CENTRAL
GEOTECHNICAL SERVICES

7662 SW Mohawk Street
Tualatin, Oregon 97062
503.616.9419
www.centralgeotech.com

August 28, 2024

Tualatin Hills Park & Recreation District
c/o AKS Engineering & Forestry, LLC
12965 SW Herman Road, Suite 100
Tualatin, Oregon 97062

Attention: Chris Knight (knightc@aks-eng.com)

Re: Geotechnical Engineering Report
CGS Project No. AKS-4-01
THPRD – HMT Pickleball Court (AKS Job No. 3291-01)
Howard M. Terpenning Recreation Complex Outdoor Court Expansion
15707 SW Walker Road
Beaverton, Oregon

Central Geotechnical Services, LLC (CGS) is pleased to submit this geotechnical engineering report for the proposed Howard M. Terpenning (HMT) Recreation Complex outdoor court expansion project located at 15707 SW Walker Road in Beaverton, Oregon. The report was prepared for conformance with the signed contract dated November 14, 2023. We appreciate the opportunity to be of service to AKS Engineering & Forestry (AKS) and Tualatin Hills Park & Recreation District (THPRD). Please feel free to call our office with questions about this report.

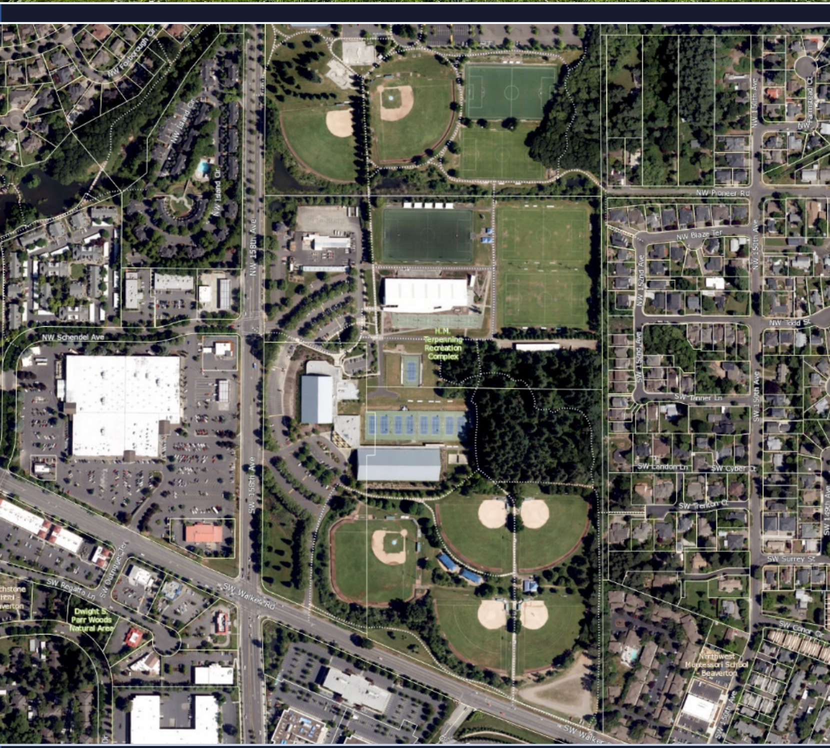
Respectfully,

Central Geotechnical Services, LLC

____
Krey D. Younger, P.E., G.E.
Principal Geotechnical Engineer

Geotechnical Engineering Services:

THPRD – HMT Pickleball Court
15717 SW Walker Road
Beaverton, Oregon



CGS Project: AKA-4-01

Prepared For:

Tualatin Hills Park & Recreation District
c/o AKS Engineering and Forestry, LLC
12965 SW Herman Road, Suite 100
Tualatin, Oregon 97062

August 28, 2024

Submitted by:

CENTRAL
Geotechnical Services, LLC

Table of Contents

1.0 INTRODUCTION 3

2.0 SCOPE OF SERVICES 3

3.0 SITE CONDITIONS 4

3.1. Site Geology 4

3.2. Surface Conditions 4

3.3. Subsurface Conditions 4

3.4. Soil Conditions 5

3.5. Groundwater Conditions 5

3.6. Dynamic Cone Penetration Testing 5

4.0 INFILTRATION TESTING 5

4.1. Test Procedure and Results 5

4.2. Suitability of Infiltration System 7

5.0 EARTHWORK RECOMMENDATIONS 7

5.1. Site Preparation 7

5.2. Demolition 7

5.3. Clearing and Grubbing 7

5.4. Stripping 8

5.5. Site Subgrade Preparation and Evaluation 8

5.6. Subgrade Protection and Wet Weather Considerations 8

5.7. Dewatering 9

5.8. Permanent Slopes 9

5.9. Temporary Excavations 9

5.10. Structural Fill and Backfill 10

5.11. Pavement Section Materials 11

6.0 STRUCTURAL DESIGN RECOMMENDATIONS 12

6.1. Foundation Support Recommendations 12

6.2. Drainage Considerations 13

6.3. Floor Slabs 14

6.4. Seismic Design 14

7.0 CONVENTIONAL RETAINING WALLS 15

7.1. Design Parameters 15

7.2. Retaining Wall Drainage Considerations 16

8.0 LIGHT POLE FOUNDATION RECOMMENDATION........16
9.0 PAVEMENT DESIGN RECOMMENDATIONS FOR SPORTS COURT AND FIRE LANE........17
9.1. AC over Aggregate Base Section........17
9.2. Drainage Considerations for Sport Court........18
10.0 DESIGN REVIEW AND CONSTRUCTION SERVICES........18
11.0 LIMITATIONS OF REPORT........18
12.0 REFERENCES........19
13.0 SIGNATURES........20

LIST OF FIGURES

Figure 1. Vicinity Map
Figure 2. Site Plan

APPENDICES

Appendix A. Field Explorations and Laboratory Testing
Figure A-1. Soil Classification System
Figure A-2 through A-8. Logs of Explorations

1.0 INTRODUCTION

Central Geotechnical Services, LLC (CGS) is pleased to submit this geotechnical engineering report for the proposed multi-use sports court expansion project at the Howard M. Terpenning (HMT) Recreation Complex located at 15707 SW Walker Road in Beaverton, Oregon. The project area is currently developed with an outdoor court and stadium seating for tennis and pickleball. The court is surrounded by a perimeter retaining wall with a terrace lawn above along the east, west, and south sides and a memorial plaza to the north. The location of the site is shown in the Vicinity Map, Figure 1.

Our understanding of the project was developed from discussions with and information provided to us by Mr. Chris Knight of AKS Engineering & Forestry, LLC (AKS), aerial images of the area from Google Earth, and geologic maps and geotechnical reports for the area in our files. Based on the information provided, we understand the project will include an air-supported fabric structure, asphalt courts with acrylic surfacing, perimeter fencing, retaining walls up to 8 feet tall, ADA pathways and plaza, perimeter court lights, a metal storage unit, a potential restroom, and a mechanical ventilation system. The project may also include a stormwater facility and a new fire truck access lane/turnaround.

At the time this report was prepared, specific structural loads were not provided. We have assumed typical load levels for the type of structure proposed to develop the proposed scope. We have also assumed that the existing court surfacing and related improvements will be removed and replaced as part of the expansion project. If site or structural development vary from the information noted above, the recommendations presented in this report should be reviewed and revised as needed.

2.0 SCOPE OF SERVICES

The purpose of this project is to provide geotechnical engineering services for the proposed court expansion project to THPRD through AKS. Our proposed scope of services is based on our understanding of the project and information provided by the project team. Our specific scope of services included the following:

1. Review of preliminary plans related to geotechnical design and construction elements provided by the project team.
2. Provided an exploration work plan detailing the approximate location and proposed depth of explorations.
3. Completed utility locates through the public one call system and private locate company.
4. Drilled five geotechnical borings to a depth between 5 to 21.5 feet bgs.
5. Completed infiltration testing at two locations recommended by the design team.
6. Conducted laboratory testing of select soil samples to evaluate pertinent physical and engineering properties.
7. Provided a geotechnical evaluation of the site and provided design recommendations in this geotechnical report addressing the following geotechnical components.
    a. A general description of the site topography, geology, and subsurface conditions.

b. Summary of infiltration testing and a discussion on suitability of on-site infiltration facilities based on subsurface conditions.

c. Recommendations for site preparation measures.

d. Recommendations for temporary excavation and temporary excavation protection.

e. Recommendations for earthwork construction.

f. Recommendations for site retaining walls for walls up to 8 feet tall.

g. Recommendations for foundations to support the proposed structures.

h. Recommendations for light pole foundations.

i. Recommendations for a pavement design section thickness.

Our geotechnical work has been directly supervised by a professional engineer licensed in the state of Oregon.

3.0 SITE CONDITIONS

3.1. Site Geology

The preliminary geologic map of the Linnton 7.5-minute quadrangle, Multnomah, and Washington Counties, Oregon (Madin et al. 2008) shows the project area as mantled by Missoula flood deposits of the latest Pleistocene. They define this alluvial sediment as silt, sand, and minor gravel deposited by the Missoula flood events, which is generally consistent with the subsurface conditions observed in our explorations.

3.2. Surface Conditions

The proposed sports court expansion is generally located in the current outdoor stadium court area in the central portion of the HMP Recreation Complex. The project area is currently developed with a stadium bleacher sports court, a perimeter retaining wall along the east, west and south with terrace lawn areas above, and a memorial plaza to the north. The existing court is surfaced with asphalt, and the retaining wall is up to 8 feet tall. The site generally slopes to the north at an inclination of less than 10 percent grade, and the landscaped areas are generally vegetated with grass.

Aerial images from the city of Portland database (PortlandMaps) show that the existing stadium court was constructed in the summer of 2004.

3.3. Subsurface Conditions

Subsurface conditions at the site were explored by completing five drilled exploratory borings (B-1 and B-5) and two infiltration test borings (INF-1 and INF-2). The borings were advanced to depths between 5 to 21.5 feet bgs on October 27 and 28, 2023. Soil cuttings from each boring were visually classified and logged. Descriptions of the subsurface exploration program and logs of explorations are presented in Appendix A.

Approximate locations of explorations are shown in the Site Plan, Figure 2.

3.4. Soil Conditions

In general, the subsurface conditions beneath the landscaping grass consists of medium-stiff to stiff silt extending to the maximum depth explored of 21.5 feet bgs. The silt is brown to gray and contains traces of fine sand. The stiffness of the silt generally decreases with depth.

3.5. Groundwater Conditions

Groundwater was encountered during drilling in borings B-1, B-3, and B-4 at a depth of between 14.5 and 19.5 feet bgs. Groundwater conditions at the site are expected to vary seasonally due to rainfall events and other factors not observed in our explorations.

3.6. Dynamic Cone Penetration Testing

We conducted DCP testing in general accordance with ASTM D6951 to estimate subgrade resilient modulus at two locations. We recorded penetration depth of the cone for each blow of the hammer and terminated testing when at refusal of penetration or end of rod length. We plotted depth of penetration versus blow count and visually assessed where the slope of the data plot was relatively constant and at depths where the slope of the data plot changed significantly. We used the slope of the data to estimate the resilient modulus of the subgrade. We used least squares regression to determine the slopes and the equation from the Oregon Department of Transportation (ODOT) Pavement Design Guide to estimate the resilient moduli using a correction factor (cf) of 0.35. Table 1 lists our estimates of subgrade resilient modulus at each test location.

TABLE 1. SUMMARY of ESTIMATED RESILIENT MODULUS

Location	Estimated Subgrade Resilient Modulus (psi)[1]
DCP-1	5,470
DCP-2	4,650

1: psi = pounds per square inch

4.0 INFILTRATION TESTING

We performed falling head infiltration testing at locations and depths provided by AKS as noted on the attached Figure 2 Site plan and Table 2 below. This test procedure includes drilling a hole using an 8.25-inch outside diameter, 6-inch inside diameter, hollow stem auger. The explorations were started on October 27, 2023 and the infiltration test depth was allowed to saturate over an approximate 24-hour period prior to completing testing.

4.1. Test Procedure and Results

The maximum head pressure during the tests was 14 to 19 inches. Water levels were measured at periodic intervals from a fixed reference point with a standard steel tape measured to the nearest 1/16 of an inch or a water level logger. The measurements were repeated until consistent falling head rates were obtained. Table 2 below presents the summary of the test results.

TABLE 2. SUMMARY OF INFILTRATION FIELD TESTING

Infiltration Test No.	Location	Depth (feet)	USCS Material Type[1]	Percent Passing U.S. Standard No. 200 Sieve	Field Measured Infiltration Drawdown Rate (in/hr) [2,3]
INF-1	See Site Plan	5.0	ML	98	0.1
INF-2	See Site Plan	8.0	ML	98	0.2

Notes:

[1]USCS = Unified Soil Classification System

[2]Appropriate factors should be applied to the field-measured infiltration drawdown rate, based on the design methodology and specific system used.

[3]in/hr = inches per hour.

The infiltration rates shown in Table 2 represent field-measured infiltration rates. Field measurements are limited to the accuracy of equipment employed to conduct the test.

Field-measured rates represent a relatively short-term infiltration rate, and factors of safety have not been applied for the type of infiltration system being considered, or for variability that may be present across large areas in the on-site soil. In our opinion, and consistent with the state of the practice, correction factors should be applied to this measured rate to reflect the localized area of testing relative to the field sizes.

Appropriate correction factors should also be applied by the project civil engineer to account for long-term infiltration parameters. From a geotechnical perspective, we recommend a factor of safety (correction factor) of at least 2 be applied to the field infiltration values to account for potential soil variability with depth and location within the area tested. In addition, the stormwater system design engineer should determine and apply appropriate remaining correction factor values, or factors of safety, to account for repeated wetting and drying that occur in this area, degree of in-system filtration, frequency, and type of system maintenance, vegetation, potential for siltation and bio-fouling, etc., as well as system design correction factors for overflow or redundancy, and base and facility size.

The actual depths, lateral extent, and estimated infiltration rates can vary from the values presented above. Field testing/confirmation during construction is often required in large or long systems or other situations where soil conditions may vary within system construction area. The results of this field testing might necessitate that the infiltration locations be modified to achieve the design infiltration rate.

The infiltration flow rate of a focused stormwater system like a drywell or small infiltration box or pond typically diminishes over time as suspended solids and precipitates in the stormwater further clog the void spaces between the soil particles or cake on the infiltration surface or in the engineered media. The serviceable life of an infiltration media in a stormwater system can be extended by pre-filtering or with ongoing accessible maintenance. Eventually, most systems will fail and will need to be replaced or have media regenerated or replaced.

We recommend that infiltration systems include an overflow that is connected to a suitable discharge point. Also, infiltration systems can cause localized, high groundwater levels and should not be located near basement walls, retaining walls, or other embedded structures unless these are specifically designed to account for the resulting hydrostatic pressure. Infiltration locations should not be located on sloping ground unless it is approved by a geotechnical engineer, and should not be infiltrated at a location that allows for flow to travel laterally toward a slope face, such as a mounded water condition or too close to a slope face that could cause instability of the slope.

4.2. Suitability of Infiltration System

Successful design and implementation of stormwater infiltration systems and whether a system is suitable for development depends on several site-specific factors. Stormwater infiltration systems are generally best suited for sites having sandy or gravelly soil with saturated hydraulic conductivities greater than 2 in/hr. The fine-grained, silt soils at this site are generally not well-suited for long-term stormwater infiltration or as a sole method of stormwater infiltration. Soils that have fine-grained matrices are susceptible to volumetric change and softening during wetting and drying cycles. Fine-grained soils also have large variations in the magnitude of infiltration rates because of bedding and stratification that occurs during alluvial deposition and often have thin layers of less permeable or impermeable soil within a larger layer.

As a result of fine-grained soil conditions and relatively low field measured infiltration rates, we recommend infiltration of stormwater not be used in the upper soils at the site as the sole method of stormwater management unless those design factors can be otherwise accounted for by increasing infiltration area or coupling with other methods of stormwater disposal. At a minimum, an overflow method should be provided for the overall system.

5.0 EARTHWORK RECOMMENDATIONS

5.1. Site Preparation

Initial site preparation includes stripping and grubbing of upper organics and removal of subsurface structures.

5.2. Demolition

All structures and belowground structures to be demolished should be completely removed from proposed structural areas and for a margin of at least 3 feet around proposed structural areas. Proposed structural areas are areas where new structures and pavement areas will be built, including the new sports court, retaining walls, storage unit and restroom, new sidewalks, and firetruck access lane. Existing utilities that will be abandoned on site should be identified prior to construction. Abandoned utility lines should be completely removed or filled with grout if left in place to reduce potential settlement or caving in the future. Materials generated during demolition should be transported off-site and properly disposed of.

Debris materials generated during demolition of existing improvements or relocation of utilities should be transported off-site for disposal. Existing voids and new depressions created during site preparation, and resulting from removal of existing utilities, or other subsurface elements, should be cleaned of loose soil or debris down to firm soil and backfilled with compacted structural fill.

5.3. Clearing and Grubbing

Site clearing will be required to remove vegetation in landscaped areas. Excavations to remove root zones should be done with a smooth bucket to minimize subgrade disturbance. Grubbed materials should be hauled off-site and properly disposed of unless otherwise allowed by the project specifications for other uses such as landscaping, stockpiling, or on-site burning.

Existing voids and new depressions created during demolition, clearing, grubbing, or other site preparation activities should be excavated to firm soil and backfilled with Imported Select Structural Fill. Greater depths of disturbance should be expected if site preparation and earthwork are conducted during periods of wet weather.

5.4. Stripping

Based on our observations at the site, we estimate that the depth of stripping should be on the order of about 8 to 9 inches. Greater stripping depths may be required to remove localized zones of loose or organic soil and in areas where moderate to heavy vegetation is present or where surface disturbance from prior use has occurred. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off-site for disposal unless otherwise allowed by the project specifications for other uses such as landscaping.

5.5. Site Subgrade Preparation and Evaluation

Upon completion of site preparation activities, exposed subgrades should be proof-rolled with a fully loaded dump truck or similar heavy rubber-tired construction equipment where space allows to identify soft, loose, or unsuitable areas. Probing may be used for evaluating smaller areas or where proof-rolling is not practical. Proof-rolling and probing should be conducted prior to placing fill and should be performed by a representative of CGS 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 or probing, these areas should be excavated to the extent indicated by our representative and replaced with structural fill.

5.6. Subgrade Protection and Wet Weather Considerations

The fine-grained soils at the site are highly susceptible to disturbance from traffic when wet. 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 gravel surfacing or haul roads, use gravel working pads and employ other methods to reduce ground disturbance. The contractor is responsible for protecting the subgrade during construction.

During wet weather, some of the exposed soils could become muddy and unstable. If so affected, we recommend that:

- The ground surface in and around the work area should 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. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work areas.
- 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.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practicable.

- When on-site soils are wet of optimum, they are easily disturbed and will not provide adequate support for construction traffic nor for the proposed development. The use of granular haul roads and staging areas will be necessary to support heavy construction traffic. Generally, a 12- to 16-inch-thick mat of Imported Select Structural Fill should be sufficient for light staging areas and activities but is not expected to be adequate to support repeated heavy equipment or truck traffic. The thickness of the Imported Select Structural Fill for haul roads and areas with repeated heavy construction traffic should be increased to between 18 and 24 inches. The actual thickness of haul roads and staging areas should be determined at the time of construction and based on the contractor's approach to site development, and the amount and type of construction traffic.
- The aggregate base thicknesses described in the "Pavement Recommendations" sections of this report are intended to support post-construction design traffic loads. The design base rock thicknesses will likely not support repeated heavy construction traffic during site construction or during pavement construction. A thicker rock section, as described above for haul roads, will likely be required to support construction traffic.

5.7. Dewatering

As discussed in the "Groundwater" section of this report, groundwater was encountered at a depth of between 14.5 and 19.5 feet bgs. We do not expect groundwater to be a major factor during shallow excavations and earthwork. Excavations that extend into saturated/wet soils or excavations that extend into perched groundwater should be dewatered. Sump pumps are expected to adequately address groundwater encountered in shallow excavations. In addition to groundwater seepage, 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.

5.8. Permanent Slopes

Permanent cut and fill slopes, where incorporated into the grading plan, should not exceed 2H:1V (horizontal to vertical). The slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

5.9. Temporary Excavations

Stability of temporary excavation slopes is a function of many factors, including soil type, soil density and consistency, slope inclination, slope height, the presence of groundwater, and duration of exposure. The likelihood of slope failure increases as the cut is deepened and as duration of exposure increases.

Temporary slope safety should remain the responsibility of the contractor, who is continually present at the site and is able to monitor performance of the excavation and modify construction practices and shoring methods to reflect varying conditions. All excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and State regulations.

Regardless of inclination, temporary slopes should be protected from surface runoff of storm water. This can typically be accomplished using berms or swales located along the top of the slope, and by placing plastic tarpaulins over the slope.

5.10. Structural Fill and Backfill

Soil conditions beneath foundations, pavements, and/or any other areas intended to support structures or within the influence zone of structures should be considered structural. Fill intended for use in structural areas should meet the criteria for structural fill presented below. 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 (3-inch-maximum particle size in structure footprints) 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.

In general, earthwork for fill material and compaction should meet the requirements outlined in the 2021 Oregon Standard Specifications for Construction (OSSC), section 00330 (Earthwork). CGS should review material submissions during construction.

5.10.1. Reuse of On-Site Soils

As described in the "Subsurface Conditions" section, on-site upper soil consists of silt with varying amounts of sand. It is likely that the on-site soil will not be suitable for use as structural fill when wet. If desired, an experienced geotechnical engineer from CGS can determine the suitability of on-site soil encountered during earthwork activities for reuse as structural fill.

5.10.2. Imported Granular Material

Select imported granular material may be used as structural fill. The imported material 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 (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 75 percent fractured particles according to AASHTO TP-61.

5.10.3. 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 backfill, Imported Select Structural Fill may be used as described above.

5.10.4. Fill Placement and 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. 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% of Optimum Moisture		
	0 to 2 Feet Below Subgrade	> 2 Feet Below Subgrade	Pipe Zone
On site Fine-grained (non-expansive)	92	92	-----
Imported Granular, maximum particle size < 1¼ inch	95	95	-----
Imported Granular, maximum particle size 1¼ inch to 6 inches (3-inch-maximum under structures)	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 over compaction 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 or a jumping jack.

Structural fill should be compacted at moisture contents that are within 3 percent of the optimum moisture content as determined by ASTM International (ASTM) 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.

A representative from CGS should evaluate the compaction of every two vertical feet (or less) and 500 cubic yards of fill material placed. Compaction should be evaluated by compaction testing unless other methods are proposed for oversized materials and are approved by CGS during construction. These other methods typically involve procedural placement and compaction specifications together with verification requirements such as proof-rolling.

5.11. Pavement Section Materials

The contractor should provide a submittal for each geotechnical construction material prior to the start of construction of pavement sections. Each submittal should include test information necessary to evaluate how the material's properties comply with the properties that were recommended or specified for the project. The geotechnical engineer and other appropriate members of the design team should review each submittal.

5.11.1. Aggregate Base

Imported granular material used as an aggregate base beneath pavement should be clean, crushed rock or crushed gravel and sand that is well graded. The aggregate base should meet the gradation defined in OSSC section 00640 (Aggregate Base and Shoulders), with the exception that the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, a maximum particle size of 1½ inches and have a minimum of 75 percent fractured particles according to AASHTO TP-61. The base aggregate should be compacted based on the ODOT specification.

5.11.2. Asphalt Concrete Pavement

The AC should be Level 2, ½-inch, dense ACP according to OSSC section 00744 (Asphalt Concrete Pavement). The minimum lift thickness for ½-inch ACP is 2.0 inches, and the maximum lift thickness should be 3.5 inches. Asphalt binder should be performance graded and conform to PG 64-22.

5.11.3. Stabilization Aggregate

Stabilization aggregate should consist of quarry-run rock, crushed rock, or crushed gravel and sand and should meet the requirements as described in OSSC Section 00330.14 (Selected Granular Backfill) and OSSC 00330.15 (Selected Stone Backfill) with a maximum particle size of 6 inches, less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve and have a minimum of 75 percent fractured particles according to AASHTO TP-61. The material should be free of organic matter and other deleterious materials. Stabilization aggregate should be placed over a geotextile fabric in one lift and compacted to a firm condition.

5.11.4. Subgrade Geotextile

The subgrade geotextile should conform to OSSC section 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6-inches is required over geotextiles.

6.0 STRUCTURAL DESIGN RECOMMENDATIONS

6.1. Foundation Support Recommendations

The proposed air-supported fabric structure, metal storage unit, restroom, and similar lightly loaded structures can be satisfactorily founded on continuous wall or isolated column footings supported on firm native soils or on structural fill placed over firm native soils. Exterior footings should be established at least 18 inches below the lowest adjacent grade. The recommended minimum footing depth is greater than the anticipated frost depth. Interior footings can be found a minimum of 12 inches below the top of the first-floor slab. Isolated columns and continuous wall footings should have minimum widths of 24 and 18 inches, respectively.

6.1.1. Foundation Subgrade Preparation

The subgrade beneath proposed structural elements should be prepared as described below and in the "Earthworks" section of this report. We recommend loose or disturbed soils resulting from foundation excavation 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 and concrete. A thin gravel layer consisting of imported granular material can be placed at the base of foundation excavations to help protect the subgrade from weather and light foot traffic. The layer thickness for the gravel layer should be determined at the time of construction but is typically 3 to 4 inches. The gravel layer should be compacted to a firm and well-keyed state.

We recommend CGS observe all foundation subgrades 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 our explorations.

6.1.2. Isolated Spread Footings

We recommend conventional footings for the proposed structures be proportioned using a maximum allowable bearing pressure of 2,000 psf if supported on firm native soils or on structural fill placed over firm native soils. This bearing pressure applies to the total of dead and long-term live loads and may be increased by one-third 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.

6.1.3. Foundation Settlement

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

6.1.4. Lateral Resistance

Lateral loads can be resisted by a combination of friction between the footing and the supporting soil, and by the passive lateral resistance of the soil surrounding the embedded portions of the footings. A coefficient of friction between the concrete and soil of 0.35 and a passive lateral resistance corresponding to an equivalent fluid density of 200 pcf may be used for design. These values are appropriate for foundation elements that are poured directly against the native soils or surrounded by compacted structural fill. 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 do not include safety factors.

6.2. Drainage Considerations

We recommend the ground surface be sloped away from the buildings at least 5 percent for a minimum distance of 10 feet measured perpendicular to the face of the wall or use of alternative drainage provisions in accordance with Section 1804.4 of the 2022 Oregon Structural Specialty Code. 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.

Due to the fine-grained material present at the site and as a matter of good construction practice, consideration should be given to installing perimeter footing drains. Footing drains should be installed at the base of exterior building footings where interior spaces should be protected from inflowing water from surrounding soils. 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 equivalent) 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.

6.3. Floor Slabs

Satisfactory subgrade support for floor slabs can be obtained, provided the floor slab subgrade is described in the "Earthwork Recommendations" section of this report. Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Subgrade support for concrete slabs can be obtained from the firm native soils or on structural fill placed over firm native soils.

For moisture sensitive areas, we recommend that on-grade slabs be underlain by a minimum 6-inch-thickness of Aggregate Base acting as a capillary break material to reduce the potential for moisture migration into the slab. The capillary break material should be placed as recommended in the "Fill Placement and Compaction" section of this report.

If dry on-grade slabs are required, for example at interior spaces where adhesives are used to anchor carpet or tile to the slab, a waterproof liner may be placed as a vapor barrier below the slab. The vapor barrier 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 the effect of the vapor barrier on concrete slab curing. Load-bearing concrete slabs should be designed assuming a modulus of subgrade reaction (k) of 150 psi per inch. We estimate that concrete slabs constructed as recommended will settle less than ½ inch. Floor slab subgrades should be evaluated according to the "Subgrade Evaluation" section of this report.

6.4. Seismic Design

Parameters provided on Table 3 are based on the conditions encountered during our subsurface exploration program and the procedure and requirements outlined in the 2022 Oregon Structural Specialty Code which is based on the 2021 IBC. Per American Society of Civil Engineers (ASCE) 7-16 Section 11.4.8, a site-specific response analysis is required for Site Class F sites, and a ground motion hazard analysis or site-specific response analysis is required to determine the design ground motions for structures on Site Class D and E sites with S_1 greater than or equal to 0.2g.

For this project, the site is classified as site class D; therefore, the provisions of 11.4.8 are applicable. Alternatively, the parameters listed in Table 4 may be used to determine the design ground motions if the exceptions provided in ASCE 7-16 Supplement 3 are met. The applicable exceptions for the project site listed in ASCE 7-16 Supplement 3 are provided below for reference. If it is desirable to avoid these exceptions, a ground motion hazard analysis would need to be completed to determine the design seismic parameters for the site. The seismic design parameters presented in Table 4 assume the exception above will be taken, and the indicated parameters have been increased accordingly.

<u>From ASCE 7-16 Supplement 3</u>

Exception: A ground motion hazard analysis not required where the value of the parameter S_{M1} determined by Eq. (11.4-2) is increased by 50% for all applications of S_{M1} in this standard. The resulting value of the parameter S_{D1} determined by Eq. (11.4-4) shall be used for all applications of S_{D1} in this standard.

TABLE 4. MAPPED 2021 IBC SEISMIC DESIGN PARAMETERS

Parameter	Recommended Value[1, ,2,4]
Site Class	D
Mapped Spectral Response Acceleration at Short Period (S_S)	0.897 g
Mapped Spectral Response Acceleration at 1 Second Period (S_1)	0.414 g
Site Modified Peak Ground Acceleration (PGA_M)	0.487 g
Site Amplification Factor at 0.2 second period (F_a)	1.141
Site Amplification Factor at 1.0 second period (F_v)	1.886
Design Spectral Acceleration at 0.2 second period (S_{DS})	0.682 g
Design Spectral Acceleration at 1.0 second period (S_{D1})	0.781 g[3]

Notes:

[1] Parameters developed based on Latitude 45.519417° and Longitude -122.836976 ° using the ASCE 7 Hazard online tool.

[2] These values are only valid if the structural engineer utilizes Exception 1 of ASCE 7-16 Supplement 3 Exception 1.

[3] Increased by a factor of 1.5 per ASCE 7-16 Supplement 3 Exception 1.

[4] Only applicable to structures with a fundamental period of vibration less than 0.5 seconds

7.0 CONVENTIONAL RETAINING WALLS

We understand that conventional retaining walls will be constructed around the perimeter of the multipurpose sports court. The recommendations presented below may be used for the design of conventional retaining walls.

7.1. Design Parameters

Retaining structures free to rotate slightly around the base should be designed for active earth pressures using an equivalent fluid unit weight (efp) of 40 pcf when the ground surface extends level behind the wall equal to a distance of at least twice the height of the wall, and 70 pcf for an inclined slope of 2H:1V above the wall. For lesser slopes between flat and 2H:1V, the efp can be linearly interpolated between the recommended values. The efp value is based on the following assumptions.

- The walls will not be restrained against rotation when the backfill is placed.
- Walls are 8 feet or less in total wall support height.
- The backfill within 2 feet of the wall consists of free-draining granular materials.
- Grades above the top of the walls are no steeper than a 2H:1V slope.
- Total wall heights are determined based on a level front slope from the base of the wall.
- Hydrostatic pressures do not develop, and drainage will be provided behind the wall.

When the ground surface extends level behind the wall equal to a distance of at least twice the height of the wall, seismically induced lateral forces on retaining walls can be calculated using a dynamic force equal to

10.5H psf, where H is the wall height. This seismic force should be applied with the centroid located at 0.6H from the wall base. These values assume that the wall is vertical and unrestrained and the backfill behind the wall is horizontal.

For site retaining walls, seismic lateral earth pressures should be computed as a part of retaining wall design using the Mononobe-Okabe equation or another method appropriate to the selected wall system.

Retaining walls, including foundation walls that are restrained against rotation during backfilling, should be designed for an at-rest equivalent fluid unit weight of 58 pcf when the ground surface extends level behind the wall equal to a distance of at least twice the height of the wall, and 85 pcf for an inclined slope of 2H:1V above the wall. For lesser slopes between flat and 2H:1V, the efp can be linearly interpolated between the recommended values.

Surcharge loads applied closer than one-half of the wall height should be considered as uniformly distributed horizontal pressures equal to one-third of the distributed vertical surcharge pressure. Footings for retaining walls should be designed as recommended for shallow foundations. Backfill should be placed and compacted as recommended for structural fill. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions. We recommend that CGS be retained to review the retaining wall design to confirm that it meets the requirements in our report. The retaining wall designer should perform global stability analysis of the proposed wall.

7.2. Retaining Wall Drainage Considerations

All structural retaining walls should be backfilled with an imported, free-draining granular material with no more than 5 percent passing the No. 200 sieve. A layer of compacted aggregate that is a minimum of 2-foot-wide should be placed behind all retaining walls to allow for proper drainage and placed utilizing the compaction recommendations described in this report. Only light-weight compaction equipment should be used immediately behind retaining walls, so that compactive effort does not damage the wall.

At the base of retaining walls and continuous with the wall backfill aggregate, a wall subdrain should be installed to divert water from the retaining the structures. The wall subdrain should consist of a 3- or 4-inch-diameter, perforated, gravity drainpipe (ADS Highway Grade or better) enveloped in at least 4 cubic feet per lineal foot of clean drain rock. The drain rock should be wrapped within geotextile filter fabric (Mirafi 140N or similar) with a minimum 6-inch overlap at joints to prevent fines from washing into the drain rock.

8.0 LIGHT POLE FOUNDATION RECOMMENDATION

Selection of light poles and lighting design for the project had not been completed at the time this report was prepared. Pole foundations for stadium-type court lighting are typically 24- to 36-inch diameter drilled shaft type foundations to accommodate embedment of a precast base for the light structure. Shaft depth to support the lighting structure is determined based on reaction loads at the pole base and the vertical and lateral support provided by on-site soil conditions.

Our borings indicate that subsurface soils at the site consist of stiff to medium-stiff silt. Geotechnical parameters for shaft foundations are summarized in Table 5.

TABLE 5. GEOTECHNICAL PARAMETERS FOR LIGHT POLE FOUNDATION DESIGN

Depth (feet bgs)	General Soil Type	Allowable End Bearing Pressure (psf)[1]	Allowable Lateral Soil Bearing Pressure[2] (psf/ft)	Allowable Skin Friction[3] (psf)
1 – 21.5	Silt (ML)	2,000	200	250

Notes:

[1] Per foot of depth below adjacent grade. Assumes that adjacent grade is flat for a minimum of 10 times the shaft diameter away from edge of shaft in all directions.

[2] Per foot of depth below adjacent grade. Assumes that adjacent grade is flat for a minimum of 10 times the shaft diameter away from edge of shaft in all directions.

[3] Based on clean contact between concrete or grout and surrounding soil in a direction parallel to the direction of load.

The top 1.5 feet of soil should be neglected when applying skin friction or allowable lateral soil bearing pressure unless the adjacent area is covered with pavement or slab-on-grade.

Shaft foundation typically include design for a performance specification to limit lateral deflection at the pole base per manufacturer's requirements, or to resist uplift and may require an extended depth of embedment beyond depth required to support pole base loads.

9.0 PAVEMENT DESIGN RECOMMENDATIONS FOR SPORTS COURT AND FIRE LANE

We understand that the proposed multi-sport court surface will likely consist of an acrylic surfacing placed on asphalt concrete (AC) over aggregate base. Our recommended pavement profile for the new sport court and fire truck access lane is based on an average resilient modulus value of 4,500 psi based on DCP test results, our analysis, and minimum industry standards.

9.1. AC over Aggregate Base Section

Our pavement structure recommendations are based on new AC over new aggregate base placed on firm medium-stiff to stiff subgrade prepared in accordance with "Earthworks" section of this report. These thicknesses are intended to be the minimum acceptable for construction completed during an extended period of dry weather. Depending on the time of construction, the silt subgrade will be easily disturbed by construction equipment. Soft subgrade areas and any areas disturbed by equipment traffic should be removed and replaced with stabilization aggregate or aggregate base.

In the event of wet weather construction, we recommend that the aggregate base thickness be increased by 6 inches. Pavement materials should follow the recommendations in the "Pavement Section Materials" portion of this report.

Recommended Sports Court and Fire Lane Pavement Structure (3 inches of AC over 8 inches of aggregate base)

- 3.0-inch-thick, Level 2, ½-inch, dense ACP wearing course (two lifts)
- 8.0-inch-thick aggregate base
- Stabilization aggregate, if required
- Subgrade geotextile

The above recommendations are based on our understanding of pavement loads which consist of occasional light trucks and infrequent heavier trucks which may consist of 75,000-pound fire trucks in the event of an emergency. Our design recommendations are based on a 20-year pavement design life.

9.2. Drainage Considerations for Sport Court

Site drainage should include sport court drainage, surface runoff collection, and conveyance to a properly designed and permitted storm water drainage facility. Pavement surfaces and open space areas on the sport court exterior should be sloped away from the court to suitable discharge points.

We recommend that a perimeter subdrain extending a minimum of 12 to 18 inches below the base of the pavement section be installed along the exterior of the court to drain water away from the pavement section. Our recommended subdrain is a 3- or 4-inch-diameter, perforated, gravity drainpipe (ADS Highway Grade or better) enveloped in at least 2 cubic feet per lineal foot of clean drain rock (2-inch to ¼-inch diameter). The drain rock should be wrapped with geotextile filter fabric (such as Mirafi 140N or equivalent) with a minimum 1-foot overlap at joints to prevent fines from washing into the drain. The subdrain should discharge by means of gravity flow to an approved non-perforated drainpipe conveyance outlet or storm drain facility.

10.0 DESIGN REVIEW AND CONSTRUCTION SERVICES

Recommendations provided in this report are based on the assumptions and preliminary 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, CGS 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.

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 CGS 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 OF REPORT

We have prepared this report for the exclusive use of the Tualatin Hills Park & Recreation District, AKS Engineering & Forestry LLC, and members of the design team, for this specific project only. The report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, CGS should be notified for review of the recommendations of this report, and revision of such if necessary.

We recommend that the CGS be retained as the geotechnical engineer of record (GER) to observe construction at the site to confirm that subsurface conditions are consistent with site subsurface determinations described in this report, and to confirm that the intent of project plans and specifications relating to earthwork, pavement and foundation construction are being met. If geotechnical construction observation services are provided by another entity, they should be provided this Geotechnical Report, and confirm in a letter stamped by a licensed professional engineer from that entity that they have reviewed the report, understand and agree with the recommendations or provide alternate recommendations, and confirm that they are taking over as GER.

Within the limitations of scope, schedule and budget, the analysis, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology in this area at the time the report was prepared. No warranty, express or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

Within the limitations of scope, schedule, and budget, our services were executed in accordance with generally accepted practices in this area at the time this report was prepared. No warranty, express or implied, should be understood.

12.0 REFERENCES

American Association of State Highway and Transportation Officials (AASHTO). 1993. Guide for Design of Pavement Structures.

Madin, I.P., 1990, Earthquake Hazard Geology Maps of the Portland Metropolitan Area, Oregon; Oregon Department of Geology and Mineral Industries, Open File Report O-90-02, map scale 1:24,000.

Oregon Department of Transportation (ODOT). 2024. Oregon Standard Specifications for Construction.

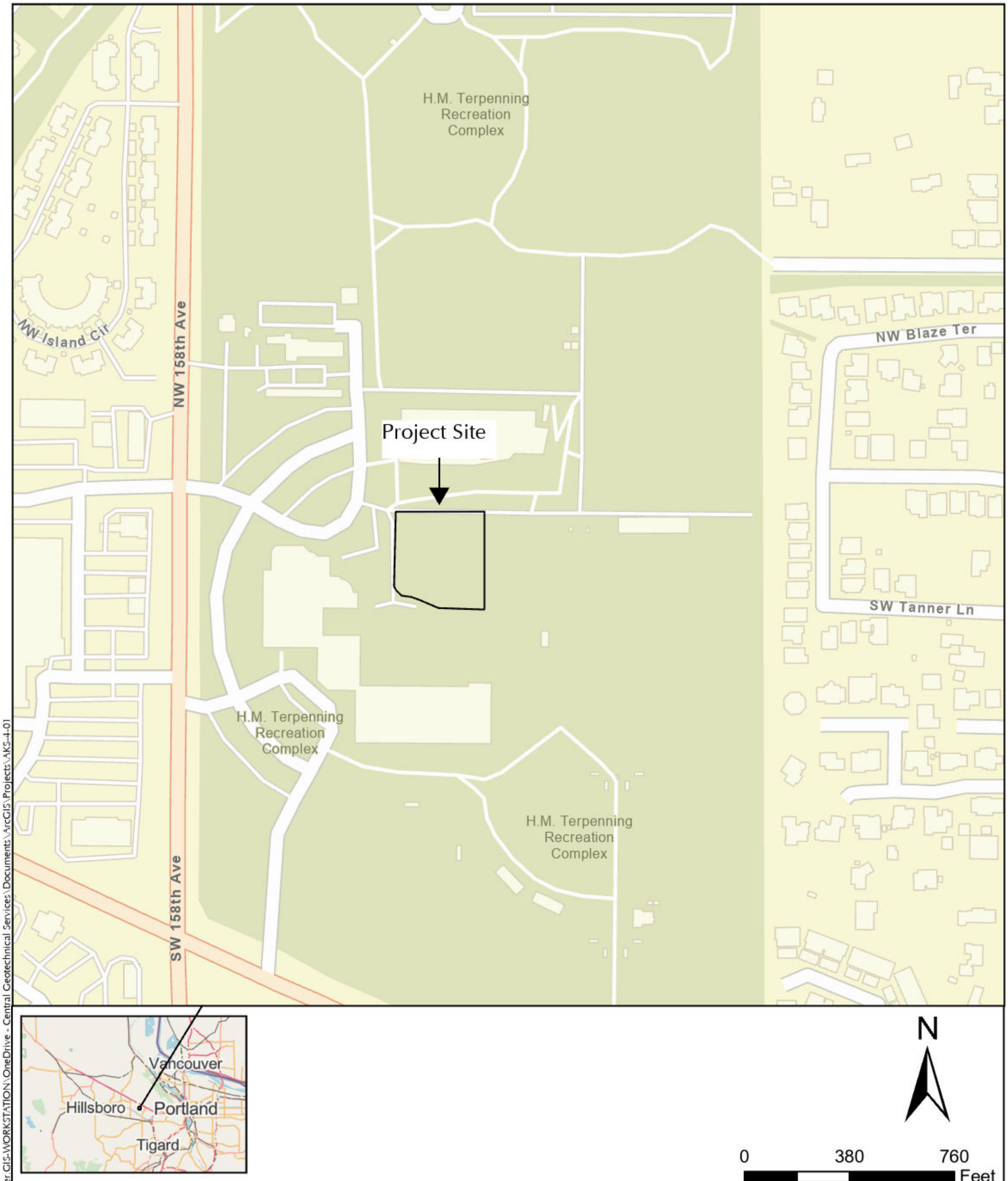
Oregon Department of Transportation (ODOT). 2019. “ODOT Pavement Design Guide.” Pavement Services Unit. January.

13.0 SIGNATURES

Thank you very much for the opportunity to work with you. If you feel obliged, we welcome referrals from our previous clients and would enjoy the opportunity to work with others in your professional and personal networks.

Central Geotechnical Services, LLC

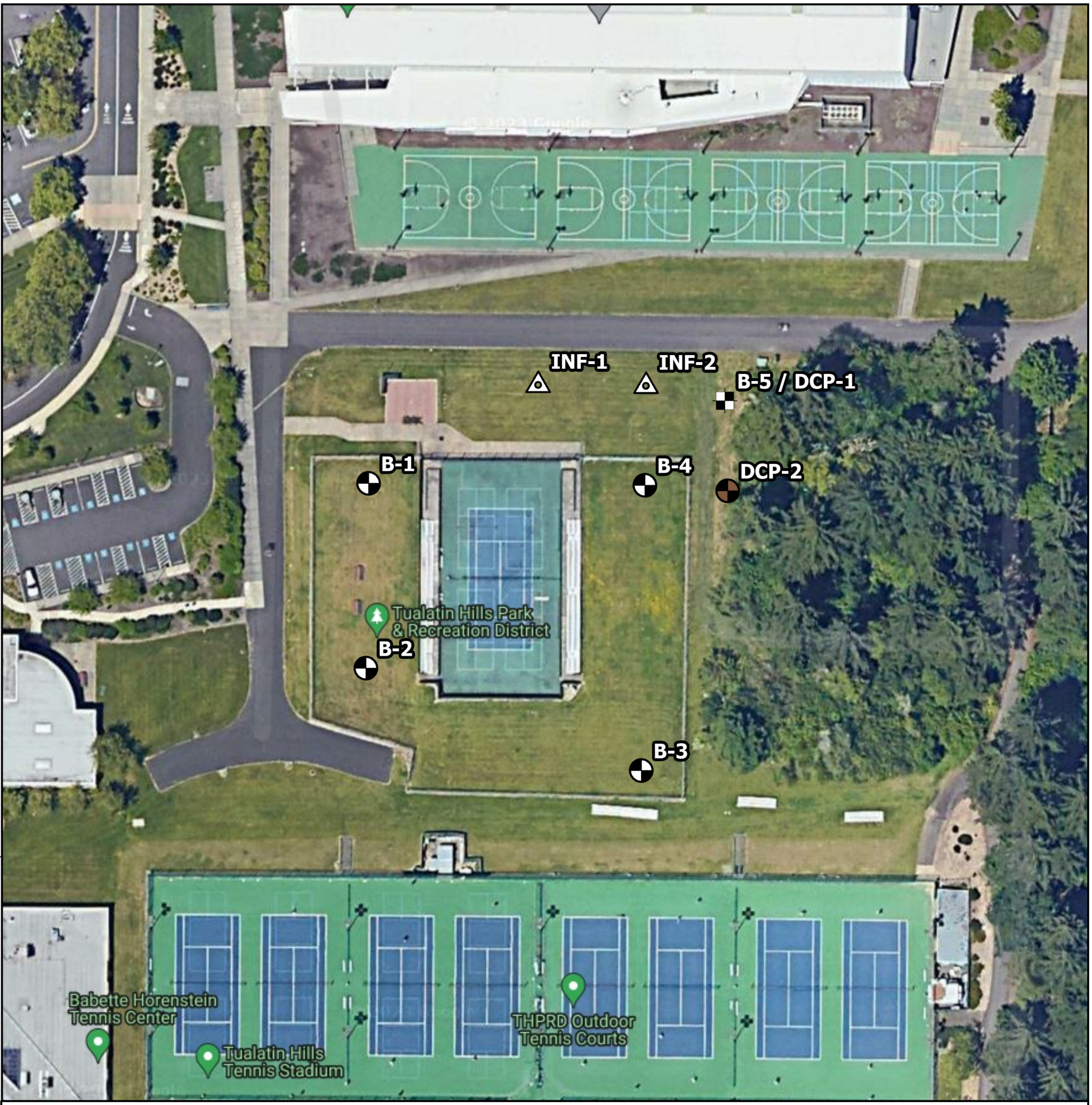
Krey D. Younger, PE, GE	Zane Rogers, PE
Principal Engineer	Project Engineer



Map data © OpenStreetMap contributors, Microsoft, Facebook, Inc. and its affiliates, Esri Community Maps contributors, Map layer by Esri, Esri Community Maps Contributors, Oregon Metro, Oregon State Parks, State of Oregon GEO, © OpenStreetMap, Microsoft, Esri, HERE, Garmin, SafeGraph, GeoTechnologies, Inc, METI/NASA, USGS, Bureau of Land Management, EPA, NPS, US Census Bureau, USDA

AKS-4-01 HMT Complex Outdoor Court Expansion
Project Vicinity
CENTRAL Geotechnical Services, LLC — Figure-1

Default Folder: C:\Users\cgsuser GIS-WORKSTATION\OneDrive - Central Geotechnical Services\Documents\ArcGIS\Projects\AKS-4-01



Legend

Exploration Designation and Approximate Location

- Boring
- Boring / DCP
- DCP
- Infiltration

N

0 150 300 Feet

Sources:Google

AKS-4-01 HMT Complex Outdoor Court Expansion	
Site Plan	
CENTRAL Geotechnical Services, LLC	Figure-2

Default Folder: C:\Users\cgsuser.GIS-WORKSTATION\OneDrive - Central Geotechnical Services\Documents\ArcGIS\Projects\AKS-4-01

CENTRAL
GEOTECHNICAL SERVICES

APPENDIX A: Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Soil and groundwater conditions at the proposed Howard M. Terpenning (HMT) Recreation Complex outdoor court expansion project were explored on October 27 and 28 of 2023, by completing five drilled borings (B-1 and B-5) and two infiltration test borings (INF-1 and INF-2). Explorations were extended to depths between 5 and 21.5 feet below the ground surface (bgs) at the approximate locations shown on Figure 2.

The drilled borings were advanced using a trailer-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 detailed logs of subsurface exploration, 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. Samplers were driven into the soil using a rope and cathead 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 testing (SPT) N-value.

Field explorations were completed by a qualified staff from our office who maintained detailed logs of subsurface explorations, visually classified the soil encountered and obtained representative soil samples from the borings. Representative soil samples were obtained from each exploration as noted in the exploration logs in Appendix A.

Recovered soil samples from exploratory borings were visually classified in the field in general accordance with ASTM D 2488 and the classification chart listed in Key to Exploration Logs, Figure A-1. Logs of the borings are presented in Figures A-2 through A-7. The logs are based on interpretation of the field and laboratory data and indicate the depth at which subsurface materials or their characteristics change, although these changes might actually be gradual.

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. A discussion of laboratory tests performed is provided below.

Moisture Content

Moisture content tests were completed in general accordance with ASTM D 2216 for representative samples obtained from the explorations. The results of these tests are presented on the exploration logs in Appendix A at the depths at which the samples were obtained.

Percent Passing U.S. No. 200 Sieve

Selected samples were "washed" through the U.S. No. 200 mesh sieve to estimate the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve. These tests were conducted to verify field descriptions and to estimate the fines content for analysis purposes. The tests were conducted in accordance with ASTM D 1140, and the results are shown on the exploration logs in Appendix A at the respective sample depths.

CENTRAL GEOTECHNICAL SERVICES, LLC

SOIL CLASSIFICATION

MAJOR DIVISIONS			SYMBOLS LETTER	SYMBOLS GRAPH	TYPICAL DESCRIPTIONS
COARSE GRAINED (MORE THAN 50% RETAINED BY NO. 200 SIEVE)	GRAVEL	CLEAN GRAVELS	GW		WELL-GRADED GRAVELS AND GRAVEL/SAND MIXTURES, LITTLE OR NO FINES
			GP		POORLY-GRADED GRAVELS, GRAVEL/SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES	GM		SILTY GRAVELS, GRAVEL/SAND/SILT MIXTURES
			GC		CLAYEY GRAVELS, GRAVEL/SAND/CLAY MIXTURES
	SAND	CLEAN SANDS	SW		WELL-GRADED SAND AND GRAVELLY SANDS, LITTLE OR NO FINES
			SP		POORLY-GRADED SAND AND GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES	SM		SILTY SANDS, SAND/SILT MIXTURES
			SC		CLAYEY SANDS, SAND/CLAY MIXTURES
FINE GRAINED (MORE THAN 50% PASSING BY NO. 200 SIEVE)	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	ML		INORGANIC SILTS, SILT WITH SLIGHT PLASTICITY
			CL		INORGANIC CLAY, CLAY WITH LOW TO MEDIUM PLASTICITY
			OL		ORGANIC SILTS, ORGANIC SILTY CLAYS WITH LOW PLASTICITY
		LIQUID LIMIT MORE THAN 50	MH		INORGANIC SILTS, SILTS WITH CLAY
			CH		INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH		ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			TS		TOPSOIL, PEAT, HUMUS, MULCH AND OTHER HIGH ORGANIC SOILS

GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits
CBR	California Bearing Ratio
CON	Consolidation
DD	Dry Density
DS	Direct Shear
HYD	Hydrometer Gradation
LL	Liquid Limit
PL	Plastic Limit
PI	Plasticity Index
MC	Moisture Content
MD	Moisture-Density
NP	Non-Plastic
OC	Organic Content
P	Pushed Sample
PP	Pocket Penetrometer
Passing No.200	Percent Passing U.S. Std. No.200 Sieve
RES	Resilient Modulus
SIEV	Sieve Gradation
TOR	Torvane
UC	Unconfined Compressive Strength
VS	Vane Shear

CONTACT TYPES

(solid line)	Distinct contact between soil strata (approximate location)
(dashed line)	Approximate contact between soil strata

ADDITIONAL MATERIALS

AC		ASPHALT CONCRETE
CC		CEMENT CONCRETE
CR		CRUSHED ROCK
SOD		SOD/FOREST DUFF
FILL		FILL

WATER LEVELS

	Water Level at Time of Drilling, or as labeled
	Water Level at End of Drilling, or as labeled
	Static Water Level, or as labeled

SOIL CHARACTERISTICS

GRADATION		CAVING	
WELL-GRADED	FULL RANGE OF GRAIN SIZES	NO	WALL STANDING VERTICAL
POORLY GRADED	LIMITED RANGE OF GRAIN SIZES	MINOR	ISOLATED SPALLING
UNIFORMLY GRADED	PREDOMINANTLY ONE GRAIN SIZE	MODERATE	COMMON SPALLING
GAP GRADED	GAPS WITHIN RANGE OF GRAIN SIZES	SEVERE	WILL NOT STAND VERTICAL

SYMBOL	SAMPLER DESCRIPTIONS	SYMBOL	SAMPLER DESCRIPTIONS
	Location of auger cuttings sample		Location of sample collected in general accordance with ASTM D1586 using Standard Penetration Test (SPT) with recovery (SS)
	Location of bulk or grab sample (GS)		Location of sample collected using the thin-wall Shelby tube or Geoprobe sample in general accordance with ASTM D1587 with recovery (SH)
	Location of rock coring interval (RC)		Location of sample collected in general accordance with ASTM D2573 using the field Vane Shear test in saturated fine-grained soils with recovery
	No Recovery		Location of sample collected using Dames & Moore sampler or pushed with recovery

CENTRAL Geotechnical Services
Central Geotechnical Services
10240 SW Nimbus Ave, Suite L6
Portland, OR 97223
Telephone: (503) 616-9419

Project No:
AKS-4-01

BORING LOG B-1

Project: THPRD - HMT Pickleball Court
Location: 15707 SW Walker Road, Beaverton, OR 97006
Client: AKS Engineering and Forestry

Date Started: 10/27/23
Date Completed: 10/27/23

Approximate Ground Elevation: 223ft
Groundwater first observed: ---
Groundwater at end of drilling: 14.60 ft / Elev 208.40 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/ REMARKS
0–1		Stiff SILT (ML), brown to gray, moist (8-inch topsoil layer)					
3			SPT 1	14	27	6-7-8	
6			SPT 2	16		5-5-6	
8		Becomes medium-stiff at 7.5-feet	SPT 3	14	34	2-3-4	
11			SPT 4	16	37	3-2-3	
15		Moisture increases to wet at 14.5-feet bgs			▼		
16			SPT 5	16	41	1-3-5	
21			SPT 6	16	35	2-3-5	
21.5							

Boring terminated at 21.5-feet bgs
Groundwater was measured at 14.6-feet after during

Operator: Western States Soil Conservation Inc.
Drilling Method: 4 3/8" Solid Stem Auger
Equipment: Trailer Mounted Drill Rig

Logged By: Troy Howard
Checked By: Jose Serrano

Approximate Location Coordinates:
Lat: Long:

Remarks:

Central Geotechnical Services
10240 SW Nimbus Ave, Suite L6
Portland, OR 97223
Telephone: (503) 616-9419

Project No:
AKS-4-01

BORING LOG B-2

Project: THPRD - HMT Pickleball Court
Location: 15707 SW Walker Road, Beaverton, OR 97006
Client: AKS Engineering and Forestry

Date Started: 10/27/23
Date Completed: 10/27/23

Approximate Ground Elevation: 223ft
Groundwater first observed: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/ REMARKS
0		Medium-stiff SILT (ML), brown, moist (9-inch topsoil layer)					
3			SPT 1	16		2-3-4	
5.5			SPT 2	16		2-3-4	
8			SPT 3	14		2-2-2	
10.5			SPT 4	16		3-2-4	
15	15.0						

Boring terminated at 15-feet bgs
Groundwater was not observed

Operator: Western States Soil Conservation Inc.
Drilling Method: 4 3/8" Solid Stem Auger
Equipment: Trailer Mounted Drill Rig

Logged By: Troy Howard
Checked By: Jose Serrano

Approximate Location Coordinates:
Lat: Long:

Remarks:

Central Geotechnical Services
10240 SW Nimbus Ave, Suite L6
Portland, OR 97223
Telephone: (503) 616-9419

Project No: AKS-4-01

BORING LOG B-3

Project: THPRD - HMT Pickleball Court
Location: 15707 SW Walker Road, Beaverton, OR 97006
Client: AKS Engineering and Forestry

Date Started: 10/27/23
Date Completed: 10/27/23

Approximate Ground Elevation: 223ft
Groundwater first observed: ---
Groundwater at end of drilling: 19.20 ft / Elev 203.80 ft

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/ REMARKS
0–21.5	Stiff SILT (ML), light brown to brown, dry to moist (9-inch topsoil layer)					
3		SPT 1	14		4-5-6	
5	Becomes medium-stiff at 5-feet					
6		SPT 2	14		2-3-4	
8		SPT 3	16		2-1-3	
11		SPT 4	14		2-2-3	
16		SPT 5	16		1-2-3	
19.2	Moisture increases to wet below 19.2-feet			▼		
21	Becomes stiff	SPT 6	16		3-4-5	
21.5						

Boring terminated at 21.5-feet bgs
Groundwater was measured at 19.2-feet at termination

Operator: Western States Soil Conservation Inc.
Drilling Method: 4 3/8" Solid Stem Auger
Equipment: Trailer Mounted Drill Rig

Logged By: Troy Howard
Checked By: Jose Serrano

Approximate Location Coordinates:
Lat: Long:

Remarks:

CENTRAL Geotechnical Services
Central Geotechnical Services
10240 SW Nimbus Ave, Suite L6
Portland, OR 97223
Telephone: (503) 616-9419

Project No:
AKS-4-01

BORING LOG B-4

Project: THPRD - HMT Pickleball Court
Location: 15707 SW Walker Road, Beaverton, OR 97006
Client: AKS Engineering and Forestry

Date Started: 10/28/23
Date Completed: 10/28/23

Approximate Ground Elevation: 228ft
Groundwater first observed: ---
Groundwater at end of drilling: 19.50 ft / Elev 208.50 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/ REMARKS
0–1		Medium-stiff to stiff SILT (ML), trace fine sand, brown, dry to moist (8-inch topsoil layer)					
3			SPT 1	14	20	4-4-4	
6			SPT 2	12		1-2-5	
8			SPT 3	14	35	2-3-4	
10–11		Becomes soft at 10-feet	SPT 4	16		1-2-2	
15–16			SPT 5	16	35	3-2-2	
19.5					▼		
20–21		Moisture increased to wet at 19.5-feet	SPT 6	16	32	2-1-1	
21.5							

Boring terminated at 21.5-feet bgs
Groundwater was measured at 19.5-feet after drilling

Operator: Western States Soil Conservation Inc.
Drilling Method: 4 3/8" Solid Stem Auger
Equipment: Trailer Mounted Drill Rig

Logged By: Troy Howard
Checked By: Jose Serrano

Approximate Location Coordinates:
Lat: Long:

Remarks:

CENTRAL GEOTECHNICAL SERVICES
Central Geotechnical Services
10240 SW Nimbus Ave, Suite L6
Portland, OR 97223
Telephone: (503) 616-9419

Project No: AKS-4-01

BORING LOG B-5

Project: THPRD - HMT Pickleball Court
Location: 15707 SW Walker Road, Beaverton, OR 97006
Client: AKS Engineering and Forestry

Date Started: 10/28/23
Date Completed: 10/28/23

Approximate Ground Elevation: 217ft
Groundwater first observed: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/ REMARKS
0		Stiff SILT (ML), brown, moist (8-inch topsoil layer)	SPT 1	12		3-4-6	
4			SPT 2	10		1-1-4	
5	5.0						

Boring terminated at 5-feet bgs
Groundwater was not observed during drilling

Operator: Western States Soil Conservation Inc.
Drilling Method: 4 3/8" Solid Stem Auger
Equipment: Trailer Mounted Drill Rig

Logged By: Troy Howard
Checked By: Jose Serrano

Approximate Location Coordinates:
Lat: Long:

Remarks:

Central Geotechnical Services
10240 SW Nimbus Ave, Suite L6
Portland, OR 97223
Telephone: (503) 616-9419

Project No:
AKS-4-01

INFILTRATION LOG INF-1

Project: THPRD - HMT Pickleball Court
Location: 15707 SW Walker Road, Beaverton, OR 97006
Client: AKS Engineering and Forestry

Date Started: 10/27/23
Date Completed: 10/27/23

Approximate Ground Elevation: 219ft
Groundwater first observed: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/ REMARKS
0–5.0		Stiff SILT (ML), brown, moist (7-inch topsoil layer)	SPT 1	12		3-3-9	
			SPT 2	14		5-7-8	Passing No. 200 Sieve: 98.0%
5.0							

Boring terminated at 5-feet bgs
Groundwater was not observed during drilling

Operator: Western States Soil Conservation Inc.
Drilling Method: 6" Hollow Stem Auger
Equipment: Trailer Mounted Drill Rig

Logged By: Troy Howard
Checked By: Jose Serrano

Approximate Location Coordinates:
Lat: Long:

Remarks:

CENTRAL Geotechnical Services
Central Geotechnical Services
10240 SW Nimbus Ave, Suite L6
Portland, OR 97223
Telephone: (503) 616-9419

Project No: AKS-4-01

INFILTRATION LOG INF-2

Project: THPRD - HMT Pickleball Court
Location: 15707 SW Walker Road, Beaverton, OR 97006
Client: AKS Engineering and Forestry

Date Started: 10/27/23
Date Completed: 10/27/23

Approximate Ground Elevation: 220ft
Groundwater first observed: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	BLOW COUNTS	LAB RESULTS/ REMARKS
0 – 8.0		Medium-stiff SILT (ML), brown, moist (8-inch topsoil layer)					
~7 – 8			SPT 1	14		1-3-3	Passing No. 200 Sieve: 98.0%

Boring terminated at 8-feet bgs
Groundwater was not observed during drilling

Operator: Western States Soil Conservation Inc.
Drilling Method: 6" Hollow Stem Auger
Equipment: Trailer Mounted Drill Rig

Logged By: Troy Howard
Checked By: Jose Serrano

Approximate Location Coordinates:
Lat: Long:

Remarks: