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Updated Geotechnical Engineering Report

Allen Boulevard Development
Beaverton, Oregon

for
Oregon Worsted

August 5, 2020

GeoEngineers
Earth Science + Technology

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Allen Boulevard Development
Beaverton, Oregon

File No. 23648-001-00

August 5, 2020

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1.0 INTRODUCTION

GeoEngineers, Inc. (GeoEngineers) is pleased to submit this updated geotechnical engineering report to Oregon Worsted Company for the proposed Allen Boulevard Development. The location of the site is shown in Figure 1, Vicinity Map.

The proposed project includes four parcels, labeled Parcel A to Parcel D. Parcel A, located in the southwest portion of the site, is designated for a four-story hotel. Parcel B, located at the northeast portion of the site, is designated for another four-story hotel. Parcel C, located in the west central portion of the site, is designated for multi-family housing with approximately 120 to 165 units, and Parcel D, located at the northwest corner of the site, is designated for single-story retail. A stormwater detention/infiltration pond is proposed on the south east side of the site. Associated parking is included for each parcel. Off-site improvements may also be required; however, the extent of these improvements have not been determined at this time. The site layout is shown in Figure 2, Site Plan.

This report has been updated to reflect changes in site use and proposed site layout since our original report provided in August 2019. Our design recommendations are specific to the areas of the site that have been adequately explored. Once the site layout has been finalized GeoEngineers should evaluate if additional explorations are recommended.

2.0 SCOPE OF SERVICES

Our specific scope of services is detailed in our proposal to you dated December 11, 2018, (authorized January 28, 2019) and our Contract Amendments, but in general included: reviewing selected geotechnical information about the site; exploring subsurface soil and groundwater conditions; collecting representative soil samples; performing infiltration testing; completing relevant laboratory testing and geotechnical analyses; preparing a draft geotechnical report with our conclusions, findings and design recommendations and a final geotechnical report that addressed comments on our draft report from the design team. This updated report addresses changes in the proposed site use and incorporates updated seismic design parameters, consistent with the current version of the building code.

3.0 SITE CONDITIONS

3.1. Area Geology

The site is in the eastern portion of the Tualatin Valley, a structural basin bounded by Miocene-age basalt but dominated by deep, relatively young, alluvial basin fill that underlies the site and vicinity.

The mapping of Madin (1990) shows the area underlain by between 450 to 600 feet of alluvial sand and silt of the fine-grained facies of the Pleistocene-age catastrophic flood deposits. All the fine sand, silt and clay alluvial materials encountered in the upper portion of the Tualatin Valley fill are included by Madin (1990) in this unit, deposited during the slack water phase of the Missoula glacial outburst floods.

The work of Wilson (1998); however, divides the fine-grained flood deposit materials in the Tualatin Basin into two informal subunits. The upper material is identified by Wilson (1998) as “Willamette Silt” that is described as “…massive, micaceous, clayey, very fine sandy silt with scattered layers of silty clay…” Wilson (1998) correlates this material with the fine-grained flood deposits of Madin (1990) and is reported

by Wilson (1998) as restricted to only the upper 24 to 25 meters (roughly 70 to 80 feet) of the Tualatin Valley fill.

Wilson (1998) considers the fine-grained sediment below the Willamette Silt to be an older, Miocene- to Plio-Pliocene-age alluvial deposit termed "Hillsboro Formation" and described as "…generally poorly sorted and range from silty sands to muds. Silt and clay are present in all samples, while very fine sand may be present in silt and clay units…" The analytical results presented in Wilson (1998) suggest that the Hillsboro Formation is largely locally-derived alluvium eroded from the surrounding highlands. This older deposit extends from the base of the Willamette Silt to the top of the Columbia River Basalt, hundreds of feet below the surface.

Based on the subsurface conditions encountered during our field investigation, we believe that the geology of the parcel generally conforms to the published geology with the minor exceptions noted below.

3.2. Surface Conditions

The project site encompasses approximately 13 acres and is bounded by existing commercial development to the west, a Beaverton School District bus facility to the east, SW Allen Boulevard to the north, Fanno Creek to the south and Oregon Highway 217 to the southwest. The property is roughly square-shaped and is relatively level to gently sloping to the north with an approximate 5-foot change in elevation across the site.

Based on our review of a series of aerial photographs, most of the site was previously developed between 1963 and 1977. A motel and office building are first visible in a 1977 aerial photograph, having been absent in the 1963 photo. The hotel and office building were demolished between 2006 and 2007. The site is currently undeveloped and generally consists of rough field grass/weeds, gravel and asphalt concrete (AC) surfacing from previous development at the site. The northeast corner of the site near the proposed hotel footprint consists of soil/gravel fill stockpiles. Associated underground utilities are located throughout the property.

3.3. Subsurface Conditions

We explored subsurface soil and groundwater conditions at the site between February 11 to February 15, 2019, by drilling 16 borings (B-1 through B-16) to depths between 6½ to 51½ feet below ground surface (bgs) and completing 4 infiltration test borings (IT-1 through IT-4) to depths of 5 feet bgs at the approximate locations shown in Figure 2, Site Plan. GeoEngineers also completed a total of 97 borings across the site area in June 2004 and January 2006 during our initial investigation for a previous planned development. The location of previous explorations is shown in Figure 3, Previous Exploration Locations.

Representative soil samples from the borings were returned to our laboratory for examination and testing. Detailed descriptions of our site exploration and laboratory-testing programs, along with exploration logs and laboratory test results, are presented in Appendix A, Field Explorations and Laboratory Testing.

In general, subsurface conditions consist of the medium stiff to very stiff (occasionally soft) silty clay to clayey silt with a veneer of man-made fill materials across portions of the site. The following paragraphs describe these materials in more detail.

3.3.1. Soil Conditions

3.3.1.1. FILL

Where encountered, the thickness of fill soil ranged from approximately 12 inches up to approximately 4½ feet. In the pavement areas (B-1, B-3, B-4, B-7, B-8, B-9 and B-13), we encountered AC approximately 3 inches in thickness (except 1½ inches at B-13). The AC overlies a variable thickness of crushed rock base and/or silty sand and gravel ranging from 12 to 24 inches. Outside of the AC surfaced areas, in areas that were previously developed, the fill consisted of a silty gravel with a thickness ranging from approximately 2 to 4½ feet. The fill was in a generally medium dense condition. The borings performed in our geotechnical report for the site in 2006 encountered fill soils up to 10 feet bgs in the south and west areas of the site. The fill in some of the 2006 borings consisted of re-worked native silt that contained organics, plastic, asphalt and concrete debris not encountered in our explorations performed in February 2019.

3.3.1.2. SILTY CLAY TO CLAYEY SILT (WILLAMETTE SILT)

Native silty clay to clayey silt with fine sand and occasional interbedded silty sand layers, locally known as Willamette Silt, was encountered below the fill or at the ground surface where fill was not encountered, to a depth of approximately 43 feet bgs. Where it was encountered at the surface, the top 3 inches generally consisted of organics. The consistency of the Willamette Silt ranges from medium stiff to very stiff, with occasional soft areas between 5 and 15 feet bgs and 30 and 35 feet bgs.

3.3.1.3. SILTY CLAY TO CLAYEY SILT (HILLSBORO FORMATION)

Native clayey silt to silty clay, locally known as the Hillsboro Formation, was encountered below approximately 43 feet bgs. This unit is similar to the Willamette Silt except soil consistency is less variable and stiffer.

3.3.2. Groundwater

We encountered groundwater at approximately 1½ to 3 feet bgs in drilled borings IT-1, IT-2, B-12/IT-3 and B-13/IT-4 that were drilled with hollow-stem auger. These borings were left open overnight and the groundwater level was measured the following day. The groundwater measurements were generally consistent with those measured in our borings performed in January and February 2006. The groundwater was observed to be confined and typically rose 2 to 6 feet higher than initially encountered during drilling. Groundwater was not measured in the other explorations due to the mud-rotary drilling method used.

Following our explorations in February 2019, the project team dug six additional test pits (TP-1 to TP-6) to further quantify the groundwater elevation. The test pits were dug on March 22nd and allowed to equilibrate overnight. Drainage tiles were encountered in TP-4 and TP-5 and a polyvinyl chloride (PVC) storm pipe was encountered in TP-6, each of which was draining into the excavations resulting in invalid determination of static groundwater at these locations. Test pits TP-1 to TP-3 appeared to be in native soils and groundwater was measured at approximately 5 to 6½ feet bgs. A summary of the groundwater depths encountered during our 2006 and 2019 borings and the 2019 test pits are presented in Table 1 below. Measured groundwater in borings that were not allowed to equilibrate overnight are not included.

Groundwater levels measured during our 2019 borings were completed during a period of extended heavy rain and may have been influenced by surface water infiltrating the hole or groundwater mounding. The readings completed in the test pits were not influenced by surface water. Further, the groundwater elevations in the test pits were surveyed in by a professional surveyor while the groundwater elevation from

the borings was completed with an electric tape and estimated elevations from the topographic survey provided.

Groundwater conditions across the site are expected to fluctuate due to the time of year, rainfall, water level fluctuations in nearby Fanno Creek, changes in surface topography or other factors not observed during our subsurface exploration program.

TABLE 1. GROUNDWATER SUMMARY

Exploration No.	Date of Exploration	Approximate Surface Elevation[1]	Depth to Groundwater (feet bgs)	Groundwater Elevation[2,3]
IT-1	February 2019	184	2.9	181.0
IT-2	February 2019	187	2.5	184.5
B-12/IT-3	February 2019	185	2.5	182.5
B-13/IT-4	February 2019	187	1.5	185.5
B-32	January 2006	186	2.0	184.0
B-50	January 2006	183	2.5	180.5
B-53	January 2006	183	2.5	180.5
B-56	January 2006	184	2.5	181.5
B-58	January 2006	187	0.5	186.5
B-60	January 2006	188	6.5	181.5
B-61	January 2006	188	0.5	187.5
B-64	January 2006	188	0.5	187.5
B-76	February 2006	186	5.5	180.5
B-82	January 2006	189	1.5	187.5
B-84	January 2006	189	1.5	187.5
B-87	January 2006	189	1.5	187.5
B-88	January 2006	189	2.0	187.0
TP-1	March 2019	186	5.65	180.35
TP-2	March 2019	184	6.85	177.15
TP-3	March 2019	188	5.15	182.85

Notes:

[1] Based on Washington County National Geodetic Vertical Datum of 1929 (NGVD 29). Elevations were not included on the logs for the 2006 Draft Geotechnical Report. Existing site elevations are assumed to be similar to those during the 2006 explorations.

[2] Groundwater elevation measured following equilibration using an electric tape.

[3] Groundwater elevation in the test pits were measured using survey equipment by the survey team.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1. General

A summary of the primary geotechnical considerations is provided below. The summary is presented for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report.

- On-site near surface soils generally consist of silt and silty gravel fill and native silt.
  - The silt soils will become significantly disturbed from earthwork occurring during periods of wet weather, or when the moisture content of the soil is more than a few percentage points above optimum. Wet weather construction practices will be required unless earthwork occurs during the dry summer months (typically mid-July to mid-September).
  - Silty gravel fill located under pavements may be reused as structural fill provided it can be separated from the underlying silt. If the gravel is mixed with the silt it will be moisture sensitive and require special handling.
- On-site silt soils may be reused as structural fill; however, the material is very moisture sensitive and will likely not be suitable for reuse except during the driest of summer months.
- Groundwater was measured between approximately Elevation 180.5 to 187.5 feet Mean Sea Level (MSL) in our February 2019 explorations. Groundwater was encountered between approximately Elevation 177 to 183 feet MSL in the March 2019 test pit explorations. We consider the groundwater elevations measured in the test pits to likely be a more accurate representation of the actual groundwater elevation, as discussed above.
- The feasibility of infiltration was assessed at the site through review of near surface soil conditions and groundwater levels. Due to a relatively shallow groundwater table, and the presence of low permeability silt soils near the ground surface, we conclude that the use of large-scale infiltration facilities is not feasible at this site.
- Structures with column loads less than 70 kips and wall loads less than 5 kips per lineal foot (klf) can be supported on shallow foundations bearing on medium stiff or stiffer silt. Soft/loose or unsuitable soil encountered beneath the foundations should be removed to medium stiff or stiffer material and replaced with compacted structural fill.
- Structures with column loads greater than 70 kips and up to 200 kips and wall loads greater than 5 klf and less than 13 klf can be supported on continuous and isolated spread foundations supported on a minimum 2-foot-thick compacted gravel pad over medium stiff or firmer native silt.
- Structures with column loads greater than 200 kips and up to 260 kips can be supported on continuous and isolated spread foundations supported on a minimum 3-foot-thick compacted gravel pad over medium stiff or firmer native silt.
- We estimate total foundation settlements will be less than approximately 1 inch, provided they are supported as recommended. If larger structural loads are anticipated, we must review and reassess the estimated settlement.
- Floor slabs having 125 pounds per square foot (psf) loads or less can be supported on aggregate base placed on native medium stiff or stiffer silt.

- Standard pavement sections prepared as described in this report will suitably support estimated traffic loads.

5.0 INFILTRATION/DETENTION CONSIDERATIONS

We attempted to conduct a total of four infiltration tests at the requested exploration locations (IT-1, IT-2, B-12/IT-3 and B-13/IT-4) as shown in Figure 2. During testing, groundwater was encountered between 1½ and 3 feet bgs making infiltration testing infeasible.

The on-site soils correspond to a Soil Group C/D as defined by the National Soil Conservation Service (NRCS). The first letter is for drained site conditions and the second letter is for undrained site conditions. Group C soils have a slow infiltration rate when thoroughly wet with saturated hydraulic conductivities typically ranging between 0.14 and 1.42 inches per hour (in/hr) (NRCS 2004). Group D soils are described as soils that have a very slow infiltration rate and high runoff potential when thoroughly wetted with saturated hydraulic conductivities of less than 0.14 in/hr (NRCS 2004). For this site, we anticipate the soil to behave as a soil Group C during the dry summer/early fall months and as a Soil Group D during the wet winter and spring months of the year. Correction factors have not been applied to the ranges of hydraulic conductivity values provided above. We can discuss the option of infiltration testing during the dry summer months if the project team anticipates that it is possible to implement the proposed stormwater facilities based on the information provided.

At the time this report was prepared, the proposed pond area on the east side of the site was proposed as a detention pond. We understand the pond will likely be unlined. We recommend that permanent cut and fill slopes be designed as recommended in this report.

6.0 EARTHWORK RECOMMENDATIONS

6.1. Site Preparation

Initial site preparation and earthwork operations will include demolition and removal of existing site improvements; stripping and grubbing; and removal/reuse of fill stockpiles, as described below.

6.1.1. Demolition

Development will include removal of pavements and hardscapes as well as drainage tiles and abandoned utilities. Demolition of existing structural improvements should include excavation and removal of all foundation elements, such as footings, stem walls and slabs. All existing utilities in the construction area should be identified prior to excavation. Live utility lines identified beneath proposed structures should be relocated. Abandoned utility lines beneath proposed structures should be completely removed or filled with grout to reduce potential settlement of new structures. Soft or loose soil encountered in utility line excavations should be removed and replaced with structural fill where it is located within structural areas. Materials generated during demolition of existing improvements should be transported off site for disposal, or processed for reuse, as described below.

6.1.2. Stripping and Clearing

Based on our observations, we estimate that the depth of stripping of organics in grass areas will be on the order of about 3 to 4 inches. Greater stripping depths may be required to remove localized zones of loose or organic soil, and in areas where moderate to heavy vegetation may be present, or surface disturbance has occurred. In addition, if present in areas of proposed development, the primary root systems of trees should be completely removed. Stripped material should be transported off site for disposal or processed and used as fill in landscaping areas.

Existing trees, brush and near-surface root zone should be grubbed to the depth of the roots, which could exceed 3 feet bgs. Depending on the methods used to remove the preceding material, considerable disturbance and loosening of the subgrade could occur.

Existing voids and new depressions created during site preparation and resulting from removal of existing utilities, foundations, root balls, or other subsurface elements, should be cleared of loose soil or debris down to firm soil and backfilled. All excavations should be backfilled with structural fill, as defined in a following section of this report.

6.1.3. Fill Stockpile Removal

We recommend that the soil fill stockpiles located in the northeastern corner of the site be evaluated for suitability for reuse as structural fill and either be reused or removed, as appropriate. Explorations were not performed within these soil stockpiles; therefore, the soil conditions are unknown. The material in the stockpile may be suitable as structural fill on site provided it meets the requirements of the "Structural Fill and Backfill" Section 6.8 of this report. We can further evaluate the suitability of these stockpiles with additional explorations or during site construction.

6.2. Subgrade Preparation and Evaluation

Upon completion of site preparation activities, the exposed subgrade should be proof-rolled with a fully loaded dump truck or similar heavy rubber-tired construction equipment to identify soft, loose or unsuitable areas. Proof-rolling should be conducted prior to placing fill, and should be observed by a representative of GeoEngineers who will evaluate the suitability of the subgrade and identify areas of yielding that are indicative of soft or loose soil. If soft or loose zones are identified during proof-rolling, these areas should be excavated to the extent indicated by our representative and replaced with "Imported Select Structural Fill" as defined in Section 6.8.3 of this report.

During wet weather, or when the exposed subgrade is wet or unsuitable for proof-rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations, probing and compaction testing should be performed by a member of our staff. Wet soil that has been disturbed due to site preparation activities or soft or loose zones identified during probing, should be removed and replaced with "Imported Select Structural Fill" as defined in Section 6.8.3 of this report.

Soft fill or fill with significant debris or unsuitable material should be removed to native medium stiff or firmer material and replaced with "Imported Select Structural Fill" as defined in Section 6.8.3 of this report.

6.3. Wet Weather Construction

The fine-grained soils at the site are highly susceptible to moisture. Wet weather construction practices will be necessary if work is performed during periods of wet weather. If site grading will occur during wet weather conditions, it will be necessary to use track-mounted equipment, load removed material into trucks supported on existing pavement, use gravel working pads and employ other methods to reduce ground disturbance. The contractor should be responsible to protect the subgrade during construction.

During wet weather we recommend that:

- The ground surface in and around the work area be sloped so that surface water is directed to a sump or discharge location. The ground surface should be graded such that areas of ponded water do not develop.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- The site soils should not be left uncompacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.

In general, if construction activities are planned during periods of wet weather, the contractor should consider using existing pavements and/or granular haul roads and staging area to reduce subgrade disturbance. Based on our experience, between 12 and 18 inches of imported granular material is generally required to provide stable staging areas and haul roads. However, the actual thickness will depend on the contractor's means and methods and accordingly, should be the contractor's responsibility. Additionally, a geotextile fabric, such as Propex Geotex 104F, or approved alternate, should be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic.

6.4. Excavation

Based on the material encountered in our subsurface explorations, it is our opinion that conventional earthmoving equipment in proper working condition should be capable of making necessary general excavations.

The earthwork contractor should be responsible for reviewing this report, including the boring logs, providing their own assessments, and providing equipment and methods needed to excavate the site soils while protecting subgrades.

6.5. Dewatering

As discussed in "Groundwater" Section 3.3.2 of this report, shallow groundwater was encountered in our explorations. If groundwater is encountered, saturated/wet soils should be dewatered. Sump pumps are expected to adequately address groundwater encountered in shallow excavations. However, excavations deeper than about 2 to 3 feet will require larger pumps and/or systems to seal off the excavation walls from the groundwater. In addition to groundwater seepage and upward groundwater flow into excavations, surface water inflow to the excavations during the wet season can be problematic. Provisions for surface

water control during earthwork and excavations should be included in the project plans and should be installed prior to commencing earthwork.

The level of effort required for dewatering will depend to a large extent on the time of year during which construction is accomplished. We recommend that construction be completed in the late summer or early autumn months when the groundwater level is typically at its lowest elevation.

6.6. Shoring

All trench excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. In our opinion, fill soils are generally OSHA Type C while native silt soils are generally OSHA Type B. Excavations deeper than 4 feet should be shored or laid back at an inclination of 1.5H:1V (horizontal to vertical) for Type C soils and 1H:1V for Type B soils, or flatter if workers are required to enter. Excavations made to construct footings or other structural elements should be laid back or shored at the surface as necessary to prevent soil from falling into excavations.

Shoring for trenches less than 6 feet deep that are above the effects of groundwater should be possible with a conventional box system. Moderate sloughing should be expected outside the box. Shoring deeper than 6 feet or below the groundwater table should be designed by a registered engineer before installation. Further, the shoring design engineer should be provided with a copy of this report.

It should be expected that unsupported cut slopes will experience some sloughing and raveling if exposed to water. Plastic sheeting, placed over the exposed slope and directing water away from the slope, will reduce the potential for sloughing and erosion of cut slopes during wet weather.

In our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to the soil and groundwater conditions. Construction site safety is generally the sole responsibility of the contractor, who also is solely responsible for the means, methods, and sequencing of the construction operations and choices regarding excavations and shoring. Under no circumstances should the information provided by GeoEngineers be interpreted to mean that GeoEngineers is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

6.7. Cut and Fill Slopes

Permanent cut and fill slopes should be inclined no steeper than 2H:1V. Slopes that are susceptible to wetting and drying cycles will experience some sloughing and raveling. Structures and roads should have a minimum set back of 5 feet from the slope crest. Fill slopes should be overbuilt by at least 2 feet and trimmed back to the required slope to maintain a firm face.

Constructed slopes should be planted with appropriate vegetation as soon as possible after grading to provide protection against erosion. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

If seepage is encountered at the face of permanent or temporary slopes, it will be necessary to flatten the slopes or install a subdrain to collect the water. We should be contacted to evaluate such conditions on a case-by-case basis.

Fill placed on slopes steeper than 50 percent (2H:1V) should be placed on a keyed and benched surface. Typically, a minimum 4-foot-wide by 2-foot-deep keyway is excavated into competent (medium stiff/medium dense or better) soils at the base of the fill. The slope of the downslope edge of this excavation should not be greater than 1H:1V. After excavation of the keyway, the slope to receive fill should be benched with the benches being excavated into medium stiff/medium dense or better soils. The keyway and benching should be observed by the geotechnical engineer or their representative during construction to verify that the keyway and benches were excavated into competent soils.

6.8. Structural Fill and Backfill

6.8.1. General

Materials used to support building foundations, floor slabs, hardscape, pavements and any other areas intended to support structures or within the influence zone of structures are classified as structural fill for the purposes of this report.

All structural fill soils should be free of debris, clay balls, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 4 inches and other deleterious materials. The suitability of soil for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines in the soil matrix increases, the soil becomes increasingly more sensitive to small changes in moisture content and achieving the required degree of compaction becomes more difficult or impossible. Recommendations for suitable fill material are provided in the following sections.

6.8.2. Use of On-site Soil

As described in "Subsurface Conditions" Section 3.3, the on-site surface soil consists of gravel fill and native silty soils. On-site soils can be used as structural fill, provided the material meets the above requirements; although due to moisture sensitivity, the native silt material will likely be unsuitable as structural fill during most of the year and should be considered for reuse only during prolonged dry weather. If the soil is too wet to achieve satisfactory compaction, moisture conditioning the material will be required. If the material cannot be properly moisture conditioned, we recommend using imported material for structural fill.

Asphalt concrete, cement concrete and over-sized materials may be used as structural fill provided the recommendations below are followed.

Processed Recycled Materials. AC, cement concrete or over-sized rock can be used as structural fill provided they are processed by crushing and screening, grinding in place or other methods to meet the structural fill recommendations in this report. They may be used as structural fill in all areas except within 3 feet beneath foundations. Recycled asphalt may not be used at any depth within the building footprint.

Unprocessed Recycled Materials. AC, cement concrete or over-sized rock fragments that have a maximum particle size of 4 inches in nominal diameter, may be mixed with on-site soil or imported fill to create a generally uniform, well-graded material and used in pavement and landscape areas. If used beneath pavements we recommend that at least 2 feet of other structural fill overlie the unprocessed recycled fill material.

An experienced geotechnical engineer from GeoEngineers should determine the suitability of on-site soil encountered during earthwork activities for reuse as structural fill.

6.8.3. Imported Select Structural Fill

Imported select granular material may be used as structural fill. Imported select structural fill should consist of pit or quarry run rock, crushed rock, or crushed gravel and sand that is fairly well-graded between coarse and fine sizes, with approximately 25 to 65 percent passing the U.S. No. 4 sieve. It should have less than 5 percent passing the U.S. No. 200 sieve and have a minimum of two mechanically fractured faces. During dry weather, the fines content can be increased to a maximum of 12 percent.

6.8.4. Aggregate Base

Aggregate base material located under floor slabs and pavements, and crushed rock used in footing overexcavations should consist of imported clean, durable, crushed angular rock. Such rock should be well-graded, have a maximum particle size of 1 inch and have less than 5 percent passing the U.S. No. 200 sieve (3 percent for retaining walls), and meet the gradation requirements in Table 2. In addition, aggregate base shall have a minimum of 75 percent fractured particles according to American Association of State Highway and Transportation Officials (AASHTO) TP-61 and a sand equivalent of not less than 30 percent based on AASHTO T-176.

TABLE 2. RECOMMENDED GRADATION FOR AGGREGATE BASE

Sieve Size	Percent Passing (by weight)
1 inch	100
½ inch	50 to 65
No. 4	40 to 60
No. 40	5 to 15
No. 200	0 to 5

6.8.5. Trench Backfill

Backfill for pipe bedding and in the pipe zone should consist of well-graded granular material with a maximum particle size of ¾ inch and less than 5 percent passing the U.S. No. 200 sieve. The material should be free of organic matter and other deleterious materials. Further, the backfill should meet the pipe manufacturer's recommendations. Above the pipe zone, "Imported Select Structural Fill" may be used as described above in Section 6.8.3.

6.8.6. Cement Treated Subgrade Design

As an alternative to the use of imported granular material for wet weather structural fill, an experienced contractor may be able to amend the on-site soil with portland cement concrete (PCC), or with limekiln dust and PCC, to obtain suitable support properties. Successful use of soil amendment depends on the use of correct mixing techniques, soil moisture content and amendment quantities. Specific recommendations, based on exposed site conditions, for soil amending can be provided if necessary. However, for preliminary planning purposes, it may be assumed that a minimum of 5 percent cement (by dry weight, assuming a unit weight of 100 pounds per cubic foot [pcf]) will be sufficient for subgrade and general fill amendment. Treatment depths of 12 to 16 inches for roadway subgrades are typical (assuming a 7-day unconfined compressive strength of at least 80 pounds per square inch [psi]), though they may be adjusted in the field depending on site conditions. Soil amending should be conducted in accordance with the specifications

provided in the current Oregon Department of Transportation (ODOT) *Standard Specifications for Construction* Section 00344 (Treated Subgrade).

6.8.7. Fill Placement and Compaction

Structural fill should be compacted at moisture contents that are within 3 percent of the optimum moisture content as determined by ASTM International (ASTM) Standard Practices Test Method D 1557 (Modified Proctor). The optimum moisture content varies with gradation and should be evaluated during construction. Fill material that is not near the optimum moisture content should be moisture conditioned prior to compaction.

Fill and backfill material should be placed in uniform, horizontal lifts and compacted with appropriate equipment. The appropriate lift thickness will vary depending on the material and compaction equipment used. Fill material should be compacted in accordance with Table 3, below. It is the contractor's responsibility to select appropriate compaction equipment and place the material in lifts that are thin enough to meet these criteria. However, in no case should the loose lift thickness exceed 18 inches.

TABLE 3. COMPACTION CRITERIA

Fill Type	Compaction Requirements		
	Percent Maximum Dry Density Determined by ASTM Test Method D 1557 at ± 3 Percent of Optimum Moisture		
	0 to 2 Feet Below Subgrade	> 2 Feet Below Subgrade	Pipe Zone
Fine-grained soils (non-expansive)	95	92	-----
Imported Granular, maximum particle size < 1¼ inch	95	95	-----
Imported Granular, maximum particle size 1¼ inch to 4 inches	n/a (proof-roll)	n/a (proof-roll)	-----
Retaining Wall Backfill*	92	92	------
Nonstructural Zones	90	90	90
Trench Backfill	95	90	90

Note:

*Measures should be taken to prevent overcompaction of the backfill behind retaining walls. We recommend placing the zone of backfill located within 5 feet of the wall in lifts not exceeding about 6 inches in loose thickness and compacting this zone with hand-operated equipment such as a vibrating plate compactor and a jumping jack.

A representative from GeoEngineers should evaluate compaction of each lift of fill. Compaction should be evaluated by compaction testing, unless other methods are proposed for oversized materials and are approved by GeoEngineers prior to fill placement. These other methods typically involve procedural placement and compaction specifications together with verifying requirements such as proof-rolling.

7.0 STRUCTURAL DESIGN RECOMMENDATIONS

7.1. Foundation Support Recommendations

7.1.1. General

Depending on the building loads, the structures can be supported on spread footings bearing on medium stiff or stiffer silt or on thickened granular fill pads over native medium stiff or stiffer silt soils.

We recommend the shallow foundations be founded at least 18 inches below the lowest adjacent grade, or as needed to meet the design loads. The recommended minimum foundation depth is greater than the anticipated frost depth.

7.1.2. Foundation Subgrade Preparation

We recommend that prepared subgrades be observed by a member of our firm, who will evaluate the suitability of the subgrade and identify areas of yielding, which are indicative of soft or loose soil.

Individual spread and continuous wall footings should be supported on native medium stiff or stiffer silt or a thickened granular fill pad bearing on medium stiff or stiffer silt, as follows:

- Structures with column loads less than 70 kips and wall loads less than 5 klf can be supported on shallow foundations bearing on medium stiff or stiffer silt.
- Structures with column loads greater than 70 kips and up to 200 kips and wall loads greater than 5 klf and less than 13 klf can be supported on continuous and isolated spread foundations supported on a minimum 2-foot-thick compacted gravel pad over medium stiff or firmer native silt.
- Structures with column loads greater than 200 kips and up to 260 kips can be supported on continuous and isolated spread foundations supported on a minimum 3-foot-thick compacted gravel pad over medium stiff or firmer native silt.

Granular pads should consist of ¾-inch-minus select granular fill placed and compacted as structural fill. Granular pads should extend outward from the edge of the footing 1 foot for every 2 feet of depth.

Any fill material encountered beneath proposed foundation elements should be removed to competent native soils and replaced with structural fill. The width of the overexcavation should extend beyond the edge of the footing a distance equal to the depth of the overexcavation below the base of the footing. The exposed subgrade soil should be probed with a ½-inch-diameter steel rod. If soft, yielding or otherwise unsuitable areas are revealed during probing the unsuitable soils should be removed and replaced with structural fill, as needed.

We recommend loose or disturbed soils be removed before placing reinforcing steel and concrete. Foundation bearing surfaces should not be exposed to standing water. If water infiltrates and pools in the excavation, the water, along with any disturbed soil, should be removed before placing reinforcing steel. A minimum 6-inch-thick layer of crushed rock Aggregate Base material should be placed over the prepared subgrade. Aggregate Base material placed directly below the mat foundation should be ¾-inch-maximum particle size or less. Compaction should be performed as described in the “Fill Placement and Compaction” Section 6.8.7.

We recommend GeoEngineers observe all foundation excavations before placing concrete forms and reinforcing steel to determine that bearing surfaces have been adequately prepared and the soil conditions are consistent with those observed during site explorations.

7.1.3. Bearing Capacity

We recommend shallow footings be proportioned using a maximum allowable bearing pressure of 2,000 psf. This bearing pressure applies to the total of dead and long-term live loads and may be increased by $^1/_3$ when considering earthquake or wind loads. This is a net bearing pressure. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

7.1.4. Foundation Settlement

Foundations designed and constructed as recommended are expected to experience settlements of less than 1 inch. Differential settlements of up to ½ of the total settlement magnitude can be expected between adjacent footings supporting comparable loads.

7.1.5. Lateral Resistance

Lateral loads on footings can be resisted by passive earth pressures on the sides of footings and by friction on the bearing surface. We recommend that passive earth pressures be calculated using an equivalent fluid unit weight of 250 pcf for foundations confined by native medium stiff or stiffer silt and 350 pcf if confined by a minimum of 2 feet of imported granular fill.

We recommend using a friction coefficient of 0.35 for foundations placed on the native medium stiff or stiffer silt, or 0.50 for foundations placed on a minimum 2-foot thickness of compacted crushed rock. The passive earth pressure and friction components may be combined provided 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 static groundwater remains below the base of the footing throughout the year. The top 1 foot of soil should be neglected when calculating passive lateral earth pressures, unless the adjacent area is covered with pavement. The lateral resistance values include a safety factor of approximately 1.5.

7.2. Drainage Considerations

We recommend the ground surface be sloped away from the buildings at least 2 percent. All downspouts should be tightlined away from the building foundation areas and should also be discharged into a stormwater disposal system. Downspouts should not be connected to footing drains.

We recommend that perimeter footing drains be installed around the proposed buildings at the base of the exterior footings. The perimeter footing drains should be provided with cleanouts and should consist of at least 4-inch-diameter perforated pipe placed on a 3-inch bed of, and surrounded by, 6 inches of drainage material enclosed in a non-woven geotextile such as Mirafi 140N (or approved alternate) to prevent fine soil from migrating into the drain material. We recommend against using flexible tubing for footing drainpipes. The perimeter drains should be sloped to drain by gravity to a suitable discharge point, preferably a storm drain. We recommend that the cleanouts be covered and placed in flush-mounted utility boxes. Water collected in roof downspout lines must not be routed to the footing drain lines.

7.3. Floor Slabs

Satisfactory subgrade support for floor slabs up to 125 psf can be obtained from the medium stiff or firmer native silt or on new structural fill placed on these materials, provided the floor slab subgrade is prepared as recommended in this report. Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Load-bearing concrete slabs should be designed assuming a modulus of subgrade reaction (k) of 125 pounds per cubic inch (pci) provided the slab subgrade is prepared as recommended in this report. Concrete slabs constructed as recommended will likely settle less than 1 inch. We recommend that concrete slabs be jointed around columns to allow the individual structural elements to settle differentially.

We recommend that on-grade slabs be underlain by a minimum 6-inch-thick capillary break layer to reduce the potential for moisture migration into the slab and to provide consistent subgrade reaction support to the slab. The capillary break material should consist of aggregate base material as described in "Structural Fill and Backfill" Section 6.8 of this report. The material should be placed as recommended in "Fill Placement and Compaction" Section 6.8.7. If dry slabs are required (e.g., where adhesives are used to anchor carpet or tile to the slab), a vapor retarder should be placed below the slab. The vapor retarder should be selected by the structural engineer and should be accounted for in the design floor section and mix design selection for the concrete, to accommodate its effect on concrete slab curing.

7.4. Retaining Walls

7.4.1. Permanent Subsurface Walls

Retaining walls should be designed for the earth pressures described below. Other surcharge loads, such as from foundations, construction equipment or construction staging areas, should be considered on a case-by-case basis as appropriate. We can provide the lateral pressures from these surcharge loads as the design progresses.

For walls that are free to yield at the top at least 0.1 percent of the height of the wall, soil pressures will be less than if movement is limited by such factors as wall stiffness or bracing. Assuming the walls are backfilled and drainage is provided as outlined in the following paragraphs, we recommend that yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 40 pcf (triangular distribution), while non-yielding walls supporting horizontal backfill be designed using an equivalent fluid density of 55 pcf (triangular distribution). For seismic loading conditions, a rectangular earth pressure equal to 9H psf (where H is the height of the wall in feet) should be added to the active/at-rest pressures. Other surcharge loading should be applied as appropriate.

We recommend all retaining wall footings bear on medium stiff native soil or a minimum of 2 feet of structural fill over approved subgrade. An allowable bearing pressure of 2,500 psf can be used for design. Lateral resistance for conventional cast-in-place walls can be provided by frictional resistance along the base of the wall and passive resistance in front of the wall. For walls founded on medium stiff native soils or on a minimum 2 feet of structural fill, the allowable frictional resistance may be computed using a coefficient of friction of 0.3 and 0.5 applied to vertical dead-load forces, respectively. The allowable passive resistance may be computed using an equivalent fluid density of 300 pcf (triangular distribution) if confined by at least 2 feet of structural fill soils. The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

The above soil pressures assume that wall drains will be installed to prevent the buildup of hydrostatic pressure behind the walls, as discussed below.

7.4.2. Drainage

Drainage behind the permanent below-grade walls is recommended. Positive drainage should be provided behind cast-in-place retaining walls above the design groundwater elevation by placing a minimum 2-foot-wide zone of retaining wall backfill as as specified in "Structural Fill and Backfill" Section 6.8 of this report. A perforated drainpipe should be placed near the base of the retaining wall to provide drainage. The drainpipe should be surrounded by a minimum of 6 inches of ⅝-inch crushed gravel, or 1-inch washed gravel, ODOT Standard Specification 02610, or an alternative approved by GeoEngineers. The ⅝-inch crushed gravel or 1-inch washed gravel should be wrapped with a geotextile filter fabric such as Mirafi 140N (or approved alternate). The wall drainpipe should be connected to a header pipe and routed to a sump or gravity drain. Appropriate cleanouts for drainpipe maintenance should be installed. A larger diameter pipe will allow for easier maintenance of drainage systems.

7.4.3. Mechanically Stabilized Earth (MSE) Walls

The reinforced zone behind MSE walls should be backfilled with aggregate base as specified in "Structural Fill and Backfill" Section 6.8 of this report. The retained and foundation soil zones should consist of undisturbed *in situ* native soil or new engineered fill.

Where traffic loads are located within a horizontal distance from the top of the wall equal to ½ the wall height, the lateral earth pressure should be increased by a surcharge load equal to 2 feet of soil (assuming a soil density of 125 pcf). For overturning and sliding analysis, this surcharge should only be applied behind the reinforced soil zone. Global stability should be evaluated by the wall designer.

The long-term design strength of any geotextile required for the wall should be equal or greater than the maximum tensile force. Unless manufacturer specific reduction factors are applied during the design, the ultimate tensile strength of the geosynthetic selected should be reduced by a factor of 7 to account for creep, installation damage, and chemical and biological degredation.

Lateral movement of up to 1 percent of the wall height commonly occur immediately adjacent to the wall, as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that top layer of geogrid be 2 feet longer than the underlying grid or construction of flat work adjacent to retaining walls be postponed at least 4 weeks after backfilling of the wall (unless survey data indicates that lateral movement is complete prior to that time).

7.5. Seismic Design

Parameters provided in Table 4 are based on the conditions encountered during our subsurface exploration program and the procedure outlined in the 2015 International Building Code (IBC). Jurisdictions are beginning to adopt the 2018 IBC, which references the 2016 Minimum Design Loads for Buildings and Other Structures (American Society of Civil Engineers [ASCE] 7-16). Per ASCE 7-16 Section 11.4.8, a ground motion hazard analysis or site-specific response analysis is required to determine the design ground motions for structures on Site Class D sites with S_1 greater than or equal to 0.2g.

For this project, the site is classified as Site Class D with an S_1 value of 0.402g; therefore, the provision of 11.4.8 applies. Alternatively, the parameters listed in Table 5 below may be used to determine the design

ground motions if Exception 2 of Section 11.4.8 of ASCE 7-16 is used. Using this exception, the seismic response coefficient (C_s) is determined by Equation (Eq.) (12.8-2) for values of $T \leq 1.5T_S$, and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$, where T represents the fundamental period of the structure and T_S=0.757 sec. If requested, we can complete a site-specific seismic response analysis, which might provide somewhat reduced seismic demands from the parameters in Table 5 and the requirements for using Exception 2 of Section 11.4.8 in ASCE 7-16. The reduced values will likely not be significant enough to warrant the additional cost of further evaluation if designing to 2018 IBC.

We recommend seismic design be performed using the values noted in Tables 4 or 5 below depending on the version of the IBC used for design.

TABLE 4. MAPPED 2015 IBC SEISMIC DESIGN PARAMETERS

Parameter	Value[1]
Site Class	D
Spectral Response Acceleration, S_s	0.992 g
Spectral Response Acceleration, S_1	0.430 g
Site Coefficient, F_a	1.103
Site Coefficient, F_v	1.570
Spectral Response Acceleration (Short Period), S_{DS}	0.729 g
Spectral Response Acceleration (1-Second Period) S_{D1}	0.450 g
Seismic Design Category	D

Note:

[1] Parameters developed based on Latitude 45.475363° and Longitude -122.786445° using the ATC Hazards online tool.

TABLE 5. MAPPED 2018 IBC SEISMIC DESIGN PARAMETERS

Parameter	Value[1,2]
Site Class	D
Mapped Spectral Response Acceleration at Short Period (S_S)	0.878 g
Mapped Spectral Response Acceleration at 1 Second Period (S_1)	0.402 g
Site Modified Peak Ground Acceleration (PGA_M)	0.479 g
Site Amplification Factor at 0.2 second period (F_a)	1.149
Site Amplification Factor at 1.0 second period (F_v)	1.898
Design Spectral Acceleration at 0.2 second period (S_{DS})	0.672 g
Design Spectral Acceleration at 1.0 second period (S_{D1})	0.509 g

Notes:

[1] Parameters developed based on Latitude 45.475363° and Longitude -122.786445° using the ATC Hazards online tool.

[2] These values are only valid if the structural engineer utilizes Exception 2 of Section 11.4.8 (ASCE 7-16).

7.5.1. Liquefaction Potential

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the

sudden loss of shear strength in a soil. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soil with low silt and clay contents is the most susceptible to liquefaction. Low plasticity, silty sand may be moderately susceptible to liquefaction under relatively higher levels of ground shaking.

The results of our analyses indicate that the liquefaction potential at the project site during the design earthquake event is relatively low due to generally medium stiff to very stiff silt and clay at the project site. However, there are isolated lenses of sandy soils that will experience small magnitudes of liquefaction-induced settlement and soft lenses of low plasticity silty soils are susceptible to cyclic softening. We estimate the settlement induced by cyclic softening and liquefaction to be on the order of 1 inch.

7.5.2. Fault Surface Rupture

The closest mapped fault to the site possibly capable of surface rupture is the Beaverton Fault Zone with its inferred extension approximately ¾ of a mile northwest of the site (Personius 2002). No faults are mapped as crossing the site, and the potential for site fault surface rupture is, therefore, very low.

8.0 PAVEMENT DESIGN RECOMMENDATIONS

8.1. General

Our pavement analyses and section recommendations are based on the results of our field testing and data in general accordance with the *AASHTO Guide for Design of Pavement Structures*. Pavement subgrades should be prepared in accordance with "Earthwork Recommendations" and "Site Preparation" Sections 6.0 and 6.1 of this report. Our interpretations of the subgrade resilient modulus are based on subsurface explorations and California Bearing Ratio (CBR) testing from our 2006 Geotechnical Report. The design of the recommended pavement sections are based on an assumed CBR of 4. The recommended pavement sections assume that the subgrade is constructed during dry weather.

Our pavement recommendations at the site are based on estimated equivalent single axle loads (ESALs) included in the *Drive Shack Geotechnical Investigation Specifications and Report Requirements* "Pavement Design Recommendations" section. We have provided pavement recommendations for a standard duty area (35,000 ESALs) and a heavy-duty area (75,000 ESALs). Light duty pavement areas are considered those accessed mainly by auto traffic (i.e., parking areas). Heavy-duty pavement areas include those within the drive path of heavy trucks and delivery vehicles.

Our pavement recommendations are based on the following assumptions and design parameters included in the *AASHTO Guide for Design of Pavement Structures*:

- The pavement subgrades, fill subgrades and site earthwork used to establish road grades below the Aggregate Subbase and Aggregate Base materials have been prepared as described in "Earthwork Recommendations" Section 6.0 of this report.
- A resilient modulus of 20,000 psi has been estimated for compacted Aggregate Subbase and Aggregate Base materials.

- A resilient modulus of 6,000 psi was estimated for subgrade prepared as recommended in "Earthworks Recommendations" Section 6.0 of this report.
- Initial and terminal serviceability indices of 4.2 and 2.0, respectively.
- Reliability and standard deviations of 85 percent and 0.45, respectively.
- Structural coefficients of 0.42 and 0.10 for the asphalt and base rock, respectively.
- PCC 28-day Compressive Strength: 4,500 psi – Class 4000.
- PCC Minimum 28-day Flexural Strength: 650 psi.
- A 20-year design life.
- Estimated traffic levels based on ESALs of 35,000 and 75,000 for a 20-year design life for standard-duty and heavy-duty areas, respectively. This assumes consistent traffic levels (zero percent growth) over the design life.

If any of the noted assumptions vary from project design use, our office should be contacted with the appropriate information so that the pavement designs can be revised or confirmed adequate. Heavy construction traffic has not been considered in our pavement design; therefore, we assume that the pavements will be constructed at the end of the project after heavy construction vehicles, such as concrete trucks and construction material delivery trucks, will no longer access the site. Construction traffic should not be allowed on new pavements. If this is not the case, we will have to re-design the pavements for those heavier loading conditions.

8.2. Drainage

Long-term performance of pavements is influenced significantly by drainage conditions beneath the pavement section. Positive drainage can be accomplished by crowning the subgrade with a minimum 1.8 percent cross slope and establishing grades to promote drainage.

8.3. Pavement Sections

Based on the estimated traffic data and our analyses, our recommended AC pavement sections are presented in Table 6.

TABLE 6. RECOMMENDED AC PAVEMENT SECTIONS

Section	Minimum Asphalt Thickness (inches)	Minimum Aggregate Base Thickness (inches)
Light-Duty (general automobile parking areas)	3	9
Heavy-Duty (drive aisles and heavy delivery areas)	3.5	9

The aggregate base course should conform to "Aggregate Base" Section 6.8.4 of this report and be compacted to at least 95 percent of the maximum dry density determined in accordance with AASHTO T-180/ASTM Test Method D 1557.

The AC pavement should conform to Section 00745 of the most current edition of the *ODOT Standard Specifications for Highway Construction*. The Job Mix Formula should meet the requirements for a ½-inch

Dense Graded Level 2 Mix. The AC binder should be PG 64-22 grade meeting the *ODOT Standard Specifications for Asphalt Materials*. AC pavement should be compacted to 91.0 percent of the Maximum Theoretical Unit Weight (Rice Gravity) as determined by AASHTO T-209.

The recommended pavement sections assume that final improvements surrounding the pavement will be designed and constructed such that stormwater or excess irrigation water from landscape areas does not infiltrate below the pavement section into the crushed base.

The recommended minimum PCC pavement sections for trash receptacle pads, light-duty and heavy-duty areas are provided in Table 7. The minimum PCC thickness in Table 7 assumes that PCC panel joints will not contain load transfer devices at the joints.

TABLE 7. MINIMUM PCC PAVEMENT DESIGN REQUIREMENTS

Area	Minimum PCC Pavement Thickness (inches)	Minimum Aggregate Base Thickness (inches)
Trash Receptacle Pads	4.0	6.0
Light Duty	4.5	6.0
Heavy Duty	5.0	6.0

For PCC pavements, concrete should be Class 4000 ¾-inch-minus with minimum 28-day flexural strength of 650 psi. A jointing plan should be provided in the project plans showing construction joints and transverse and longitudinal joints to control cracking consistent with American Concrete Pavement Association (ACPA) recommendations. Longitudinal joints typically coincide with PCC panel widths, and transverse joints are generally relatively equally spaced and close to the same spacing as longitudinal joints so that panels are relatively square. If limited panel width results in no longitudinal joints, the jointing plan should maintain a relatively square panel section based on the panel width.

9.0 OTHER CONSIDERATIONS

9.1. Frost Penetration

The near-surface soils are slightly to moderately susceptible to frost heave. However, foundation and floor slab elements are expected to bear on compacted granular fill. We anticipate that the depth of frost penetration in this region is approximately 12 inches. The recommended exterior and interior footing embedment depths provided above should allow adequate frost protection. Frost susceptibility in pavement areas is also expected to be low if they are constructed and supported as recommended.

9.2. Expansive Soils

Based on our laboratory test results and experience with similar soils in the area, we do not consider the soils encountered in our borings to be expansive.

9.3. Corrosivity

We completed resistivity and pH tests on soil samples obtained from the borings B-1 through B-3 and B-8 and B-9 as an indicator of corrosion potential. Based on the test results, we conclude that there is a

moderate to high risk of corrosion to steel and iron pipes at this site. While soluble sulfate testing was not completed, based on the results of resistivity and pH tests, soil type and groundwater conditions, we consider that there is a moderate to high risk of corrosion to concrete. The results of the laboratory tests are provided in Table A-1 in Appendix A.

10.0 DESIGN REVIEW AND CONSTRUCTION SERVICES

Recommendations provided in this report are based on the assumptions and design information stated herein. We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, 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. Our design recommendations are specific to the areas of the site that have been adequately explored. Once the site layout has been finalized GeoEngineers should confirm design recommendations presented in this report and evaluate if additional explorations are recommended. Satisfactory foundation and earthwork performance depends to a large degree on quality of construction. Sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

We recommend that GeoEngineers be retained to observe construction at the site to confirm that subsurface conditions are consistent with the site explorations and to confirm that the intent of project plans and specifications relating to earthwork, pavement and foundation construction are being met.

11.0 LIMITATIONS

We have prepared this report for the exclusive use of Oregon Worsted Company and their authorized agents and/or regulatory agencies for the proposed Allen Boulevard Development Project southeast of Highway 217 and east of SW Allen in Beaverton, Oregon.

This report is not intended for use by others and the information contained herein is not applicable to other sites. No other party may rely on the product of our services unless we agree in advance and in writing to such reliance.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the area at the time this report was prepared. 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.

12.0 REFERENCES

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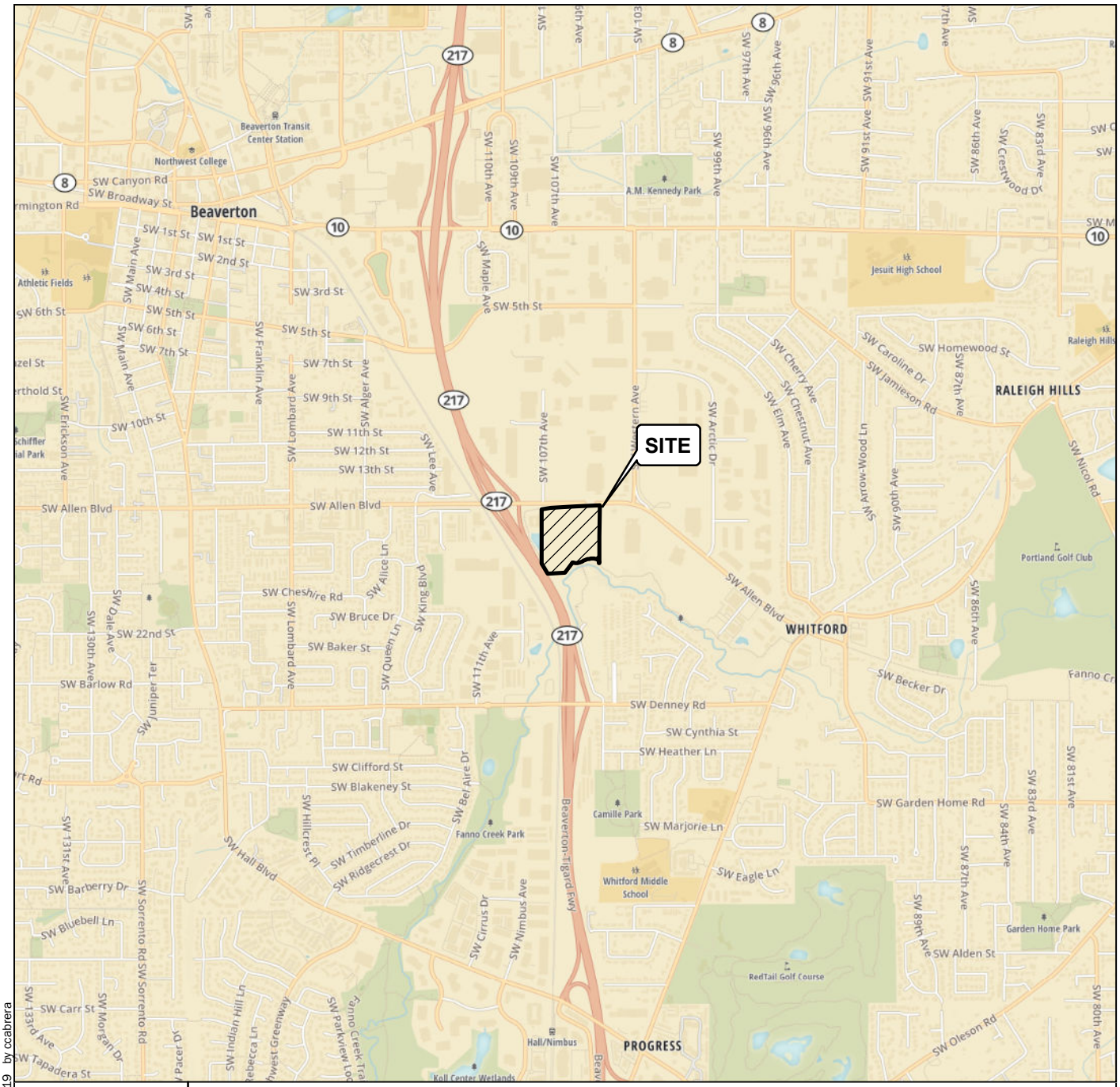
FIGURES

GeoEngineers



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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: Mapbox Open Street Map, 2016

Projection: NAD 1983 UTM Zone 10N

Vicinity Map

Allen Boulevard Development
Beaverton, Oregon

GeoEngineers

Figure 1

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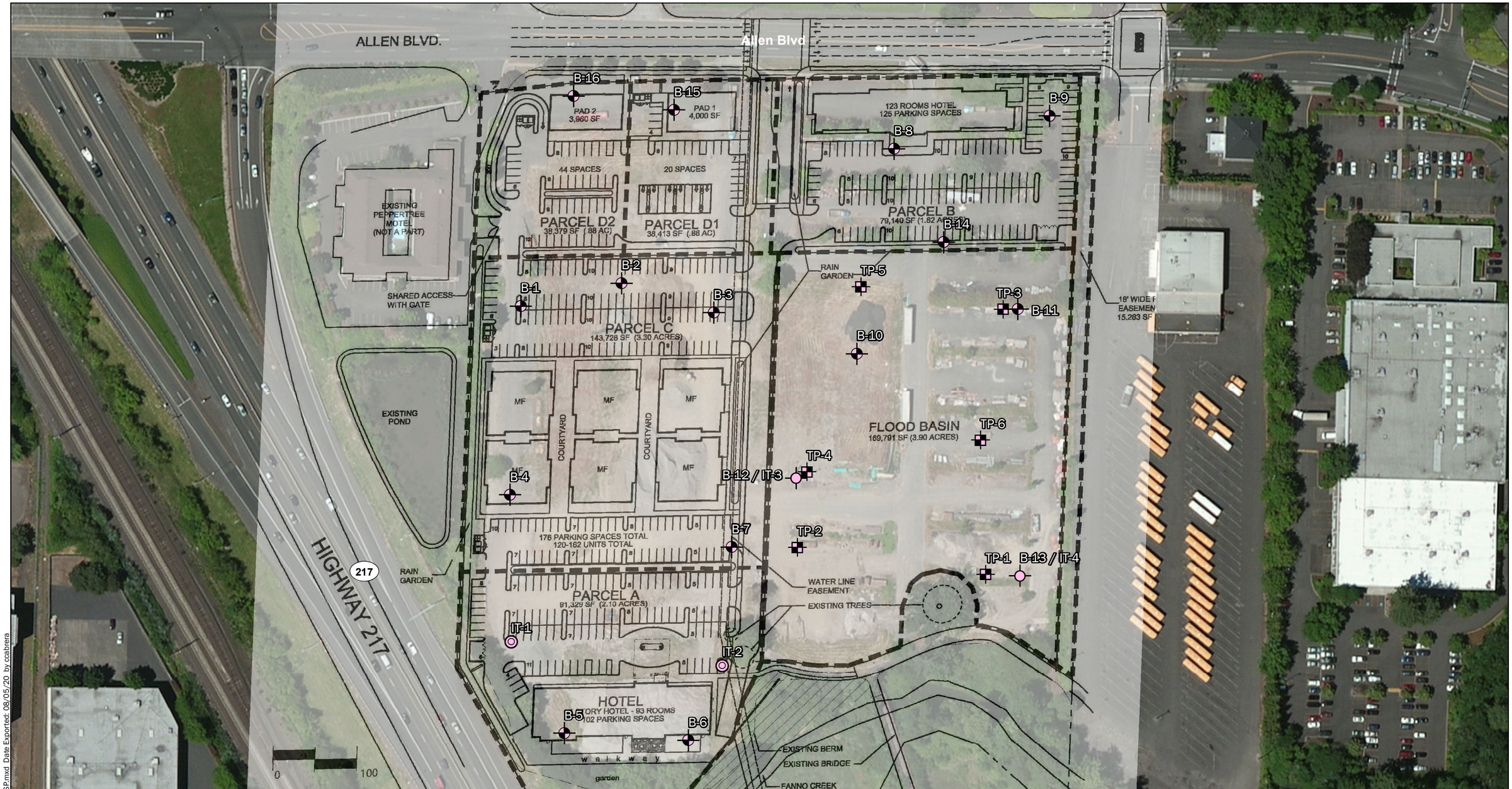

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers,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

Data Source: ESRI

Projection: NAD 1983 StatePlane Oregon North FIPS 3601 Feet

Legend

- Boring Number and Approximate Location
- Boring/Infiltration Test Number and Approximate Location
- Infiltration Test Number and Approximate Location
- Test Pit Number and Approximate Location

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Site Plan

Allen Boulevard Development
Beaverton, Oregon

GeoEngineers

Figure 2

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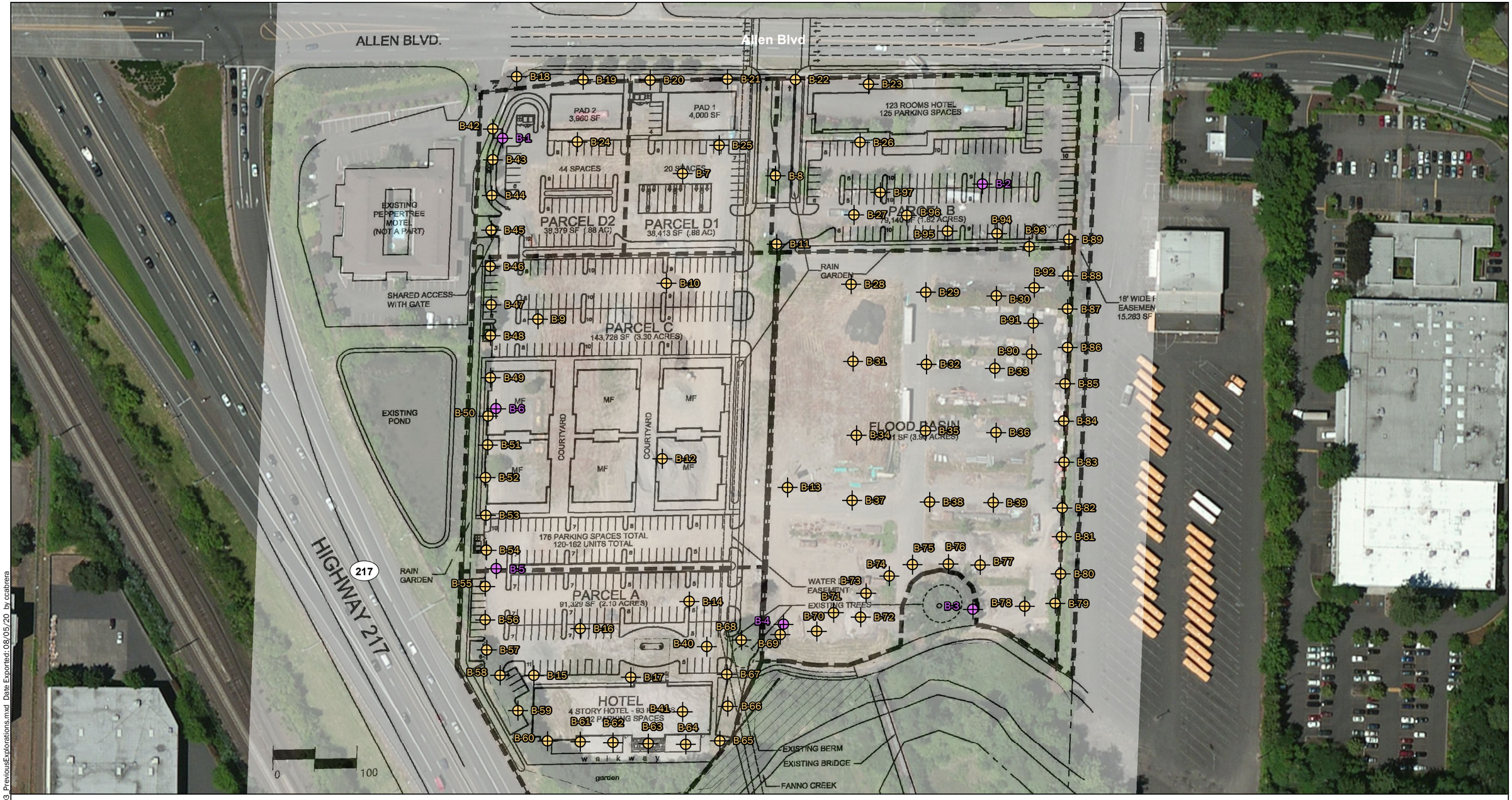

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers,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

Data Source: ESRI

Projection: NAD 1983 StatePlane Oregon North FIPS 3601 Feet

Legend

- Previous Boring Number and Approximate Location (2004)
- Previous Boring Number and Approximate Location (2006)

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Previous Exploration Locations

Allen Boulevard Development
Beaverton, Oregon

GEOENGINEERS

Figure 3

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APPENDICES

GeoEngineers

APPENDIX A
Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Soil and groundwater conditions at the site were explored between February 11 to February 15, 2019 by completing 16 borings (B-1 to B-16) and 4 infiltration test borings (IT-1 through IT-4) at the approximate locations shown in the Figure 2, Site Plan. The borings were advanced with mud-rotary and hollow-stem auger methods, using a CME 75 truck-mounted drill rig owned and operated by Western States Soil Conservation, Inc.

The drilling was continuously monitored by a staff engineer from our office who maintained a detailed log of subsurface explorations, visually classified the soil encountered and obtained representative soil samples from the borings. Samples were collected using a 1-inch, inside-diameter, standard split spoon sampler and a 3-inch, inside-diameter, Dames and Moore split spoon sampler. Samplers were driven into the soil using a manual driven 140-pound hammer, free-falling 30 inches on each blow. The number of blows required to drive the sampler each of three, 6-inch increments of penetration were recorded in the field. The sum of the blow counts for the last two, 6-inch increments of penetration was reported on the boring logs as the ASTM International (ASTM) Standard Practices Test Method D 1556 standard penetration test (SPT) N-value. The approximate N-values for D&M samples were converted to SPT N-values using the Lacroix-Horn Conversion [N(SPT) = (2*N1*W1*H1)/(175*D1*D1*L1), where N1 is the non-standard blowcount, W1 is the hammer weight in pounds (140), H1 is the hammer drop height in inches (30), D1 is the non-standard sampler outside diameter in inches (3.23) and L1 is the length of penetration in inches (12)].

Recovered soil samples were visually classified in the field in general accordance with ASTM D 2488 and the classification chart listed in Figure A-1, Key to Exploration Logs. The logs of the borings are presented in Figures A-2 through A-19, Logs of Borings. The log is 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.

Laboratory Testing

Soil samples obtained from the explorations were visually classified in the field and in our laboratory using the Unified Soil Classification System (USCS) and ASTM classification methods. ASTM Test Method D 2488 was used to visually classify the soil samples, while ASTM D 2487 was used to classify the soils based on laboratory tests results. Moisture content tests were performed in general accordance with ASTM D 2216-05 and moisture density tests of the ring samples were estimated in general accordance with ASTM Test Method D 7263. Two Atterberg limits test was performed in accordance with ASTM D 4318. Results of the moisture content and moisture density testing are presented in the appropriate exploration logs at the respective sample depths. The Atterberg limits test results are presented in Figure A-20, Atterberg Limits Test Results in this appendix.

We also completed two pH tests in general accordance with the ASTM D 4972 test method and two resistivity tests in general accordance with the ASTM test method G 51 on composite samples within borings B-1 to B-3 and B-8 to B-9. These tests are intended to serve as indicators of the corrosion potential of the native fine-grained soils. Test results are summarized in Table A-1 below.

TABLE A-1. PH AND CORROSIVITY TEST RESULTS

	Borings B-1 to B-3	Borings B-8 to B-9
Soil Classification	ML	ML
Sample Depth	2.5-7.5’	2.5-5’
pH	7.0	7.2
Resistivity (Ohm-cm)	1,000	2,500

Source: Palmer, J.F. 1974. "Soil Resistivity Measurements and Analysis," Materials Performance, Vol.13.

Based on the test results, we conclude that there is a moderate (B-1 to B-3) to high (B-8 to B-9) risk of corrosion to steel and iron pipes at the test site.

SOIL CLASSIFICATION CHART

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

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

ADDITIONAL MATERIAL SYMBOLS

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

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

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

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

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

Laboratory / Field Tests

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

Sheen Classification

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

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

Key to Exploration Logs

GEOENGINEERS

Figure A-1

Rev 06/2017

Drilled	Start 2/11/2019	End 2/11/2019	Total Depth (ft) 31.5	Logged By JJW	Checked By TNG	Driller Western States	Drilling Method Mud Rotary
Surface Elevation (ft) Vertical Datum	184 NGVD29		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment	CME-75 Truck
Latitude Longitude	45.283215 -122.471419		System Datum	WGS84 (feet)		Groundwater not observed at time of exploration	

Notes: D&M values reduced using Lacroix Horn Conversion to correlate with SPT N-values.

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							AC	Approximately 3 inches of asphalt concrete			
								GM	Approximately 12 inches of gray fine to coarse silty gravel (medium dense, moist) (fill)			
								ML	Gray silt (stiff, moist) (Willamette Silt)			
180		X	9	13		1 MC				37		PP=2 tsf
	5	X	0	5		2			Becomes medium stiff, wet			
175		X	18	7		3 MC				43		PP=1.5 tsf
									Grades with orange mottling			
	10	X	18	8		4 MC				42		PP=2.5 tsf
									Becomes dark gray, medium stiff to stiff			
170												
	15	X	18	26		5						PP=4.5 tsf
									Becomes very stiff			
165												
	20		18	8		6 MD						DD=96 pcf
									Grades with trace fine sand, medium stiff to stilff			
160												
	25	X	18	18		7			Becomes very stiff			PP=4 tsf
155												
	30	X	18	18		8						PP=4 tsf

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

Log of Boring B-1

GEOENGINEERS

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-2
Sheet 1 of 1

Drilled	Start 2/11/2019	End 2/11/2019	Total Depth (ft) 31.5	Logged By JJW	Checked By TNG	Driller Western States	Drilling Method Mud Rotary
Surface Elevation (ft) Vertical Datum	185 NGVD29		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment	CME-75 Truck
Latitude Longitude	45.283304 -122.471211		System Datum	WGS84 (feet)		Groundwater not observed at time of exploration	

Notes: D&M values reduced using Lacroix Horn Conversion to correlate with SPT N-values.

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							GM	Approximately 24 inches of gray fine to coarse gravel with concrete debris (medium dense, moist) (fill)			
			5	10		1 MC		ML	Light gray silt with orange mottling (stiff, moist) (Willamette Silt)	54		
180	5		9	10		2 AL			Grades with no orange mottling, wet	32		PP=0.5; AL(PI=10; LL=37)
			24	P		3						Driller indicates 525 psi during push
175	10		18	3		4 MD			Becomes soft			DD=76 pcf
170	15		18	3		5			Grades with trace fine sand			
165	20		18	12		6			Becomes stiff			
160	25		18	16		7			Becomes stiff to very stiff			PP=1.5
155	30		18	12		8			Becomes stiff			

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

Log of Boring B-2

GEOENGINEERS

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-3
Sheet 1 of 1

Drilled	Start 2/12/2019	End 2/12/2019	Total Depth (ft) 31.5	Logged By JJW	Checked By TNG	Driller Western States	Drilling Method Mud Rotary
Surface Elevation (ft) Vertical Datum	185 NGVD29		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment	CME-75 Truck
Latitude Longitude	45.283262 -122.471084		System Datum	WGS84 (feet)		Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							AC	Approximately 3 inches of asphalt concrete			
								GM	Gray silty fine to coarse gravel (medium dense, moist) (fill)			
		X	6	38		1 MC				18		
								SM	Gray silty fine to coarse sand with gravel (dense, wet)			
180	5	X	18	9		2 MC		ML	Gray silt (stiff, wet) (Willamette Silt)	40		PP=1.25
		X	18	5		3 MC				47		PP=1.25
									Becomes medium stiff			
175	10	X	18	6		4 MC				48		PP=0.75
170	15	X	18	12		5			Becomes stiff			
165	20	X	18	16		6			Grades with occassional fine sand, stiff to very stiff			
160	25	X	18	15		7			Grades with fine sand, becomes stiff			
155	30	X	18	10		8						

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

Log of Boring B-3

GEOENGINEERS

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-4
Sheet 1 of 1

Drilled	Start 2/12/2019	End 2/12/2019	Total Depth (ft) 41.5	Logged By JJW	Checked By TNG	Driller Western States	Drilling Method Mud Rotary
Surface Elevation (ft) Vertical Datum	183 NGVD29		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment	CME-75 Truck
Latitude Longitude	45.283009 -122.471432		System Datum	WGS84 (feet)		Groundwater not observed at time of exploration	

Notes: D&M values reduced using Lacroix Horn Conversion to correlate with SPT N-values.

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							AC	Approximately 3 inches of asphalt concrete			
								GM	Approximately 3 feet of gray silty gravel (medium dense, moist) (fill)			
180		X	5	48		1		SM	Gray silty fine to coarse sand with gravel (dense, wet)			
								ML	Gray silt with orange mottling (medium stiff, wet) (Willamette Silt)			
	5	X	18	5		2 MC				25		
175		X	18	5		3 MC				46		
	10	X	18	5		4 MC				40		
									Orange mottling, trace organic matter			
170												
	15	X	18	6		5						
									Grades with trace fine sand			
165												
	20	X	18	8		6						
									Becomes medium stiff to stiff			
160												
	25		18	13		7 MD			Becomes stiff			DD=93 pcf
155												
	30	X	18	13		8						
150												
	35											

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

Log of Boring B-4

GEOENGINEERS

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-5
Sheet 1 of 2

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		18	10		9						
145								SM	Brown silty fine sand (medium dense, wet)			
	40		18	16		10						

Log of Boring B-4 (continued)

GeoEngineers

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-5
Sheet 2 of 2

Drilled	Start 2/12/2019	End 2/12/2019	Total Depth (ft) 41.5	Logged By JJW	Checked By TNG	Driller Western States	Drilling Method Mud Rotary
Surface Elevation (ft) Vertical Datum	186.5 NGVD29		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment	CME-75 Truck
Latitude Longitude	45.282721 -122.471311		System Datum	WGS84 (feet)		Groundwater not observed at time of exploration	

Notes: D&M values reduced using Lacroix Horn Conversion to correlate with SPT N-values.

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							TS	Approximately 3 inches of brown silt with orgainic matter (Willamette Silt)			
185								ML	Gray silt with occasional fine sand and trace organic matter (stiff, wet)			
		☒	18	12		1 MC				36		
	5	☒	18	3		2 MC			Grades with no organic matter, trace occasional fine sand, soft	29		
180		☒	18	7		3 MC			Grades with orange mottling, medium stiff	32		
	10	☒	18	7		4 MC				43		PP=1.5
175									Grades with no fine sand			
	15	☒	18	3		5						PP=1.0
170									Becomes soft			
	20	▥	18	9		6 MD						DD=96 pcf
165									Becomes stiff			
	25	☒	18	12		7						
160									Grades with occasional fine sand			
	30	☒	18	14		8						
155												
	35											

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

Log of Boring B-5

GeoEngineers

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-6
Sheet 1 of 2

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		18	9		9						
150												
	40		18	14		10			Grades with fine sand			
145												

Log of Boring B-5 (continued)

GeoEngineers

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-6
Sheet 2 of 2

Drilled	Start 2/12/2019	End 2/12/2019	Total Depth (ft) 41.5	Logged By JJW	Checked By TNG	Driller Western States	Drilling Method Mud Rotary
Surface Elevation (ft) Vertical Datum	188 NGVD29		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment	CME-75 Truck
Latitude Longitude	45.2827 -122.471038		System Datum	WGS84 (feet)		Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							GM	Gray silty coarse gravel (medium dense, wet) (fill)			
185		X	5	14		1						
								ML	Brown silt with orange mottling (very soft, wet) (Willamette Silt)			
	5	X	18	2		2 MC				35		
180		X		3		3 MC			Grades with no mottling, soft	35		
	10		18	5		4 MD			Becomes medium stiff			DD=90 pcf
175												
	15	X	18	7		5			Grades with occasional fine sand			
170												
	20	X	18	49		6a 6b		SM	Gray silty medium sand (dense, wet)			
								ML	Gray silt (hard, wet)			
165												
	25	X	18	22		7			Becomes very stiff			
160												
	30	X	18	7		8			Becomes medium stiff			
155												
	35											

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

Log of Boring B-6

GEOENGINEERS

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-7
Sheet 1 of 2

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
150	35	X	18	3		9			Becomes soft			
	40	X	18	16		10		SM	Reddish-brown silty medium to coarse sand (medium dense, wet)			

Log of Boring B-6 (continued)

GeoEngineers

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-7
Sheet 2 of 2

Drilled	Start 2/13/2019	End 2/13/2019	Total Depth (ft) 41.5	Logged By JJW	Checked By TNG	Driller Western States	Drilling Method Mud Rotary
Surface Elevation (ft) Vertical Datum	184 NGVD29		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment	CME-75 Truck
Latitude Longitude	45.282938 -122.471035		System Datum	WGS84 (feet)		Groundwater not observed at time of exploration	

Notes: D&M values reduced using Lacroix Horn Conversion to correlate with SPT N-values.

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							AC	Approximately 3 inches of asphalt concrete			
								GM	Approximately 12 inches of gray silty fine to coarse gravel (medium dense, moist) (fill)			
180			15	10		1 MC		ML	Gray silt with orange mottling (stiff, moist)	27		PP=1
	5		18	3		2 MC			Grades with no mottling, soft, wet	33		PP=0.5
175			18	3		3 MC			Grades to dark brown	20		PP=0.5
	10		18	5		4 MC			Grades to dark gray, medium stiff	50		PP=1.5
170												
	15		18	3		5			Grades with occasional fine sand, soft			PP=0.5
165												Driller notes stiffening at 19 feet
	20		18	14		6			Becomes sandy, stiff			
160								ML/SM	Gray sandy silt/silty fine sand (very stiff/medium dense, wet)			
	25		18	21		7 MD						DD=99 pcf
155								ML	Gray silt with fine sand (very stiff, wet)			
	30		18	18		8						
150	35											

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

Log of Boring B-7

GEOENGINEERS

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-8
Sheet 1 of 2

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		18	5		9			Grades with occasional fine sand, medium stiff			
145												
	40		18	17		10			Grades to light brown with trace fine sand, very stiff			

Log of Boring B-7 (continued)

GeoEngineers

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-8
Sheet 2 of 2

Drilled	Start 2/13/2019	End 2/13/2019	Total Depth (ft) 51.5	Logged By JJW	Checked By TNG	Driller Western States	Drilling Method Mud Rotary
Surface Elevation (ft) Vertical Datum	188 NGVD29		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment	CME-75 Truck
Latitude Longitude	45.283454 -122.470764		System Datum	WGS84 (feet)		Groundwater not observed at time of exploration	

Notes: D&M values reduced using Lacroix Horn Conversion to correlate with SPT N-values.

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							AC	Approximately 3 inches of asphalt concrete			
								GM	Approximately 12 inches of gray silty fine to coarse gravel (medium dense, moist) (fill)			
185			9	7		1 MC		ML	Brown silt with trace fine sand and gravel (medium stiff, moist)	26		
	5		5	9		2 MC				30		
									Becomes stiff, wet			
180			18	7		3 AL			Becomes medium stiff	34		AL(PI=3; LL=32)
	10		5	10		4 MC				38		
									Grades to dark gray, stiff			
175												
	15					5						
170			18	14		6			Grades with occasional fine sand			
	20		18	5		7						
									Grades with 1-inch interbedded fine sand lenses			
165												
	25		18	10		8 MD						DD=91 pcf
									Grades with fine sand, stiff			
160												
	30		18	10		9						
155												
	35											

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

Log of Boring B-8

GEOENGINEERS

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-9
Sheet 1 of 2

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35	X	18	10		10						
150												
	40	X	18	24		11						
145												
	45	■	18	9		12		CL	Gray clay (stiff, wet) (Hillsboro Formation)			
140												
	50	X	18	31		13			Becomes very stiff			

Log of Boring B-8 (continued)

GEOENGINEERS

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-9
Sheet 2 of 2

Drilled	Start 2/14/2019	End 2/14/2019	Total Depth (ft)	51.5	Logged By	JJW	Checked By	TNG	Driller	Western States	Drilling Method	Mud Rotary
Surface Elevation (ft) Vertical Datum	192 NGVD29				Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop			Drilling Equipment	CME-75 Truck		
Latitude Longitude	45.283514 -122.470533				System Datum	WGS84 (feet)			Groundwater not observed at time of exploration			

Notes: D&M values reduced using Lacroix Horn Conversion to correlate with SPT N-values.

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							AC	Approximately 3 inches of asphalt concrete			
								GM	Approximately 12 inches of gray silty fine to coarse gravel (medium dense, moist) (fill)			
190			18	7		1 MC		ML	Brown silt with trace organic matter (medium stiff, moist) (Willamette Silt)	32		
	5		18	5		2						
185									Grades with no organic matter, with occasional fine sand, orange mottling			
			18	6		3 MC				30		
									Grades with fine sand			
	10		10	2		4						
180									Grades to gray with no orange mottling, sandy, very soft to soft			
						5						
								SM	Gray fine to medium silty sand (dense, wet)			
	15		18	44		6a 6b		ML	Gray silt with occasional fine sand (hard, wet)			
175												
	20		18	7		7 MD			Becomes medium stiff			DD=87 pcf
170												
	25		18	17		8			Grades with fine sand, very stiff			
165												
	30		18	2		9			Grades with no fine sand, very soft to soft			
160												
	35											

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

Date:4/30/19 Path:P:\23\23648001\GINT\23648001GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_STANDARD_%F_NO_GW

Log of Boring B-9

GeoEngineers

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-10
Sheet 1 of 2

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35	X	18	12		10			Grades with trace fine sand, stiff			PP=2.0
155												
	40			11		11		SM	Brown-reddish silty fine to medium sand (medium dense, wet)			
150								ML	Gray silt with orange mottling (stiff, wet)			
	45	X	18	11		12						
145												
	50	X	18	18		13			Becomes very stiff			

Log of Boring B-9 (continued)

GEOENGINEERS

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-10
Sheet 2 of 2

Drilled	Start 2/14/2019	End 2/14/2019	Total Depth (ft) 26.5	Logged By JJW	Checked By TNG	Driller Western States	Drilling Method Mud Rotary
Surface Elevation (ft) Vertical Datum	186 NGVD29		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment	CME-75 Truck
Latitude Longitude	42.283197 -122.47083		System Datum	WGS84 (feet)		Groundwater not observed at time of exploration	

Notes: D&M values reduced using Lacroix Horn Conversion to correlate with SPT N-values.

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
185	0							GM	Gray silty gravel with concrete debris (very dense, wet) (fill)			
			3	50		1						
180	5		18	6		2 MC		ML	Gray silt (medium stiff, wet) (Willamette Silt)	28		
			0	6		3						
175	10		18	8		4 MC			Becomes medium stiff to stiff	27		
170	15		18	8		5			Grades with fine to medium sand			
165	20		0	5		6			Becomes medium stiff			
160	25		1	14		7			Grades with no sand, stiff			

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

Log of Boring B-10

GEOENGINEERS

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-11
Sheet 1 of 1

Drilled	Start 2/15/2019	End 2/15/2019	Total Depth (ft) 26.5	Logged By JJW	Checked By TNG	Driller Western States	Drilling Method Mud Rotary
Surface Elevation (ft) 187.5	Vertical Datum NGVD29		Hammer Data Autohammer 140 (lbs) / 30 (in) Drop			Drilling Equipment CME-75 Truck	
Latitude 45.283262	Longitude -122.470538		System Datum WGS84 (feet)			Groundwater not observed at time of exploration	

Notes: D&M values reduced using Lacroix Horn Conversion to correlate with SPT N-values.

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							ML	Gray silt with orange mottling (medium stiff, moist) (Willamette Silt)			
185		X	18	7		1 MC				32		
	5		18	9		2			Grades to brown with no mottling, stiff			
180		X	18	4		3 MC 4			Grades with fine sand, soft to medium stiff, wet	33		
	10		18	6		5			Grades with trace sand, medium stiff			
175												
	15	X	18	4		6			Becomes soft to medium stiff			
170								SM	Gray silty fine sand (medium dense, wet)			
	20		18	13		7						
165								ML	Gray silt with occasional fine sand (very stiff, wet)			
	25	X	18	23		8						

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

Log of Boring B-11

GEOENGINEERS

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-12
Sheet 1 of 1

Drilled	Start 2/15/2019	End 2/15/2019	Total Depth (ft) 21.5	Logged By JJW Checked By TNG	Driller Western States	Drilling Method Hollow-stem Auger
Surface Elevation (ft) Vertical Datum	185 NGVD29		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME-75 Truck
Latitude Longitude	45.283051 -122.470934		System Datum	WGS84 (feet)	See "Remarks" section for groundwater observed	

Notes: D&M values reduced using Lacroix Horn Conversion to correlate with SPT N-values.

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							GM	Gray silty gravel with fine to coarse sand (medium dense, wet) (fill)			
		X	12	17		1						Groundwater measured at 2¼ feet at 5:45 p.m. 2/14/19 Groundwater measured at 2½ feet at 8 a.m. and 10:40 a.m. on 2/15/19
180	5	X	18	4		2 MC		ML	Gray silt with orange mottling (soft to medium stiff, wet) (Willamette Silt)	27		PP=1 tsf
175	10		18	5		3			Becomes medium stiff			
170	15	X	18	10		4			Becomes stiff			
165	20	X	18	8		5			Grades with occasional fine sand (medium stiff to stiff)			

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

Log of Boring B-12/IT-3

GEOENGINEERS

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-13
Sheet 1 of 1

Drilled	Start 2/13/2019	End 2/15/2019	Total Depth (ft) 21.5	Logged By JJW	Checked By TNG	Driller Western States	Drilling Method Hollow-stem Auger
Surface Elevation (ft) Vertical Datum	187 NGVD29		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment	CME-75 Truck
Latitude Longitude	45.282962 -122.470543		System Datum	WGS84 (feet)		See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							AC	Approximately 1½ inches of asphalt concrete			
185								ML	Gray-brown silt (stiff, wet) (Willamette Silt)			Groundwater measured at 1½ feet at 8 a.m and 10:40 p.m. 2/15/19 Groundwater measured 2½ feet at 5:45 p.m. 2/14/19
		X	18	10		1 MC				28		
	5	X	18	7		2						
180									Grades with occasional fine sand, medium stiff			
		X	18	6		3 MC				27		
									Grades to brown, with fine sand, wet			
	10											
175												
	15	X	18	7		4			Grades to gray with fine sand			
170												
	20	X	18	17		5			Becomes very stiff			

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

Log of Boring B-13/IT-4

GeoEngineers

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-14
Sheet 1 of 1

Drilled	Start 2/15/2019	End 2/15/2019	Total Depth (ft) 6.5	Logged By JJW	Checked By TNG	Driller Western States	Drilling Method Mud Rotary

Surface Elevation (ft) Vertical Datum	187 NGVD29	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME-75 Truck
Latitude Longitude	45.283407 -122.470663	System Datum	WGS84 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							AC	Approximately 1½ inches of asphalt concrete			
185								ML	Brown silt (medium stiff, moist) (Willamette Silt)			
		X	18	6		1 / MC				28		
	5	X	18	8		2 / MC				30		
									Becomes medium stiff to stiff			

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

Log of Boring B-14

GeoEngineers

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-15
Sheet 1 of 1

Drilled	Start 2/15/2019	End 2/15/2019	Total Depth (ft) 6.5	Logged By JJW	Checked By TNG	Driller Western States	Drilling Method Mud Rotary
Surface Elevation (ft) Vertical Datum	183 NGVD29		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment	CME-75 Truck
Latitude Longitude	45.283491 -122.471162		System Datum	WGS84 (feet)		Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							AC	Approximately 1½ inches of asphalt concrete			
								ML	Gray silt (stiff, moist) (Willamette Silt)			
180		X	18	9		1 MC				26		
	5	X	18	6		2 MC				30		
									Becomes medium stiff			

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

Log of Boring B-15

GEOENGINEERS

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-16
Sheet 1 of 1

Drilled	Start 2/15/2019	End 2/15/2019	Total Depth (ft) 6.5	Logged By JJW	Checked By TNG	Driller Western States	Drilling Method Mud Rotary
Surface Elevation (ft) Vertical Datum	183 NGVD29		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment	CME-75 Truck
Latitude Longitude	45.28351 -122.47135		System Datum	WGS84 (feet)		Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							AC	Approximately 1½ inches of asphalt concrete			
180		X	18	9		1		GM	Gray silty coarse to fine gravel with coarse sand (loose, wet) (fill)			
	5	X	18	7		2 MC		ML	Gray silt (medium stiff, wet) (Willamette Silt)	37		

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

Log of Boring B-16

GeoEngineers

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-17
Sheet 1 of 1

Drilled	Start 2/11/2019	End 2/11/2019	Total Depth (ft) 5	Logged By JJW	Checked By TNG	Driller Western States	Drilling Method Hollow-stem Auger
Surface Elevation (ft) Vertical Datum	184 NGVD29		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop		Drilling Equipment	CME-75 Truck
Latitude Longitude	45.282833 -122.471432		System Datum	WGS84 (feet)		See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							ML	Gray silt (medium stiff, wet) (Willamette Silt)			
180		☒	18	7		1						Groundwater observed at approximately 3 feet below ground surface
	5											

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

Log of Boring IT-1

GEOENGINEERS

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-18
Sheet 1 of 1

Drilled	Start 2/11/2019	End 2/11/2019	Total Depth (ft) 5	Logged By JJW / Checked By TNG	Driller Western States	Drilling Method Hollow-stem Auger
Surface Elevation (ft) / Vertical Datum	187 NGVD29		Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME-75 Truck
Latitude / Longitude	45.282778 -122.471025		System Datum	WGS84 (feet)	See "Remarks" section for groundwater observed	

Notes:

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							TS	Approximately 3 inches of brown silt with organic matter (topsoil)			
185								ML	Gray silt with occasional sand and gravel (stiff, wet) (Willamette Silt)			
		X	15	11		1						Groundwater observed at 2½ feet below ground surface
	5											

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

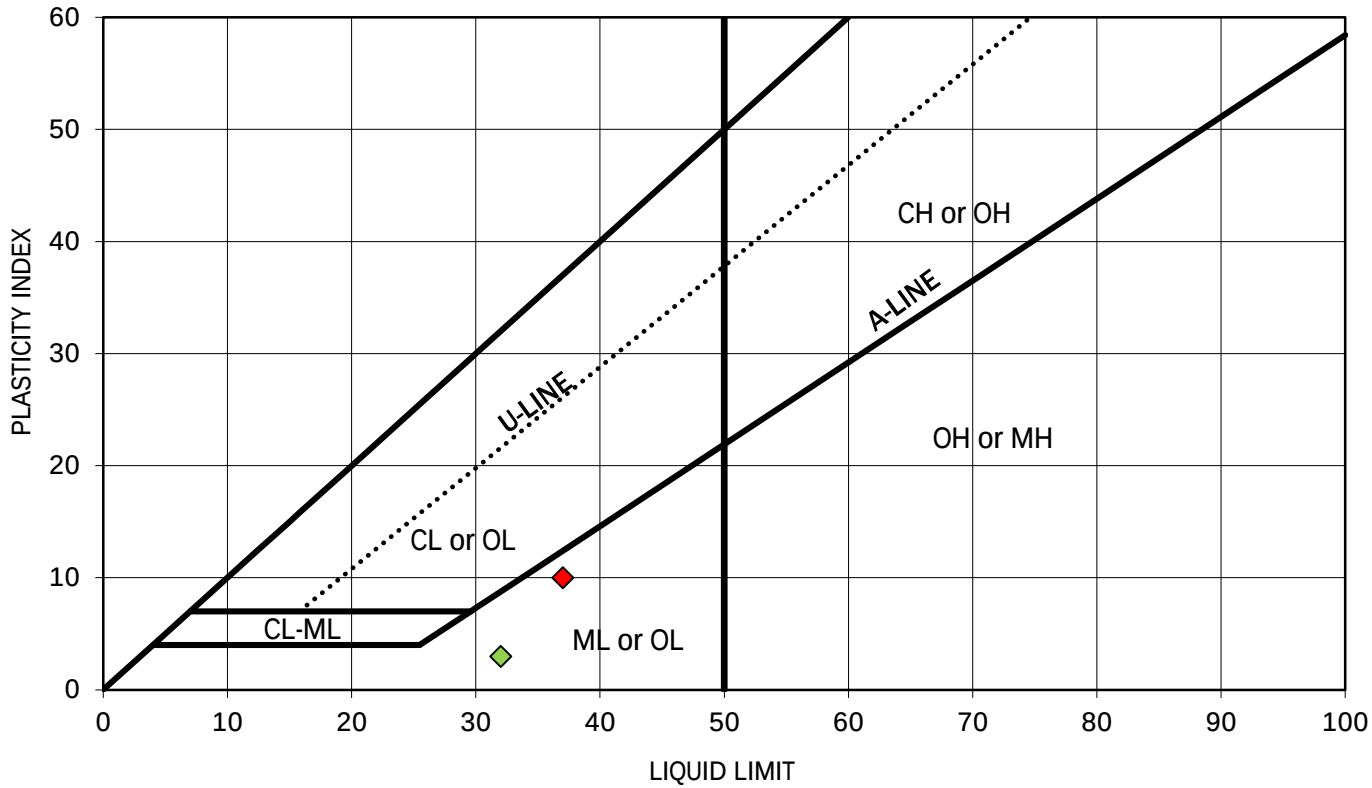
Log of Boring IT-2

GeoEngineers

Project: Allen Boulevard Development
Project Location: Beaverton, Oregon
Project Number: 23648-001-00

Figure A-19
Sheet 1 of 1

PLASTICITY CHART



Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆	B-2	5	32	37	10	Gray silt (ML)
◆	B-8	7.5	35	32	3	Brown silt (ML)

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

The liquid limit and plasticity index were obtained in general accordance with ASTM D 4318.

Atterberg Limits Test Results

Allen Boulevard Development
Beaverton, Oregon

GeoEngineers

Figure A-20

23648-001-00 Date Exported: 02/27/19 CJE

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 about 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 Oregon Worsted Company 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, our services have been executed in accordance with our Agreement with Oregon Worsted Company dated December 11, 2018 (authorized January 28, 2019) and generally accepted geotechnical practices in this area at the time this report was prepared. 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 proposed Allen Boulevard Development Project southeast of Highway 217 and SW Allen Boulevard in Beaverton, Oregon. 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 GBA, GeoProfessional Business Association; www.geoprofessional.org.

- elevation, configuration, location, orientation or weight of the proposed structure;

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.

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.

GeoEngineers