a basis for providing geotechnical design and construction recommendations for the proposed development. In general, our authorized services included: reviewing selected geotechnical information about the site; completing geotechnical analyses; and preparing this geotechnical report with our conclusions, findings and recommendations. Our services are being provided in general accordance with our agreement with MC Construction Consultants authorized February 22, 2022. Our complete scope of services is provided in our proposal dated February 3, 2022.

3.0 SITE CONDITIONS

3.1. Surface Conditions

The site is located southwest of the intersection of Pioneer Way and 3rd Street SW in downtown Puyallup and is bounded to the north and east by city street right-of-way and by commercial lots to the west and south. The site is currently used as an asphalt paved parking area. Landscaping areas that include small trees, grasses, and shrubs are located on the perimeter.

The site is relatively level with small variations in topography between opposite sides. We understand that prior development of the site included a two-story building in the southeast corner and a grocery store in the center of the site, both of which were removed prior to construction of the parking lot.

3.2. Literature Review

3.2.1. Geologic Conditions

Based on our review of the map titled "Geologic Map of the Tacoma 1:100,000-scale Quadrangle, Washington" (Schuster et. al. 2015) the site is underlain by Holocene Alluvium (map unit Qa). This deposit is described as comprising a mixture of sand, silt, gravel and cobbles. In addition, alluvium deposits in this region can be underlain by lahars and mudflow deposits from Mt. Rainier.

3.2.2. Prior Geotechnical Studies

In addition to the 2011 Report prepared by GeoEngineers for this site, we reviewed two other geotechnical studies that were completed at the site:

- "Groundwater Level Monitoring and Preliminary Infiltration Feasibility Evaluation" Aspect Consulting, June 2, 2021
- "Supplemental Geotechnical Report Small Scale Infiltration Test" Leroy Surveyors and Engineers, Inc., January 6, 2022

These reports were prepared primarily to evaluate stormwater infiltration feasibility at the site.

GeoEngineers prior work at the site also includes completing a Phase 1 Environmental Site assessment for the City of Puyallup (report dated September 15, 2011). This report can be provided for review, if requested.

3.3. Subsurface Conditions

3.3.1. Soil Conditions

As part of GeoEngineers 2011 report, three borings were advanced at the site to depths between 21.5 feet and 80 feet below ground surface (bgs). The locations of these borings are shown on the Site Plan, Figure 2 and the summary explorations logs are included in Appendix A. Borings B-1 and B-2 for this study were completed as monitoring wells; details of well construction are also included in Appendix A. Additional borings were not completed as part of the Aspect Consulting and Leroy Surveyors Reports. A shallow excavation for an infiltration test was completed as part of the Leroy Surveyors report. The location of the infiltration test is also shown on the Site Plan.

The borings completed for the 2011 report were advanced in areas surfaced with asphalt concrete. Asphalt thickness was on the order of 2 inches and was underlain by about 2 inches of base course. Below the asphalt, soil conditions described generally consisted of fill underlain by alluvium.

Fill extended approximately 2 to 5 feet below the ground surface. Fill consisted of brown silty sand and sandy silt in a moist condition and was typically in a loose or soft condition.

Alluvium underlying the fill generally consisted of layers of silt, silty sand, and sand with silt. Within about 20 feet of the ground surface, the alluvium was typically very loose to loose (or very soft to medium stiff). Below about 20 feet the relative density of the alluvium generally increased and was typically medium dense to dense, however intermittent layers of loose soil conditions were also noted. B-1 and B-2 were terminated around 21.5 feet bgs. B-3 was terminated around 80 feet bgs.

3.3.2. Groundwater Conditions

Groundwater was reported between 6 and 7 feet at the time of drilling. Groundwater monitoring in the B-1 and B-2 monitoring wells was completed by Aspect Consulting between December 8, 2020 and May 11, 2021. During that timeframe, seasonal high groundwater levels were measured between 3.5 and 4.5 feet bgs. A plot of groundwater levels provided in the Aspect Consulting Report is included as Figure 3 for reference.

We expect that groundwater levels will fluctuate throughout the year but will typically be within 3 to 7 feet of the ground surface. This interpretation is consistent with the groundwater monitoring competed by Aspect Consulting and our experience in the area.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1. Seismic Design Considerations

4.1.1. Seismic Design Parameters

We understand that seismic design will be completed using procedures outlined in the 2018 International Building Code (IBC). Per the 2018 IBC, structures shall be designed and constructed to resist the effects of earthquake motions in accordance with American Society of Civil Engineers (ASCE) 7-16.

As discussed below, the alluvial soils at the site are potentially liquefiable during the design seismic event. Due to the presence of potentially liquefiable soils, the site is classified as Site Class F, and a site-specific response analysis could be required.

However, an exception is provided in ASCE 7-16 Section 20.3.1. Site-specific response analysis is not required for liquefiable soils, provided the structure has a fundamental period of vibration equal or less than 0.5 seconds. Provided this exception is true, the site-specific response spectrum for Site Class D may be used as a basis for a simplified design and analysis.

Additionally, in accordance with ASCE 7-16 Section 11.4.8, a ground motion hazard analysis is required for sites classified as Site Class D and because the spectral response acceleration at 1-second periods (S_1) is greater than or equal to 0.2. However, an exception is allowed, provided specific requirements are satisfied, related to the fundamental period of the considered structure.

Table 1 below provides recommended seismic design parameters for Site Class D. These values are only valid if the exceptions provided in ASCE 7-16 Sections 11.4.8 and 20.3.1 described apply to the structures. If these expectations do not apply, we should be consulted further as a site-specific response analysis could be required.

TABLE 1. RECOMMENDED SEISMIC DESIGN PARAMETERS

2018 IBC (ASCE 7-16) Seismic Design Parameters	Recommended Value[1,2,3]
Site Class	D
Mapped Spectral Response Acceleration at Short Period (S_S)	1.273 g
Mapped Spectral Response Acceleration at 1 Second Period (S_1)	0.438 g
Site Amplification Factor at 0.2 second period (F_a)	1.0
Site Amplification Factor at 1.0 second period (F_v)	1.862
Design Spectral Acceleration at 0.2 second period (S_{DS})	0.849 g
Design Spectral Acceleration at 1.0 second period (S_{D1})	0.544 g
Site Modified Peak Ground Acceleration (PGA_M)	0.55 g

Notes:

[1] Parameters developed based on Latitude 47.189333307° and Longitude -122.296787743°.

[2] These values are only valid for structures with fundamental periods less than 0.5 seconds.

[3] A ground motion hazard analysis may be required in accordance with Section 11.4.8 of ASCE 7-16 (Site Class D and $S_1 \geq 0.2$).

4.1.2. Liquefaction

Liquefaction refers to a condition where vibration or shaking of the ground, usually from earthquake forces, results in development of excess pore pressures in saturated soils and a subsequent loss of soil strength. In general, soils that are susceptible to liquefaction include loose to medium dense “clean” to silty sands and non-plastic silts that are below the water table. We evaluated the soil profile for liquefaction potential using methods developed by Idriss and Boulanger (2008). This method compares the predicted cyclic shear stress (CSS) induced by the design earthquake to the cyclic shear resistance (CSR) determined by correlations with standard penetration test (SPT) blow counts. The ratio of the CSR to the CSS is the cyclic shear ratio and is considered the factor of safety against liquefaction.

Based on the results of our liquefaction analysis, the alluvium at the site is, in our opinion, potentially liquefiable. Based on the conditions described on the B-3 boring log, the bottom of the potentially liquefiable soils appears to be around 60 feet bgs.

Our analyses indicates that between about 12 and 18 inches of liquefaction-induced settlement could occur within the upper 60 feet of the soil profile during the design seismic event. Due to the variability of underlying soils and the inherent unpredictability of seismic soil liquefaction, differential settlements could be more than half to equal the total estimated settlement between similarly loaded foundations within a distance greater than about 50 to 100 feet apart.

4.1.3. Lateral Spreading Potential

Lateral spreading related to seismic activity typically involves lateral displacement of large, surficial blocks of non-liquefied soil when a layer of underlying soil loses strength during seismic shaking. Lateral spreading usually develops in areas where sloping ground or large grade changes (including retaining walls) are present. Based on the relatively flat topography of the site, our understanding of the liquefaction risk at the site, and the proposed improvements, it is our opinion that the risk of lateral spreading is low.

4.1.4. Surface Rupture

According to the Washington State Department of Natural Resources Interactive Natural Hazards Map (accessed January 31, 2022), there are no mapped faults or other seismogenic features within about 1 mile of the site. Based on the distance to the nearest mapped fault or seismogenic feature, it is our opinion the risk for surface rupture at this site is low.

4.2. Foundation Support

4.2.1. General

We expect that the estimated liquefaction settlement magnitudes will be excessive from a structural perspective and that liquification mitigation or alternative foundation support methods will be necessary. Based on conversations with you, we understand that your preferred approach to foundation support is conventional shallow foundations underlain by ground improvement. Alternatively, we expect that the proposed structure could be supported on deep foundations (driven piles, augercast piles, drilled shafts, etc.). The sections below provide recommendations for design of ground improvement and shallow foundations located within ground improvement areas and outside of ground improvement areas. We can provide recommendations for design of other foundation support methods, if requested.

4.2.2. Ground Improvement

4.2.2.1. General

We understand that compacted aggregate piers (CAPs), is the current ground improvement method proposed for this site. CAPs, which are often referred to by a trade name, GeoPiers or Rammed Aggregate Piers. CAPs consist of discrete columns of compacted crushed rock that are installed on a regular pattern below the proposed improvements, typically a building footprint. There are several benefits that can be achieved by installing CAPs. CAPs can reduce the magnitude of static settlement, increase the allowable soil bearing resistance and reduce the magnitude of total and differential settlement caused by liquefaction. Other ground improvement types including stone columns, or rigid inclusions which are also be feasible for this site. Because many ground improvement methods are proprietary designs, we recommend that the ground improvement system be designed by the ground improvement contractor selected to perform the work. The design criteria for the ground improvement system are summarized in the section below.

4.2.2.2. Ground Improvement Design Criteria

The primary intent of the ground improvement design should be to mitigate the liquefaction settlement hazard and provide an increased bearing resistance for the proposed structure. The ground improvement should encompass the entire building footprint and extend at least 5 feet beyond the footprint of the structure as well as below any other critical/settlement sensitive infrastructure proposed outside of the main structure. We recommend the design of the ground improvement, including the actual layout, length and minimum diameter of each column or pier based on the final foundation plan. The ground improvement designer may determine the required depth of the ground improvement based on the design criteria provided below. We recommend minimum ground improvement elements be at least 30 feet below primary bearing surfaces such as building slabs and foundations. Some alternative depths could be appropriate depending on type, spacing and diameter.

We recommend that the ground improvement be designed to achieve the following minimum performance criteria. It is possible to design ground improvement to achieve higher allowable bearing capacities and less settlement. If a higher level of performance is required for the ground improvement, we should be notified to review the specific application and design prior to preparation of final construction documents. The performance criteria below must be reviewed by the project structural engineer who should confirm that the criteria is appropriate for the proposed building and provide revised performance criteria, if necessary.

- Allowable soil bearing resistance of 3,000 pounds per square foot (psf) with an allowable increase of ⅓ for transient loading conditions.
- Total long-term static settlement of 1 inch and differential static settlement of 0.5 inch over a distance of 40 feet.
- Total liquefaction-induced settlement of 4 inches for the improved area.
- Differential liquefaction-induced settlement of 2 inches over a distance of 40 feet; some variations of this minimum may be accommodated by the structure and with structural design; we suggest we assist with additional review for these cases.

The contractor performing the work should provide adequate verification that the specified design criteria has been achieved after ground improvement installation. This could include modulus tests to verify the specified bearing resistance was achieved and pre-treatment and post-treatment cone penetrometer tests (CPTs) to verify that the specified liquefaction mitigation was achieved. Post treatment performance criteria should be required as part of the project plans and specifications and contractor submittal requirements. We can and recommend we assist with specifications and/or criteria for verification of post treated soil and specific bearing resistance or alternatively, we recommend we review proposed designers' performance verification criteria.

4.2.3. Foundation Support Within Ground Improvement

4.2.3.1. General

The foundation support recommendations provided below assume that ground improvement designed to meet the performance criteria specified above is installed below the proposed structure. We have also developed recommendations for design of foundations outside of the ground improvement area. We recommend a minimum footing width of 1.5 feet for continuous wall footings and 2 feet of isolated column footings. All footing elements should be embedded at least 18 inches below the lowest adjacent external grade.

4.2.3.2. Bearing Surface Preparation

Depending on the ground improvement method selected, shallow foundations will either bear directly on top of the exposed ground improvement elements, or on a load transfer pad that will be specified in the ground improvement design. Load transfer pads typically consist of a few feet of compacted structural fill installed between the top of the ground improvement elements and the design bottom of footing elevation or other structural bearing element. In either case, we recommend that foundation bearing surfaces be proof compacted in place to a uniformly firm and unyielding condition prior to placement of formwork or rebar. Loose or disturbed materials present at the base of footing excavations should be removed or compacted. Prepared foundation bearing surfaces should be observed and evaluated by a member of our firm prior to placement of formwork or steel reinforcement. Our representative will confirm that the bearing surfaces have been prepared in accordance with our recommendations and the project documents.

4.2.3.3. Allowable Soil Bearing Resistance

Provided ground improvement meeting the design criteria described above is installed at the site we recommend that foundations for the proposed structures within the ground improvement be designed assuming an allowable soil bearing resistance of 3,000 psf. The provided bearing pressures apply to the total of dead and long-term live loads and may be increased by one-third when considering total loads, including earthquake or wind loads. These are net bearing pressures. The weight of the footing and overlying backfill can be ignored in calculating footing sizes. The ground improvement designer must confirm that the minimum allowable bearing pressure stated above is achievable with their proposed design. Some designs may yield and attain higher values. This should be reviewed by project geotechnical and structural engineers.

4.2.3.4. Foundation Static Settlement

We estimate that static settlement of footings designed and constructed as recommended will be less than 1 inch, with differential settlements of less than ½ inch between comparably loaded isolated column footings or along 50 feet of continuous footing. These settlement estimates must be confirmed by the ground improvement designer. We estimate that liquefaction induced settlements will be as described previously.

4.2.3.5. Lateral Resistance

The ability of the soil to resist lateral loads is a function of frictional resistance, which can develop on the base of footings and slabs and passive resistance, which can develop on the face of below-grade elements of the structure as these elements tend to move into the soil. The allowable frictional resistance on the base of the footing may be computed using a coefficient of friction of 0.40 applied to the vertical dead-load forces. The allowable passive resistance on the face of the footing or other embedded foundation elements may be computed using an equivalent fluid density of 275 pounds per cubic foot (pcf) for undisturbed site soils or structural fill extending out from the face of the foundation element a distance at least equal to two and one-half times the depth of the element. These values include a factor of safety of about 1.5.

The passive earth pressure and friction components may be combined provided that the passive component does not exceed two-thirds of the total. The passive earth pressure value is based on the assumptions that the adjacent grade is level and that groundwater remains below the base of the footing throughout the year. The top foot of soil should be neglected when calculating passive lateral earth pressure unless the area adjacent to the foundation is covered with pavement or a slab-on-grade.

4.2.3.6. Footing Drains

We recommend that perimeter foundation drains be installed at the base of exterior footings. The perimeter drains should be provided with cleanouts and at minimum, should consist of a 4-inch-diameter perforated pipe surrounded on all sides by 6 inches of drain material enclosed in a non-woven geotextile fabric for underground drainage to prevent fine soil from migrating into the drain material. We recommend that the drainpipe consist of either heavy-wall solid pipe or rigid corrugated smooth interior polyethylene pipe. We do not recommend using flexible tubing for footing drainpipes. The drain material should consist of pea gravel or material similar to “Gravel Backfill for Drains” per WSDOT Standard Specifications Section 9-03.12(4). The perimeter drains should be sloped to drain by gravity, if practical, to a suitable discharge point. Water collected in roof downspout lines must not be routed to the perimeter footing drains.

4.2.4. Foundations Outside of Ground Improvement Zone

Small, non-critical structures that can tolerate differential settlements during a seismic event without risking life safety or the functionality of the primary structure can be supported on shallow foundations without ground improvement. We recommend that foundations in areas outside of the ground improvement zone be underlain by at least an 18-inch-thick layer of compacted structural fill. Foundation bearing surfaces should be thoroughly compacted to a dense, non-yielding condition. Loose or disturbed materials present at the base of foundation excavations should be removed or compacted. Foundation bearing surfaces should not be exposed to standing water. Should water infiltrate and pool in the excavation, it should be removed and surface repaired before placing structural fill or reinforcing steel.

We recommend that footings in non-ground improvement areas with bearing surfaces prepared as described above be proportioned using an allowable soil bearing pressure of 2,000 psf. This is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. We estimate that settlements of footings due to static column loads less than about 30 kips will be less than 1 inch. We estimate that differential settlements across the base of foundations will be less than ½ inch. These estimates are exclusive of settlement resulting from fill placed to raise site grades. The lateral resistance parameters provided previously can also be used for design of footings located outside of ground improvement areas.

4.2.5. Slab on Grade Floors

We understand that the ground level of the structure will be used for vehicle parking and large at grade building slabs are not envisioned. We expect that relatively small slab on grade floors will be included at ground level for entrances and lobby areas. It is also possible that the ground level parking area pavements will be designed as a slab on grade or mat foundation for structural reasons. We recommend that ground improvement be included below parking areas that are within the building footprint and below ground level slab on grade floors.

We recommend that the slab subgrades be prepared in accordance with Section 4.6.6 "Subgrade Preparation" of this report and that the slab be underlain by at least 8 inches of capillary break material consisting of crushed surfacing base course (CSBC) conforming 9-03.9(3) of the Washington State Department of Transportation (WSDOT) Standard Specifications with the exception that the percent of material passing the No.200 sieve should be less than 5 percent.

Provided that loose soil is removed and the subgrade is prepared as recommended, we recommend slabs-on-grade be designed using a modulus of subgrade reaction of 300 pounds per cubic inch (pci). We estimate that settlement for slabs-on-grade with improved ground constructed as recommended will be less than ¾ inch for a floor load of 500 psf.

4.3. Retaining Walls and Below-Grade Structures

4.3.1. Design Parameters

We recommend the following lateral earth pressures be used for design of conventional retaining walls and below-grade structures up to about 10 feet in height. Our design pressures assume that the ground surface around the structures will be level or near level. If drained design parameters are used, drainage systems must be included in the design in accordance with the recommendations presented in the "Drainage" section below.

- Active soil pressure may be estimated using an equivalent fluid density of 35 pcf for the drained condition.
- Active soil pressure may be estimated using an equivalent fluid density of 80 pcf for the undrained condition; this value includes hydrostatic pressures.
- At-rest soil pressure may be estimated using an equivalent fluid density of 55 pcf for the drained condition.
- At-rest soil pressure may be estimated using an equivalent fluid density of 90 pcf for the undrained condition; this value includes hydrostatic pressures.
- For seismic considerations, a uniform lateral pressure of 11H psf (where H is the height of the retaining structure or the depth of a structure below ground surface) should be added to the lateral earth pressure.

- Active soil pressure condition assumes the wall is free to move laterally 0.001 H, where H is the wall height). The at-rest condition is applicable where walls are restrained from movement.
- For backfill sloping conditions up to 2H:1V, the soil pressures presented above should be increased by 15 percent.
- A typical traffic surcharge representing an additional 2 feet of fill equal to 250 psf should be included if vehicles are allowed to operate within ½ the height of the retaining walls.
- Other surcharge and backfill conditions can increase the magnitude of the loads upon the wall requiring alternative design considerations. We should be consulted if other surcharge or backfill conditions will be considered above retaining walls. Examples of other loading conditions may include nearby structures, construction equipment and stockpiled soil or materials.

Over-compaction of fill placed directly behind retaining walls or below-grade structures must be avoided. We recommend use of hand-operated compaction equipment and maximum 6-inch loose lift thickness when compacting fill within about 5 feet of retaining walls and below-grade structures.

Retaining wall foundation bearing surfaces should be prepared following Section "4.2 Foundation Support" of this report. Provided bearing surfaces are prepared as recommended retaining wall foundations may be designed using the allowable soil bearing values and lateral resistance values presented above. In general, we estimate settlement of retaining structures will be similar to the values previously presented for spread foundations.

In applications where retaining walls are designed as a fill wall and fill soil is added behind the wall to generate new grade and the new grade, or height of the wall exceeds about 4 to 5 feet, there is a potential for additional static settlement if subsurface soil below the retaining wall if unimproved. We recommend we provide further review of this specific situation where the wall becomes greater than about 4 feet, will retain new fill, and be on unimproved ground. A specific overexcavation depth and possibly a pre-load could be required for this specific situation and will be based, in part on the new fill and depths placed.

4.3.2. Drainage

If retaining walls or below-grade structures are designed using drained parameters, a drainage system behind the structure must be constructed to collect water and prevent the buildup of hydrostatic pressure against the structure. We recommend the drainage system include a zone of free-draining backfill a minimum of 18 inches in width against the back of the wall. The drainage material should consist of coarse sand and gravel containing less that 5 percent fines based on the fraction of material passing the ¾-inch sieve. Other systems, such as waffle drain boards may also be considered. Drainage products should be reviewed to determine adequate coverage, drainage flow and proper connection to outlets.

A perforated, rigid, smooth-walled drainpipe with a minimum diameter of 4 inches should be placed along the base of the structure within the free-draining backfill and extend for the entire wall length. The drainpipe should be metal or rigid PVC pipe and be sloped to drain by gravity. Discharge should be routed properly to reduce erosion potential.

Cleanouts should be provided to allow routine maintenance. We recommend roof downspouts or other types of drainage systems not be connected to retaining wall drain systems

4.4. Pavement Design

4.4.1. General

Paved areas are expected to include parking areas, driveways and sidewalk areas. Based on our experience, we provide recommended conventional asphalt concrete pavement (ACP) and Portland cement concrete (PCC) sections below. These pavement sections may not be adequate for heavy construction traffic loads such as those imposed by concrete transit mixers, dump trucks or cranes. Additional pavement thickness may be necessary to prevent pavement damage during construction if other loading types are planned. The recommended sections assume that final improvements surrounding the pavements will be designed and constructed such that stormwater or excess irrigation water from landscape areas does not accumulate below the pavement section or pond on pavement surfaces.

Existing pavements, hardscaping or other structural elements should be removed prior to placement of new pavement sections. Pavement subgrade should be prepared as recommended in Section "4.4.6 Subgrade Preparation" of this report. Crushed surfacing base course and subbase should be moisture conditioned to near optimum moisture content and compacted to at least 95 percent of the theoretical MDD per ASTM D 1557.

CSBC and crushed surfacing top course (CSTC) should conform to applicable sections of 4-04 and 9-03.9(3) of the WSDOT Standard Specifications. The top approximate 2 inches of the CSBC sections provided may consist of CSTC as a leveling layer and for more precise grade development.

Hot mix asphalt should conform to applicable sections of 5-04, 9-02 and 9-03 of the WSDOT Standard Specifications.

PCC mix design should conform with Section 5-05.3(1) of the WSDOT Standard Specifications. Aggregates for PCC should conform to applicable sections of 9-03.1 of the WSDOT Standard Specifications.

Some areas of pavement may exhibit settlement and subsequent cracking over time. Cracks in the pavement will allow water to infiltrate to the underlying base course, which could increase the amount of pavement damage caused by traffic loads. To prolong the effective life of the pavement, cracks should be sealed as soon as possible.

4.4.2. Asphalt Concrete Pavement Sections

Recommended minimum ACP sections are provided below.

4.4.2.1. Standard-Duty ACP – Automobile Driveways and Parking Areas

- 2 inches of hot mix asphalt, class ½ inch, PG 58-22
- 4 inches of compacted CSBC
- 6 inches of subbase consisting of imported granular structural fill to provide uniform grading and pavement support, to maintain drainage, and to provide separation from fine-grained subgrade soil
- Native soil, existing fill or structural fill prepared as recommended in Section "4.5.6 Subgrade Preparation" of this report

4.4.2.2. *Heavy-Duty ACP – Areas Subject to Heavy-Duty Traffic*

- 3 inches of hot mix asphalt, class ½ inch, PG 58-22
- 6 inches of compacted CSBC
- 6 inches of subbase consisting of imported granular structural fill to provide uniform grading and pavement support, to maintain drainage, and to provide separation from fine-grained subgrade soil
- Native soil, existing fill or structural fill prepared as recommended in Section "4.5.6 Subgrade Preparation" of this report

4.4.3. Portland Cement Concrete Pavement Design

Recommended minimum PCC pavement sections are provided below. In our opinion steel reinforcement does not need to be included in PCC pavements that will be primarily used in landscaping and pedestrian areas (areas not subjected to heavy vehicle traffic). Reinforcement could be considered to reduce the potential for cracking in areas where the concrete slabs have irregular shapes or where new slabs abut existing concrete slabs, and the joint layout between the slabs cannot be matched. If reinforcement is considered, we are available to discuss typical steel reinforcement volumes with the project structural engineer, who ultimately designs the location, size and layout of reinforcement.

4.4.3.1. *Sidewalk PCC Pavement – Pedestrian Areas Not Subjected to Vehicle Loading*

- 4 inches of PCC with a minimum 14-day flexural strength of 650 pounds per square inch (psi)
- 2 inches of compacted CSBC
- Native subgrade or structural fill prepared in accordance with Section "4.5.6 Subgrade Preparation" of this report

4.4.3.2. *Standard PCC Pavement – Automobile Driveways and Parking Areas*

- 6 inches of PCC with a minimum 14-day flexural strength of 650 psi
- 4 inches of compacted CSBC
- Native subgrade, existing fill or structural fill prepared in accordance with Section "4.5.6 Subgrade Preparation" of this report

4.4.3.3. *Heavy Duty PCC Pavement – Areas Subject to Heavy Truck Traffic*

- 9 inches (minimum) of PCC with a minimum 14-day flexural strength of 650 psi
- 4 inches of compacted CSBC
- Native subgrade, existing fill or structural fill prepared in accordance with Section "4.5.6 Subgrade Preparation" of this report.

4.5. Earthwork

4.5.1. General

We anticipate that site development and earthwork will include demolition of existing features, excavating for shallow foundations, utilities, and other improvements, establishing subgrades for structures and hardscaping, and placing and compacting fill and backfill materials. We expect that site grading and earthwork can be accomplished with conventional earthmoving equipment. We strongly recommend that site development and earthwork activities be scheduled during dry weather months when groundwater levels will be at their lowest. The following sections provide our recommendations for earthwork activities at the site.

4.5.2. Clearing, Stripping and Demolition

We recommend that existing pavements and hardscaping be completely removed from areas that will be developed. During removal and/or demolition, excessive disturbance of surficial soils may occur, especially if left exposed to wet conditions. Disturbed and demolition areas may require additional remediation during construction and grading.

Within landscaped areas, stripping depths on the order of 3 to 6 inches should be expected. The primary root system of trees and shrubs should be removed during stripping activities. Stripped material should

If existing utilities exist beneath new structures, they should be removed and the area backfilled, if practical, or abandoned in place. Abandonment can include filling or pumping using a controlled density fill or other approved flowable fill material that will fill the utility cavity completely and offer support similar to backfill soil. Utility use, ownership and rights of way should also be considered.

4.5.3. Erosion and Sedimentation Control

Erosion and sedimentation rates and quantities can be influenced by construction methods, slope length and gradient, amount of soil exposed and/or disturbed, soil type, construction sequencing and weather. Implementing an Erosion and Sedimentation Control Plan will reduce the project impact on erosion-prone areas. The plan should be designed in accordance with applicable city, county and/or state standards. The plan should incorporate basic planning principles, including:

- Scheduling grading and construction to reduce soil exposure;
- Re-vegetating or mulching denuded areas;
- Directing runoff away from exposed soils;
- Reducing the length and steepness of slopes with exposed soils;
- Decreasing runoff velocities;
- Preparing drainage ways and outlets to handle concentrated or increased runoff;
- Confining sediment to the project site; and
- Inspecting and maintaining control measures frequently.

Some sloughing and raveling of exposed or disturbed soil on slopes should be expected. We recommend that disturbed soil be restored promptly so that surface runoff does not become channeled.

Temporary erosion protection should be used and maintained in areas with exposed or disturbed soils to help reduce erosion and reduce transport of sediment to adjacent areas and receiving waters. Permanent erosion protection should be provided by paving, structure construction or landscape planting.

Until the permanent erosion protection is established, and the site is stabilized, site monitoring may be required by qualified personnel to evaluate the effectiveness of the erosion control measures and to repair and/or modify them as appropriate. Provisions for modifications to the erosion control system based on monitoring observations should be included in the Erosion and Sedimentation Control Plan.

4.5.4. Temporary Excavations and Dewatering

Excavations deeper than 4 feet must be shored or laid back at a stable slope if workers are required to enter. Shoring and temporary slope inclinations must conform to the provisions of Title 296 Washington Administrative Code (WAC), Part N, “Excavation, Trenching and Shoring.” Regardless of the soil type encountered in the excavation, shoring, trench boxes or sloped sidewalls will be required under Washington Industrial Safety and Health Act (WISHA).

In general, temporary cut slopes at this site should be inclined no steeper than about 1½H to 1V (horizontal to vertical). This guideline assumes that all surface loads are kept at a minimum distance of at least one-half the depth of the cut away from the top of the slope and that seepage is not present on the slope face. We expect that flatter slopes or shoring will be necessary when excavating below the water table which is expected to be present between 3 to 5 feet below ground surface.

We anticipate that dewatering will typically be required to complete excavations extending deeper than 5 feet below existing site grade. If the planned excavation is completed during dry weather months, is only extended a few feet below the groundwater table and will remain open for a short period of time, managing groundwater inflow using sump pumps could be feasible. We expect that dewatering will be necessary to complete deeper excavations at the site or excavations that will remain open for an extended period of time.

Excavation, shoring, and dewatering are interrelated; the design and implementation of these elements must be coordinated and must consider the over-all construction staging to ensure a consistent and compatible approach. We recommend that the contractor performing the work be made responsible for designing and installing construction shoring and for controlling and collecting groundwater encountered. The contract documents must specify that the contractor is responsible for selecting excavation and dewatering methods, monitoring the excavations for safety, and providing shoring, as required, to protect personnel and structures.

4.5.5. Surface Drainage

Surface water from roofs, pavements and landscape areas should be collected and controlled. Curbs or other appropriate measures such as sloping pavements, sidewalks and landscape areas should be used to direct surface flow away from buildings, erosion sensitive areas and from behind retaining structures. Roof and catchment drains should not be connected to wall or foundation drains.

4.5.6. Subgrade Preparation

Subgrades that will support slab-on-grade floors and pavements should be thoroughly compacted to a uniformly firm and unyielding condition on completion of stripping/excavation and before placing structural fill. We recommend that subgrades for structures and pavements be evaluated, as appropriate, to identify areas of yielding or soft soil. Probing with a steel probe rod or proof-rolling with a heavy piece of wheeled construction equipment are appropriate methods of evaluation.

If soft or otherwise unsuitable subgrade areas are revealed during evaluation that cannot be compacted to a stable and uniformly firm condition, we recommend that: (1) the unsuitable soils be scarified (e.g., with a ripper or farmer’s disc), aerated and recompacted, if practical; or (2) the unsuitable soils be removed and replaced with compacted structural fill, as needed.

4.5.7. Subgrade Protection and Wet Weather Considerations

The wet weather season generally begins in October and continues through May in Western Washington; however, periods of wet weather can occur during any month of the year. The soils encountered in our explorations contain a significant amount of fines. Soil with high fines content is very sensitive to small changes in moisture and is susceptible to disturbance from construction traffic when wet or if earthwork is performed during wet weather. If wet weather earthwork is unavoidable, we recommend that the following steps be taken.

- The ground surface in and around the work area should be sloped so that surface water is directed away from the work area. The ground surface should be graded so that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Earthwork activities should not take place during periods of heavy precipitation.
- Slopes with exposed soils should be covered with plastic sheeting.
- The contractor should take necessary measures to prevent on-site soils and other soils to be used as fill from becoming wet or unstable. These measures may include the use of plastic sheeting and controlling surface water with sumps with pumps and grading. The site soils should not be left uncompacted and exposed to moisture. Sealing the exposed soils by rolling with a smooth-drum roller prior to periods of precipitation will help reduce the extent to which these soils become wet or unstable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with working pad materials not susceptible to wet weather disturbance.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practical.
- During periods of wet weather, concrete should be placed as soon as practical after preparation of the footing excavations. Foundation bearing surfaces should not be exposed to standing water. If water pools in the base of the excavation, it should be removed before placing structural fill or reinforcing steel. If footing excavations are exposed to extended wet weather conditions, a lean concrete mat or a layer of clean crushed rock can be considered for foundation bearing surface protection.

4.6. Fill Materials

4.6.1. Structural Fill

The workability of material for use as structural fill will depend on the gradation and moisture content of the soil. We recommend that washed crushed rock or select granular fill, as described below, be used for structural fill during the rainy season. If prolonged dry weather prevails during the earthwork phase of construction, materials with a somewhat higher fines content may be acceptable. Weather, material use, schedule, duration exposed, and site conditions should be considered when determining the type of import fill materials purchased and brought to the site for use as structural fill.

Material used for structural fill should be free of debris, organic contaminants and rock fragments larger than 6 inches. For most applications, we recommend that structural fill material consist of material similar to “Select Borrow” or “Gravel Borrow” as described in Section 9-03.14 of the Washington State Department of Transportation (WSDOT) Standard Specifications.

4.6.2. Select Granular Fill/Wet Weather Fill

Select granular fill should consist of well-graded sand and gravel or crushed rock with a maximum particle size of 6 inches and less than 5 percent fines by weight based on the minus ¾-inch fraction. Organic matter, debris or other deleterious material should not be present. In our opinion, material with gradation characteristics similar to WSDOT Specification 9-03.9 (Aggregates for Ballast and Crushed Surfacing), “Gravel Backfill for Walls” as described in Section 9-03.12(2) of the WSDOT Standard Specifications, or Section 9-03.14 (Borrow) is suitable for use as select granular fill, provided that the fines content is less than 5 percent (based on the minus ¾-inch fraction) and the maximum particle size is 6 inches.

4.6.3. Pipe Bedding

Trench backfill for the bedding and pipe zone should consist of well-graded granular material similar to “gravel backfill for pipe zone bedding” described in Section 9-03.12(3) of the WSDOT Standard Specifications. The material must be free of roots, debris, organic matter and other deleterious material. Other materials may be appropriate depending on manufacturer specifications and/or local jurisdiction requirements.

4.6.4. Fill Material Below Groundwater Level

If fill or trench backfill will be placed below or near the groundwater level, we recommend imported material consisting of either permeable ballast or quarry spalls be used.

Permeable ballast should consist of material with gradation characteristics similar to WSDOT Standard Specification 9-03.9 (2). We recommend that quarry spalls consist of 2- to 4-inch washed, crushed stone similar to that described in Section 9-13 of the WSDOT Standard Specifications. Alternative stone size ranges may be considered, depending on the application and availability.

4.6.5. Drainage Zone Material

Free-draining backfill should comprise material similar to WSDOT Standard Specification 9-03.12(2) “Gravel Backfill for Walls.”

4.6.6. On-Site Soil

Existing site soils must not be used as base course, top course or as drainage material. Due to moisture content and fines content of existing site soil, in general, we recommend against use of on-site material as a structural fill. If still necessary, we recommend contingencies in the project budget be included for handling, drying, and/or amending site materials as well as importing granular structural fill. We recommend that a representative from GeoEngineers be on site during earthwork activities to evaluate if the existing soil generated during excavation is suitable for reuse and to provide alternative recommendations, if necessary.

The soils at the site contain a significant amount of fines and are extremely moisture sensitive and will be very difficult or impossible to properly compact when wet. Soils generated from below the water table will likely be saturated or at a moisture content above what is optimum for compaction. In this case, the soils would need to be moisture conditioned prior to re-use. Space for drying out material during dryer weather or covering on-site materials generated during wet weather will be necessary. During wetter or even slightly colder times of year, such as when temperatures reach below about 60 degrees, drying becomes more difficult and accommodations to cover and protect stockpiled material generated on-site for re-use should be planned. In many cases, covering of stockpiled material will not be sufficient to allow for the material to dry when near or below this temperature.

4.7. Fill Placement and Compaction

4.7.1. General

To obtain proper compaction, fill soil should be compacted near optimum moisture content and in uniform horizontal lifts. Lift thickness and compaction procedures will depend on the moisture content and gradation characteristics of the soil and the type of equipment used. The maximum allowable moisture content varies with the soil gradation and should be evaluated during construction. Generally, 8- to 12-inch loose lifts are appropriate for steel-drum vibratory roller compaction equipment. Thinner lifts are appropriate for smaller compaction equipment. Compaction should be achieved by mechanical means. During fill and backfill placement, sufficient testing of in-place density should be conducted to check that adequate compaction is being achieved.

4.7.2. Area Fills and Pavement Bases

Fill placed to raise site grades and materials under pavements and structural areas should be placed on subgrades prepared as previously recommended. Fill material placed below structures and footings should be compacted to at least 95 percent of the theoretical MDD per ASTM D 1557. Fill material placed shallower than 2 feet below pavement sections should be compacted to at least 95 percent of the MDD. Fill placed deeper than 2 feet below pavement sections should be compacted to at least 90 percent of the MDD. Fill material placed in landscaping areas should be compacted to a firm condition that will support construction equipment, as necessary, typically at least 85 to 90 percent of the MDD.

4.7.3. Backfill Behind Retaining Walls and Below-Grade Structures

Backfill behind retaining walls or below-grade structures should be compacted to between 90 and 92 percent of the MDD. Overcompaction of fill placed directly behind below-grade structures should be avoided. We recommend use of hand-operated compaction equipment and maximum 6-inch loose lift thickness when compacting fill within about 5 feet behind below-grade structures.

4.7.4. Trench Backfill

For utility excavations, we recommend that the initial lift of fill over the pipe be thick enough to reduce the potential for damage during compaction, but generally should not be greater than about 18 inches above the pipe. In addition, rock fragments greater than about 1 inch in maximum dimension should be excluded from this lift.

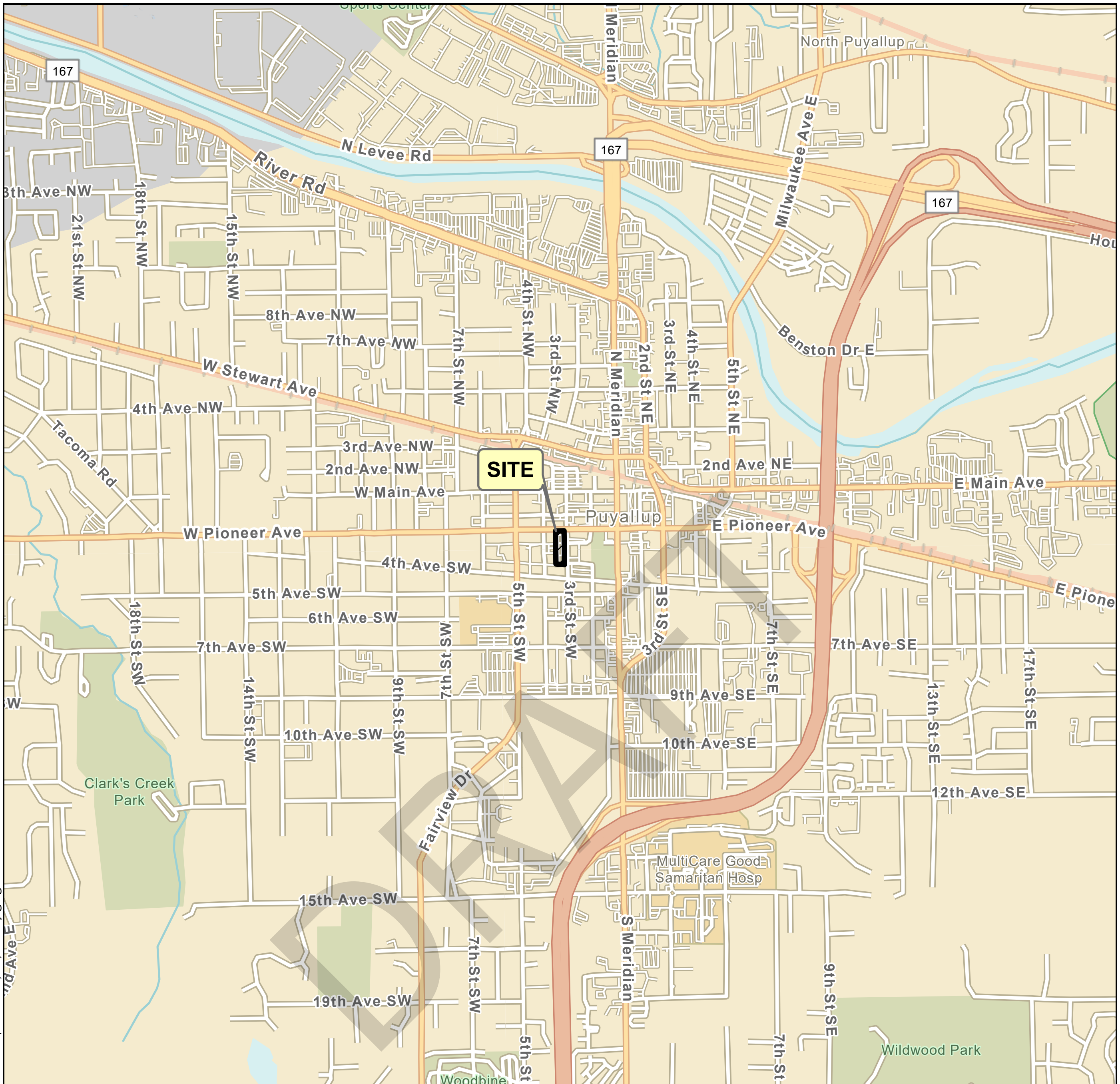
Trench backfill material placed below structures and footings should be compacted to at least 95 percent of the MDD. In paved areas, trench backfill should be uniformly compacted in horizontal lifts to at least 95 percent of the MDD in the upper 2 feet below subgrade. Fill placed below a depth of 2 feet from subgrade in paved areas must be compacted to at least 90 percent of the MDD. In non-structural areas, trench backfill should be compacted to a firm condition that will support construction equipment as necessary.

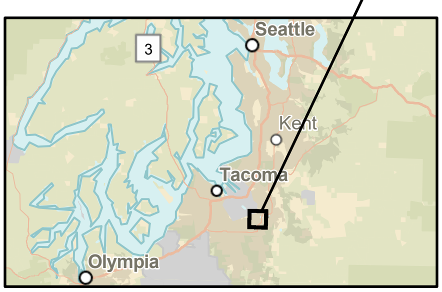
5.0 LIMITATIONS

We have prepared this report for MC Construction Consultants, for the Puyallup AOB Site project in Puyallup, Washington. MC Construction Consultants may distribute copies of this report to owner and owner's 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 for geotechnical engineering 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, express or implied, applies to the services or this report.

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





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Feet

Notes:

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

Data Source: ESRI

Projection: NAD 1983 UTM Zone 10N

Vicinity Map

Puyallup AOB Site
Puyallup, WA

GEOENGINEERS

Figure 1

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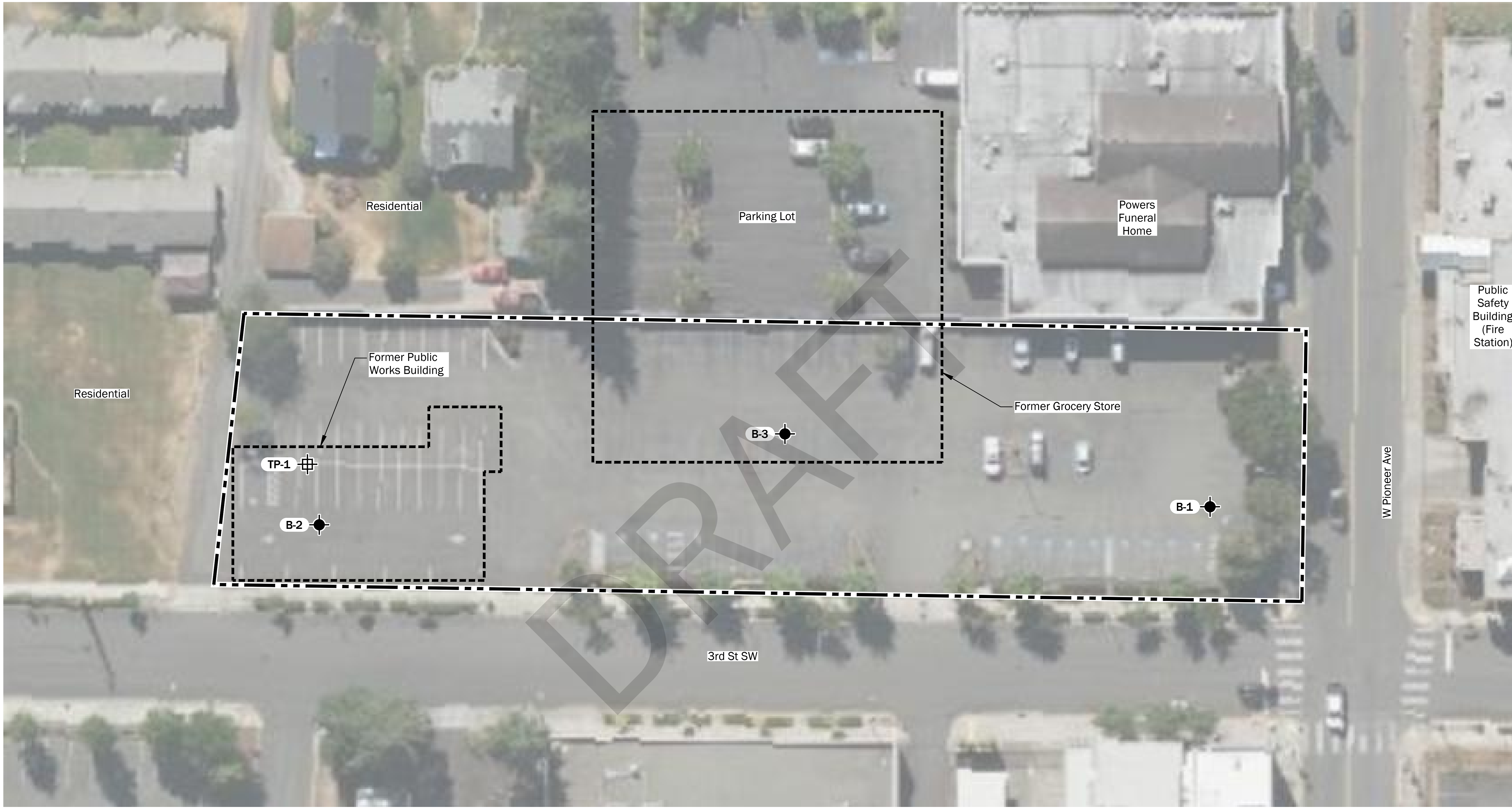
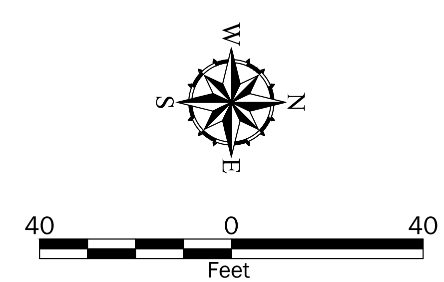


Legend

	Property Boundary
	Footprint of Former Building
TP-1	Test Pit by LS&E, 2022
B-1	Boring by GeoEngineers, Inc.



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: Aerial from Microsoft Bing Images.

Projection: Wahshington State Plane, South Zone, NAD83, US Foot

Site Plan

Puyallup AOB Site
Puyallup, WA

GeoEngineers

Figure 2

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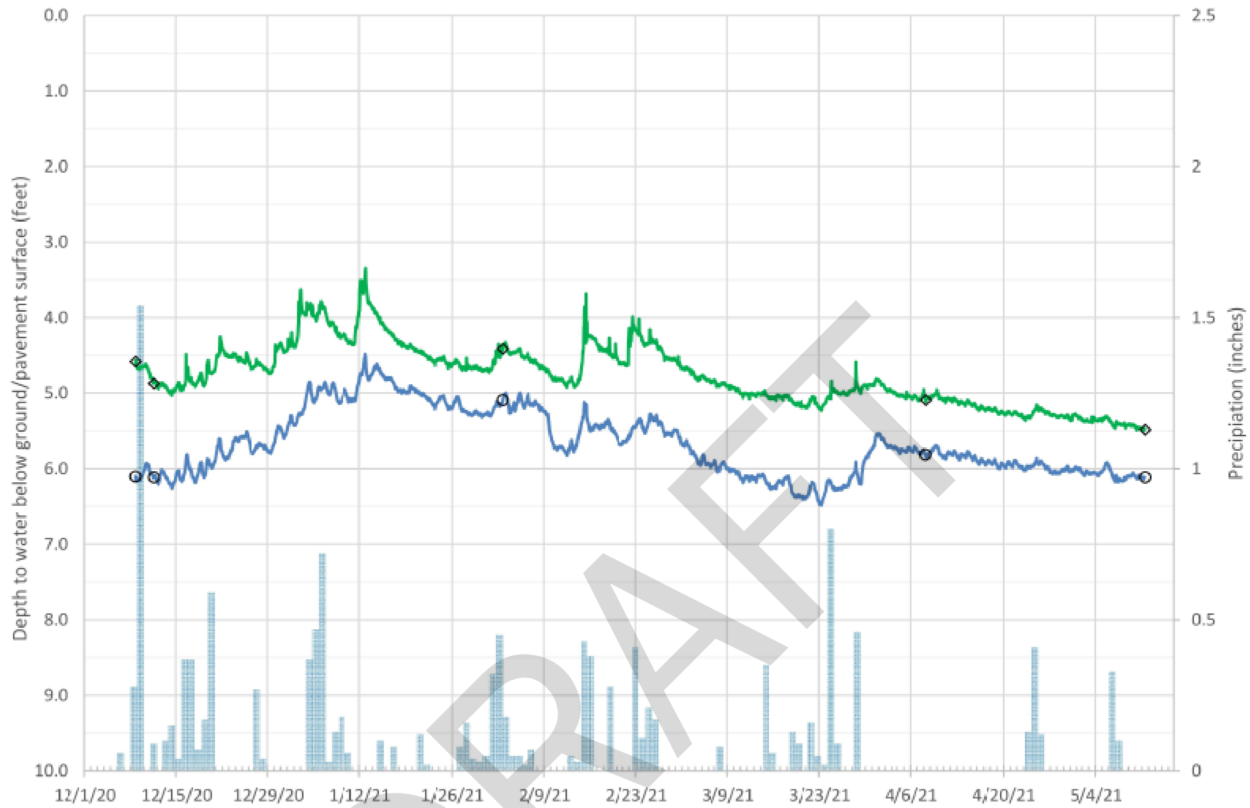


Legend

- Precipitation
- B-1
- B-2

Note:
bgs = feet below ground surface. Ground surface taken at top of existing monument cover.

Precipitation Date Source:
National Oceanic and Atmospheric Administration (NOAA)

Recorded Groundwater Levels

Groundwater Level Monitoring and
Preliminary Infiltration Feasibility Study
City of Puyallup AOB Parking Lot
330 3rd St NW, Puyallup, Washington

Aspect CONSULTING	May-2021	BY: ECS/CMV	FIGURE NO.
	PROJECT NO. 200589	REVISED BY: -	1

CAD Path: Q:\City of Puyallup\200589\200589-01 May 2021.dwg Fig 1 || Date Saved: May 19, 2021 2:09pm || User: cvanslyke

B-1 and B-1 Groundwater Plot

Puyallup AOB Site
Puyallup, Washington

GEOENGINEERS

Figure 3

Data Source: "Groundwater Level Monitoring and Preliminary Infiltration Feasibility Evaluation" Aspect Consulting, June 2, 2021

APPENDIX A
Boring Logs from 2011 GeoEngineers Report

SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Measured groundwater level in exploration, well, or piezometer

Groundwater observed at time of exploration

Perched water observed at time of exploration

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Material Description Contact

Distinct contact between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Sonic Core

Bulk or grab

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.

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

Laboratory / Field Tests

%F	Percent fines
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
OC	Organic content
PM	Permeability or hydraulic conductivity
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
NT	Not Tested

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

KEY TO EXPLORATION LOGS

GEOENGINEERS

FIGURE A-1

Drilled	Start	End	Total Depth (ft)	Logged By / Checked By	Driller	Drilling Method
	8/15/2011	8/15/2011	21.5	MJH / MJH	Holocene	HSA

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	BK-81
Surface Elevation (ft) Vertical Datum	Undetermined	Top of Casing Elevation (ft)	
Easting (X) Northing (Y)		Horizontal Datum	

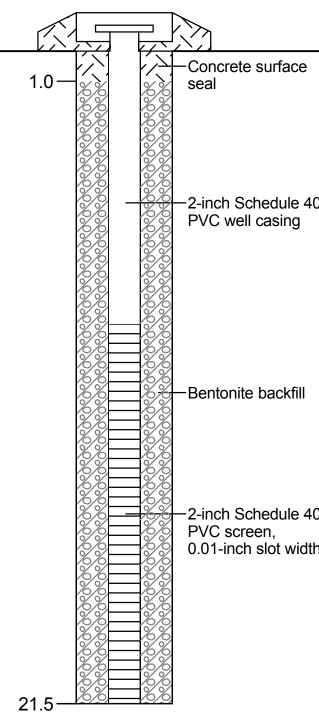
Licensing agency well number: **940**
A 2 (in) well was installed on to a depth of (ft).

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
9/15/2011	7.6	

Notes: Well No. 940

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)
	0	16	12	1 / SA	AC	2 inches asphalt concrete with 2 inches base course	6	
					SM	%F=16 Brown fine to medium silty sand (medium dense, moist) (fill)		
	5	18	2	2 & 3	ML	Brown silt with sand (soft, moist)		
					SP	Black fine to medium sand, trace silt (very loose, moist)		
					MH	Gray silt, trace sand and organics (soft, moist)		
	10	15	7	4	SP	Black fine to medium sand, trace silt (loose, wet)		
					ML	Green silt, trace sand (medium stiff, wet)		
					ML	Gray sandy silt (medium stiff, wet)		
	15	18	7	5 / MC	ML	Gray sandy silt interbedded with green silt, trace sand (loose to medium stiff, wet) %F=50	37	
	20	17	8	6 & 7				
					SP-SM	Gray fine to medium sand with silt (loose, wet)		

WELL LOG



Note: See Figure A-1 for explanation of symbols.

Log of Boring B-1

GEOENGINEERS

Project: City of Puyallup - AOB Site
Project Location: Puyallup, Washington
Project Number: 0402-030-00

Figure A-2
Sheet 1 of 1

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	8/15/2011	8/15/2011	21.5	MJH	MJH	Holocene	HSA

Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	BK-81
Surface Elevation (ft) Vertical Datum	Undetermined	Top of Casing Elevation (ft)	
Easting (X) Northing (Y)		Horizontal Datum	

Licensing agency well number: **941**
A 2 (in) well was installed on to a depth of (ft).

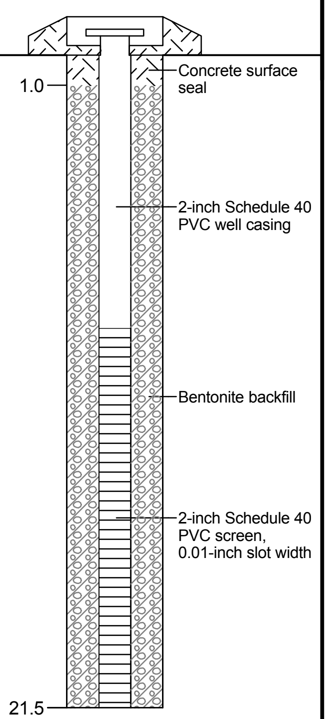
Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
8/15/2011	6.4	

Notes: Well No. 941

FIELD DATA

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Collected Sample	Sample Name	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)
	0	12	10		1		AC	2 inches asphalt concrete with 2 inches base course		
							SM	Brown silty fine to medium sand (medium dense, moist) (fill)		
		18	6		2 & 3 SA		ML	Gray mottled silt with sand (loose, moist) %F=83	32	
							SP-SM	Brown mottled fine to medium sand with silt (loose, moist)		
	5	18	1		4 MC	▼	ML	Gray silt with sand and trace organics (very soft, moist) %F=78	64	
	10	15	3		5	▽		Grades to soft and wet		
							SP-SM	Gray fine to medium sand with silt (loose, wet)		
	15	18	6		6 &7		ML	Gray silt with sand (medium stiff, wet)		
	20	18	7		8					

WELL LOG



Note: See Figure A-1 for explanation of symbols.

Log of Boring B-2

GEOENGINEERS

Project: City of Puyallup - AOB Site
Project Location: Puyallup, Washington
Project Number: 0402-030-00

Figure A-3
Sheet 1 of 1

Drilled	Start 8/15/2011	End 8/15/2011	Total Depth (ft)	80	Logged By MJH Checked By MJH	Driller Holocene	Drilling Method HSA

Surface Elevation (ft) Vertical Datum	Undetermined	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop
Drilling Equipment	BK-81		
Easting (X) Northing (Y)		System Datum	
Groundwater	Date Measured	Depth to Water (ft)	Elevation (ft)
Notes:			

FIELD DATA

Elevation (feet)	Depth (feet)	Interval / Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0							ML	Brown sandy silt (loose, moist) (fill)			
		12	4		1 / SA					29		%F=57
	5							ML/SP	Gray silt with sand interbedded with black sand, trace silt (loose to medium stiff, moist)			
		13	4		2							
	10							ML	Gray silt with sand and organics (1 inch thick wood) (soft, wet)			
		18	3		3 / MC					42		%F=93
	15							ML/SP	Gray silt, trace sand interbedded with black sand, trace silt (soft to very loose, wet)			
		14	3		4							
	20							ML	Gray silt with sand (medium stiff, wet)			
		14	6		5 & 6 / MC					33		%F=80
								SP-SM	Black fine to medium sand with silt (loose, wet)			
	25											
		14	12		7				Grades to medium dense			
	30											
		15	28		8 / MC				Grades to with occasional fine gravel, dense	23		%F=6
	35											
		15	32		9							
	40											

Note: See Figure A-1 for explanation of symbols.

Log of Boring B-3

GEOENGINEERS

Project: City of Puyallup - AOB Site
Project Location: Puyallup, Washington
Project Number: 0402-030-00

Figure A-4
Sheet 1 of 2

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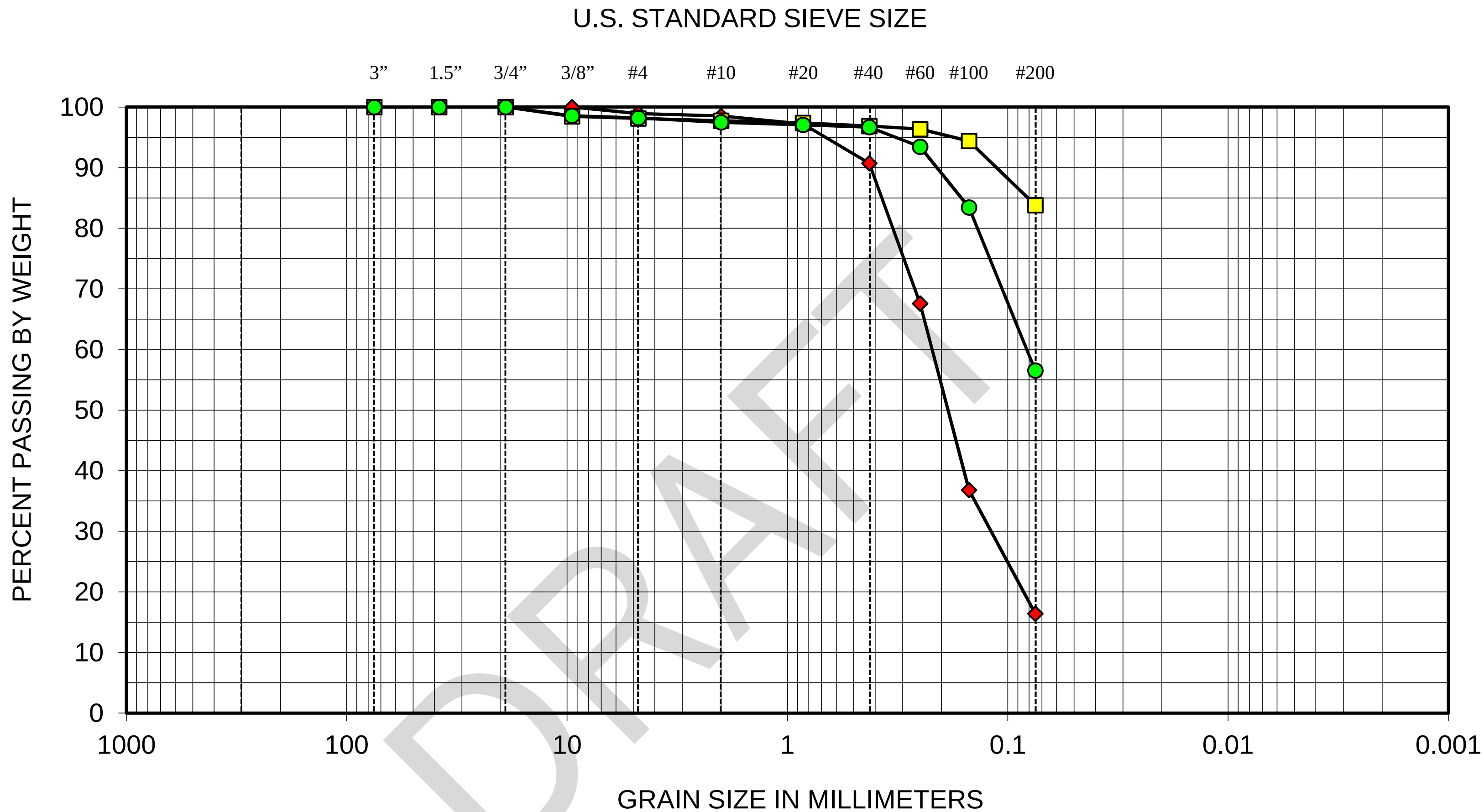
Log of Boring B-3 (continued)

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	40												
			10	35		10				Grades to with gravel			
	45												Driller indicates intermittent hard drilling from 45 to 50 feet
									SM	Gray silty fine to coarse sand with gravel (very loose, wet)			
			6	3		11							Rock in shoe tip
	50												
									GP-GM	Gray fine to coarse gravel with silt and sand (medium dense, wet)			
			2	16		12							
	55												
									SM	Gray silty fine to coarse sand, occasional gravel (loose wet)			
			15	6		13 / MC					19		%F=27
	60												
									SP	Black fine to medium sand, trace silt, occasional gravel (dense, wet)			
			10	35		14							
	65												
									SM	Gray silty fine to medium sand with gravel (dense, wet)			
			10	42		15 / MC					19		%F=33
	70												
									ML	Gray silt with sand (very stiff, wet)			
			16	16		16 / MC					37		%F=64
	75												
			15	42		17 & 18							
									SP	Gray fine sand, trace silt (dense, wet)			
	80												

Note: See Figure A-1 for explanation of symbols.

GEOENGINEERS

Project: City of Puyallup - AOB Site
Project Location: Puyallup, Washington
Project Number: 0402-030-00



BOULDERS	COBBLES	GRAVEL		SAND			SILT OR CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE	

SYMBOL	EXPLORATION NUMBER	DEPTH (ft)	MOISTURE (%)	SOIL CLASSIFICATION
◆	B-1	0	6	Silty sand (SM)
■	B-2	2.5	32	Silt with sand (ML)
●	B-3	2.5	29	Sandy silt (ML)

GeoEngineers

Sieve Analysis Results

City of Puyallup – AOB Site
Puyallup, Washington

Figure A-5

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 Services are Performed for Specific Purposes, Persons and Projects

This report has been prepared for MC Construction Consultants and for the Project(s) 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, our services have been executed in accordance with generally accepted geotechnical practices in this area at the time this report was prepared, and our Agreement with MC Construction Consultants dated February 22, 2022. We do 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 Puyallup AOB Site project located in Puyallup, 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.

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

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

- The function of the proposed structure;
- 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, we can provide written modifications or confirmation, as appropriate.

Environmental Concerns are Not Covered

Unless environmental services were specifically included in our 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.

Information Provided by Others

GeoEngineers has relied upon certain data or information provided or compiled by others in the performance of our services. Although we use sources that we reasonably believe to be trustworthy, GeoEngineers cannot warrant or guarantee the accuracy or completeness of information provided or compiled by others.

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 and Geologic Findings are Professional Opinions

Our interpretations of subsurface conditions are based on field observations 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. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

Geotechnical Engineering Report Recommendations are Not Final

We have 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 we do not perform construction observation.

We 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.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors 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

Our 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 specialized field.