GEOTECHNICAL ENGINEERING REPORT - DRAFT

PETROVITSKY PARK SPLASH PAD
16400 SE PETROVITSKY ROAD
KING COUNTY (RENTON), WASHINGTON

ZGA Project No. 2781.01
May 15, 2024

Prepared for:

Otak and King County Parks

Prepared by:

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ZGA Project No. 2781.01
May 15, 2024

Otak, Inc.
11241 Willows Road Northeast, Suite 200
Redmond, Washington 98052

Attention: Mr. Keith Bates, ASLA

Subject: Geotechnical Engineering Report - **DRAFT**
Petrovitsky Park Splash Pad
16400 SE Petrovitsky Road
King County (Renton), Washington
Otak Project no. 21250.B00

Dear Keith:

In accordance with your request, Zipper Geo Associates, LLC (ZGA) has completed the subsurface exploration and geotechnical engineering evaluation for the proposed Petrovitsky Park Splash Pad project. This report presents the findings of the subsurface exploration and geotechnical recommendations for the project. Our work was completed in general accordance with the scope of services described in our *Subconsultant Agreement* (dated April 4, 2024). We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further assistance, please contact us.

Sincerely,

Zipper Geo Associates, LLC

DRAFT

Martin R. Cross, LG
Project Geologist

DRAFT

Robert A. Ross, PE
Managing Principal

Distribution: Addressee (1 pdf)

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1.0 INTRODUCTION

This report presents the surface and subsurface conditions encountered at the site and our geotechnical engineering recommendations for the proposed Petrovitsky Park Splash Pad. Our scope of services included reviewing readily available geologic data, a site reconnaissance, subsurface evaluation, laboratory testing, geotechnical engineering analysis, and preparation of this report. The project description, site conditions, and our geotechnical conclusions and recommendations are presented in the text of this report. Supporting data including detailed exploration logs and field exploration procedures and the results of laboratory testing are presented as appendices.

1.1 Site Description

Petrovitsky Park comprises one irregularly-shaped parcel (King County Parcel No. 362305-9007) in the Fairwood neighborhood of unincorporated King County, Washington. The site is located generally northeast of the SE Petrovitsky Road and Parkside Way SE intersection. The property encompasses approximately 88 acres and is bordered by forested land to the north, residential development to the east, SE Petrovitsky Road and residential development to the south, and Ridgewood Elementary School and residential developments to the west.

1.2 Project Understanding

We understand the proposed project will include construction of the new splash pad amenity. The improvements are planned for construction adjacent to the existing play areas and covered picnic shelter as shown on the Site and Exploration Plan, Figure 1. The splash pad will include a holding/recirculation tank located below the splash pad, a maintenance and chemical storage building to the south of the existing picnic shelter, low retaining walls around the pad, and a new asphalt-paved walkway. We understand that the excavation for the holding tank may be as deep as about 15 feet below grade. The storage building will be one story and likely of light metal frame and skin construction.

2.0 SURFACE CONDITIONS

The location of the proposed splash pad consists of open lawn that gently slopes downward at approximately 7 percent towards the north to northeast to an asphalt-paved walkway that runs east-west and above cut slope descending to the ball fields to the north. According to the survey of the *Petrovitsky Park Existing Conditions* (dated April 17, 2024) provided to us by Otak, ground surface elevations range from a high of about 579 feet at the south of the existing play area and picnic shelter to about 570 feet along the asphalt walk to the north. A stormwater management pond is located a short distance east of the picnic shelter. Review of historical aerial photographs suggests that the pond holds water most of the year.

3.0 SUBSURFACE CONDITIONS

3.1 General

In order to characterize subsurface soil and groundwater conditions at the project site, we reviewed readily available published surficial geologic mapping and advanced four (4) geotechnical borings on February 8, 2024. The following sections describe our understanding of subsurface soil and groundwater conditions based on the mapping and our borings.

3.2 Published Surficial Geologic Mapping

The publication *Surficial Geologic Map of the Maple Valley Quadrangle, King County, Washington* (USGS, Map MF-2287, 1995) indicates that the subject site and surrounding area have been mapped as containing glacial till deposits (Qgt). Glacial till is a glacially consolidated and heterogenous soil consisting of clay, silt, sand, gravel, cobbles, and boulders in varying amounts. Glacial till is colloquially termed "hardpan" in western Washington due to its compact and dense nature resulting from the deposit forming below the overburden of historical glaciers. As a result of the soil density and generally high fines content (the soil fraction passing the US No. 200 sieve), glacial till is generally characterized by a low permeability. Given the developed nature of the site, fill material is likely present as well.

3.3 Soil Conditions

The borings were completed at the locations of the proposed splash pad features approximately as shown on Figure 1. The borings were advanced to depths of approximately 7 to 12-¾ feet. Subsurface conditions disclosed by the borings were generally consistent with the published mapping. In general, each boring encountered a shallow fill horizon underlain by native glacial till that extended to the borings' termination depths.

Soils were visually described during recovery in general accordance with the *Explanation of Exploration Logs* provided in Appendix A. Detailed descriptive logs of the subsurface explorations and the procedures utilized in the subsurface exploration program are presented in Appendix A. Generalized descriptions of subsurface soil conditions observed in specific areas of the site are presented below. Please refer to the exploration logs in Appendix A for a more detailed description of the conditions encountered at the exploration locations. Stratification boundaries on the boring logs represent the approximate depth of changes in soil types, although the transition between materials may have been gradual. If variations become apparent during construction, it may be necessary to re-evaluate the recommendations of this report.

For purposes of describing soil conditions observed in the borings, and for reference in other sections of this report, soils with similar engineering characteristics were grouped together into Engineering Stratigraphic Units (ESUs). The following paragraphs provide our interpretation of ESUs disclosed by the borings. ESUs are ordered in a top-down stratigraphic sequence. Shallow surficial conditions such as topsoil and gravel surfacing are not described as separate ESUs below, and the reader is referred to the boring logs attached in Appendix A for information regarding shallow surficial conditions.

ESU-1 Fill: Soils interpreted to be fill were observed at each boring location extending from below the grass and topsoil to about 1-½ to 3 feet below existing grade. ESU-1 soils were observed to generally consist of loose to medium dense, dark brown to brown, silty sand and sandy silt with a varying gravel content and trace organics.

ESU-2 Weathered Glacial Till: We observed soils interpreted to be weathered glacial till at the locations of borings B-1, B-3, and B-4 below the fill, extending to about 4 to 5 feet below grade. ESU-2 soils generally consisted of medium dense to dense, brown to light gray with some oxidation mottling, silty sand with a varying gravel content. Given the relatively high fines content and density of these soils, unweathered till typically has a relatively low permeability.

ESU-3 Unweathered Glacial Till: We observed soils interpreted to be unweathered glacial till below the weathered glacial till (borings B-1, B-3, an B-4) or below the fill (B-2), extending to the termination of each boring between 7 to 12-¾ feet below grade. ESU-3 soils generally consisted of dense to very dense, light brown to gray, sand with a high gravel and silt content. Given the relatively high fines content and density of these soils, unweathered till typically has a relatively low permeability.

3.4 Groundwater Conditions

Perched groundwater was encountered within the ESU-2 (B-3) and ESU-3 (B-1, B-2) soils at approximately 2-½ to 7-½ feet below existing grade. We have interpreted that the perched groundwater encountered at the time of drilling develops due to the hydraulically restrictive nature of the glacial till. It should be noted that groundwater and soil moisture content conditions will likely vary seasonally and in response to precipitation events, land use, irrigation, and other factors. In general, seasonal high groundwater in western Washington occurs toward the end of the local wet season, typically around the end of May.

For discussion purposes, it should be noted that the subject site within the likely depth of excavations necessary for construction does not support a “groundwater table”. Rather, sandy horizons and zones within and above the nature glacial till will tend to wet to saturated during the wetter time of year. These areas of perched groundwater may vary considerably in depth and be laterally discontinuous.

3.5 Summary of Laboratory Testing

Laboratory testing was completed on select soil samples recovered from the borings. The results of moisture content testing are shown on the boring logs in Appendix A. The results of grain size analysis are presented in Appendix B.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 General

Based on our understanding of the proposed project, our review of the documents referenced herein, our site reconnaissance, subsurface explorations, and laboratory testing results, we have concluded that the proposed project appears feasible from the geotechnical perspective. Our general geotechnical considerations are summarized below.

- The site-characteristic medium dense to very dense glacial till soils are well suited for support of the proposed splash pad elements including slabs, building and wall foundations, and the below-grade storage tank.

- The low permeability of the unweathered glacial till is such that the excavation housing the below-grade storage tank may fill with water following construction (unless it is drained). We recommend constructing the tank with measures intended to counteract possible buoyant forces acting on the tank following construction.

- The glacial till soils have a high fines content. Consequently, they should be considered moisture-sensitive and scheduling grading and foundations construction for the dryer summer and early fall months will reduce, but may not eliminate, the likelihood of grading delays due to wet weather and the need for select import fill materials.

- The low permeability of the glacial till soils and the development of a seasonal shallow perched groundwater condition is such that stormwater infiltration at the splash pad location is not feasible from the geotechnical perspective, in our opinion.

The following sections present specific geotechnical recommendations. Our recommendations are based on the observed soil conditions at specific exploration locations. Differing soil conditions than those observed at the boring locations may become evident during construction. Our recommendations are further based on the assumption that earthwork for site grading, utilities, foundations, and slabs will be monitored by a ZGA representative.

4.2 Geologically Hazardous Areas

In general accordance with Chapter 21A.24 of the King County Code (KCC), we utilized existing site plans, site observations, County mapping, online mapping applications, and readily available geologic maps and publications to determine the presence of regulated geologically hazardous areas in the project vicinity. Our conclusions regarding geologic hazard critical areas are summarized below, with italics indicating KCC definitions and our responses following.

Erosion Hazard

Per KCC 21A.06.415, an erosion hazard is "*…an area underlain by soils that is subject to severe erosion when disturbed. These soils include, but are not limited to, those classified as having a severe to very severe erosion hazard according to the United States Department of Agriculture Soil Conservation Service, the 1990 Snoqualmie Pass Area Soil Survey, the 1973 King County Soils Survey or any subsequent revisions or addition* *by or to these sources such as any occurrence of River Wash ("Rh") or Coastal Beaches ("Cb") and any of the following when they occur on slopes inclined at fifteen percent or more:*

The Alderwood gravely sandy loam ("AgD");

The Alderwood and Kitsap soils ("AkF");

The Beausite gravely sandy loam ("BeD" and "BeF");

The Kitsap silt loam ("KpD");

The Ovall gravely loam ("OvD" and "OvF");

The Ragnar fine sandy loam ("RaD"); and

The Ragnar-Indianola Association ("RdE")".

The Natural Resource Conservation Service (formerly the Soil Conservation Service) has mapped the proposed splash pad location as mantled by the Alderwood Gravelly Sandy Loam, 0 – 8 percent slopes, (AgB). These soils do not meet the KCC criteria for an erosion hazard.

Steep Slope Hazard

The KCC characterizes regulated steep slope hazard areas as described below:

An area on a slope of forty percent inclination or more within a vertical elevation change of at least ten feet. For the purpose of this definition, a slope is delineated by establishing its toe and top and is measured by averaging the inclination over at least ten feet of vertical relief. Also for the purpose of this definition:

- *The "toe" of a slope means a distinct topographic break in slope that separates slopes inclined at less than forty percent from slopes inclined at forty percent or more. Where no distinct break exists, the "toe" of a slope is the lower most limit of the area where the ground surface drops ten feet or more vertically within a horizontal distance of twenty five feet; and*

- *The "top" of a slope is a distinct topographic break in slope that separates slopes inclined at less than forty percent from slopes inclined at forty percent or more. Where no distinct break exists, the "top" of a slope is the upper-most limit of the area where the ground surface drops ten feet or more vertically within a horizontal distance of twenty-five feet.*

Based on our review of mapped site topography and field measurements, the cut slope extending below and north-northwest of the splash pad location toward field 1 includes segments inclined at or exceeding

40 percent and with 10 or more feet of relief. As such, these areas slope meet the KCC criteria for regulated steep slopes.

Based on KCC 21A.24.310, development standards for steep slope hazard areas, a buffer is required from all edges of steep slope hazard areas, and the size of the buffer is based upon a critical area report prepared by a geotechnical engineer. Because grading of the regulated steep slope or the area immediately above is not planned, stormwater from the splash pad area will be contained, and none of the steep slope vegetation (mowed grass) will be disturbed, and as the slope was created through previous legal activity, it is our opinion that a buffer is not necessary for the regulated steep slope north-northwest of the splash pad provided that proper erosion control measures are in place during construction.

Landslide Hazard

The KCC characterizes landslide hazard areas as follows:

An area subject to severe risk of landslide, such as:

A. An area with a combination of:

- *Slopes steeper than fifteen percent of inclination;*
- *Impermeable soils, such as silt and clay, frequently interbedded with granular soils, such as sand and gravel; and*
- *Springs or ground water seepage;*

B. An area that has shown movement during the Holocene epoch, which is from ten thousand years ago to the present, or that is underlain by mass wastage debris from that epoch;

C. Any area potentially unstable as a result of rapid stream incision, stream bank erosion or undercutting by wave action;

D. An area that shows evidence of or is at risk from snow avalanches; or

E. An area located on an alluvial fan, presently or potentially subject to inundation by debris flows or deposition of stream-transported sediments.

Published mapping and our explorations and observations confirm that the site and the constructed cut slopes to the north of the splash pad are comprised of dense to very dense native glacial till. We have not observed evidence of groundwater seepage from the slopes during multiple site visits over the course of a year, and the slopes do not contain evidence of continual or periodic seepage. Consequently, we have concluded that the north cut slopes do not meet the criteria for a landslide hazard.

Coal Mine Hazard

KCC characterizes coal mine hazard areas into three distinct categories, as defined below:

- *Declassified coal mine areas are those areas where the risk of catastrophic collapse is not significant and that the hazard assessment report has determined do not require special engineering or architectural recommendations to prevent significant risks of property damage. Declassified coal mine areas typically include, but are not limited to, areas underlain or directly affected by coal mines at depths of more than three hundred feet as measured from the surface;*

- *Moderate coal mine hazard areas are those areas that pose significant risks of property damage that can be mitigated by implementing special engineering or architectural recommendations. Moderate coal mine hazard areas typically include, but are not limited to, areas underlain or directly affected by abandoned coal mine workings from a depth of zero, which is the surface of the land, to three hundred feet or with overburden-cover-to-seam thickness ratios of less than ten to one depending on the inclination of the seam; and*

- *Severe coal mine hazard areas are those areas that pose a significant risk of catastrophic ground surface collapse. Severe coal mine hazard areas typically include, but are not limited to, areas characterized by unmitigated openings such as entries, portals, adits, mine shafts, air shafts, timber shafts, sinkholes, improperly filled sinkholes and other areas of past or significant probability for catastrophic ground surface collapse; or areas characterized by , overland surfaces underlain or directly affected by abandoned coal mine workings from a depth of zero, which is the surface of the land, to one hundred fifty feet.*

Based on our review of on-line coal mine mapping provided by the Department of Natural Resources and King County iMap, underground coal mining is reported to have occurred below and north of fields 3 and 4 and also at and to the east of the splash pad location. The USGS survey map *Preliminary Geologic Map and Brief Description of the Coal Fields of King County, Washington* (1945) indicates that mining occurred at depths of approximately 250 to 435 below the ground surface in the New Lake Youngs Mine near the splash pad. The overburden-cover-to seam thickness ratio of the apparent shallowest workings is reported as approximately 35:1. Based on the distance below the ground surface that mining occurred and the overburden-cover-to seam thickness, this condition meets the criteria for a declassified coal mine hazard, in our opinion. In accordance with KCC21A.24.210.B, all alterations are allowed within declassified coal mine areas.

Seismic Hazard

KCC 21A.06.1045 defines a seismic has as "…*an area subject to severe risk of earthquake damage from seismically induced settlement or lateral spreading as a result of soil liquefaction in an area underlain by cohesionless soils of low density and usually in association with a shallow groundwater table*". The site is underlain by dense to very dense glacial till in turn underlain by sedimentary bedrock and lacks significant groundwater. Consequently, the risk of liquefaction occurring during a seismic event is very low, and the site does not meet the criteria per the seismic hazard definition.

4.3 Site Preparation

4.3.1 Erosion Control Measures

We expect site preparation will begin with installation of erosion control measures. Stripped surfaces and soil stockpiles are typically a source of runoff sediments. We recommend that silt fences, berms, and/or swales be installed around the downslope side of stripped areas and stockpiles, in order to capture runoff water and sediment. If earthwork occurs during wet weather, we recommend that all stripped surfaces be covered with straw to reduce runoff erosion, whereas soil stockpiles should be protected with anchored plastic sheeting. Where recommend the use of biodegradable straw, coir, or compost wattles as an alternative to conventional plastic and steel silt fence in order to reduce the use of material that is impossible or difficult to re-use or recycle.

4.3.2 Temporary Drainage

Stripping, excavation, grading, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and provide proper control of erosion. The site should be graded to prevent water from ponding in construction areas and/or flowing into and/or over excavations. Exposed grades should be crowned, sloped, and smooth-drum rolled at the end of each day to facilitate drainage if inclement weather is forecasted. Accumulated water must be removed from subgrades and work areas immediately and prior to performing further work in the area. Equipment access may be limited, and the amount of soil rendered unfit for use as structural fill may be greatly increased if drainage efforts are not accomplished in a timely manner.

4.3.3 Clearing, Grubbing and Stripping

In preparation for grading, we recommend removal of all existing surficial vegetation. Following clearing of surficial vegetation, organic-rich topsoil (soils containing more than 4 percent organic material by weight) should be stripped from any locations that are to receive structural fill, footing and slab subgrades, and utility trenches. These materials should be wasted away from the improvement area or removed from the project site. Based on our observations, the thickness of surficial organic material requiring removal may be on the order of about 3 inches, but variation in this depth should be expected.

4.3.4 Existing Fill Removal

Site preparation is recommended to include selective removal of existing undocumented fill material containing substantial organics or deleterious debris and underlying relic organic topsoil. Please note that the nature of fill is such that its extent, composition, and thickness can vary over relatively short distances. Fill depth and composition was observed to vary across the site from about 1-½ to 3 feet. These materials should be evaluated during construction and removed as necessary under the observation of ZGA representative. Our representative will identify unsuitable materials that should be removed or may be re-used as structural fill. The resultant excavations should be backfilled in accordance with the subsequent recommendations for structural fill placement and compaction.

4.3.5 Subgrade Preparation and Protection

Once site preparation is complete, all areas that are at design subgrade elevation or areas that will receive new structural fill should be compacted to a firm and unyielding condition if possible. In the event the exposed subgrade becomes unstable, yielding, or unable to be compacted due to high moisture conditions, we recommend that the materials be over-excavated and replaced with structural fill compacted in accordance with the recommendations detailed in Section 4.4 – *Structural Fill Materials, Placement, and Compaction*.

4.3.6 Freezing Conditions

If earthwork takes place during freezing conditions, all exposed subgrades should be allowed to thaw and then be compacted prior to placing subsequent lifts of structural fill, pouring concrete, or paving. Alternatively, the frozen material could be stripped from the subgrade to expose unfrozen soil. The frozen soil should not be re-used as structural fill until allowed to thaw and adjusted to the proper moisture content; please note that this may not be feasible during winter months.

4.4 Structural Fill Materials, Placement, and Compaction

Structural fill includes any material placed below foundations, slabs, and pavement sections, within utility trenches, and behind retaining walls. Prior to the placement of structural fill, all surfaces to receive fill should be prepared as previously recommended in the *Site Preparation* section.

4.4.1 Re-use of Site Soils as Structural Fill

The suitability for re-use of site soils as structural fill depends on the composition and moisture content of the soil. Soils observed at the boring locations generally consisted of sand with a relatively high silt and varying gravel content. As the fines content increases, soil becomes increasingly sensitive to small changes in moisture content. Soils containing more than about 5 percent fines cannot be consistently compacted to the appropriate levels when the moisture content is more than approximately 2 percent above or below the optimum moisture content (per ASTM D1557). The optimum moisture content is the moisture content which results in the greatest compacted dry density with a specified compactive effort.

During dry weather, we expect site soils will be suitable for re-use as structural fill. However, some moisture conditioning consisting of drying may be required if grading occurs during wet weather or under wet site conditions. Due to the relatively high fines content of the glacial till soils, these soils should be considered highly moisture sensitive. From a compositional standpoint, the non-organic site soils are considered adequate for use as structural fill. However, the feasibility of using the on-site soils as structural fill depends greatly on the weather conditions at the time of placement and compaction. If site soils are planned for re-use, they should be protected from an increase in moisture content during periods of wet weather. At a minimum, we recommend stockpiles of excavated material to be used as structural fill be covered with plastic sheeting if rain is forecast.

We recommend that site soils used as structural fill have less than 3 percent organics on a dry weight basis, have no woody debris greater than ½-inch in diameter, and contain no other deleterious materials. We recommend that all pieces of organic material greater than ½-inch in diameter be picked out of the fill before it is placed and compacted. Deleterious debris includes waste building materials, organics, trash, and asphalt and, if encountered, it should be removed from the soil prior to its re-use as structural fill.

4.4.2 Imported Structural Fill

Imported structural fill may be required. For general purposes, we recommend imported fill meet the following requirements:

- During Extended Periods of Dry Weather: Common Borrow Options 1 or 2 per Section 9-03.14(3) of the 2022 WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* (Publication M41-10). The on-site soil would be classified as Common Borrow.

- During Wet Weather: Gravel Borrow per Section 9-03.14(1) of the WSDOT Standard Specs.

It should be noted that Common Borrow typically contains a significant fraction of fines (silt and clay). During wet weather, Common Borrow may likely become too wet for re-use as structural fill.

4.4.3 Moisture Content

The suitability of soil for use as structural fill will depend on the prevailing weather at the time of construction, the moisture content of the soil, and the fines content (that portion passing the U.S. No. 200 sieve) of the soil. Imported fills should be delivered to the site at a moisture content within ±2 percent of optimum. This is important as there will be minimal room on this site to moisture condition imported fill materials should they arrive at the site wet or dry of optimum.

4.4.4 Fill Placement and Compaction

We recommend that structural fill be placed in horizontal lifts of a thickness that can be compacted to the recommended levels with the equipment available (typically 8 to 12 inches). Our recommendations for soil compaction are summarized in the following table. We recommend that a ZGA representative be present during grading so that an adequate number of density tests may be conducted as structural fill placement occurs.

Recommend Soil Compaction Levels	
Location	**Minimum Percent Compaction***
Fill below foundations	95
Fill below concrete slabs exclusive of above the below-grade storage tank	95
Below-grade storage tank excavation cavity	Per tank manufacturer recommendation
Cast-in-place retaining walls	92 - 95
Upper two feet of utility trench backfill	95
Utility trenches below two feet	90
Landscape areas	90 or as required by Parks
* ASTM D1557 Modified Proctor Maximum Dry Density	

4.5 Temporary and Permanent Slopes

Temporary excavations are expected to construct the below-grade storage tank, pump house foundations, and for installation of underground utilities. Temporary excavation slope stability is a function of many factors, including:

- The presence and abundance of groundwater;
- The type and density of the various soil strata;
- The depth of cut;
- Surcharge loadings adjacent to the excavation; and
- The length of time the excavation remains open.

It is exceedingly difficult under the variable circumstances to pre-establish a safe and “maintenance-free” temporary cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe temporary slope configurations since the contractor is continuously at the job site, able to observe the nature and condition of the cut slopes, and able to monitor the subsurface materials and groundwater conditions encountered. Unsupported vertical slopes or cuts deeper than 4 feet are not recommended. The cuts should be adequately sloped, shored, or supported to prevent injury to personnel from local sloughing and spalling. The excavation should conform to applicable Federal, State, and Local regulations.

According to Chapter 296-155 of the Washington Administrative Code (WAC), the contractor should make a determination of excavation side slopes based on classification of soils encountered at the time of excavation. Temporary cuts may need to be constructed at flatter angles based upon the soil moisture, soil density, and groundwater conditions at the time of construction. Adjustments to the slope angles should be determined by the contractor at that time.

Installation of the below-grade storage tank will require a substantial excavation. The existing fill and medium dense weathered glacial till (ESU-1 and ESU-2), which extended to depths of about 1.5 to 5 feet, respectively, meet the criteria for Type C soils per Chapter 296-155 WAC, *Part N, Excavation, Trenching, and Shoring.* Temporary excavation slopes as deep as 20 feet in these soils may generally be planned with an inclination no steeper than 1.5H:1V (Horizontal: Vertical). The underlying dense to very dense native glacial till (ESU-3) meets the criteria for Type A soil and temporary excavations as deep as 20 feet may be designed with an inclination no steeper than 0.75H:1V. The locations of existing facilities, such as the picnic shelter, relative to the extent of temporary excavations should be taken into account during the design process. The feasibility of these temporary excavation slope inclinations should be verified during excavation, and it should be recognized that conditions disclosed during excavation may warrant the use of shallower temporary excavation slope inclinations. Otherwise, temporary excavation shoring may be required.

We recommend designing permanent cut or fill slopes constructed in native or properly compacted fill soils at a 2H:1V inclination or flatter. All permanent cut and fill slopes should be adequately protected from erosion both temporarily and permanently.

4.6 Shallow Foundation Recommendations

The project will include construction of a small storage building and possibly low cast-in-place concrete retaining walls. Based on the soil conditions and our analyses, conventional shallow spread continuous and column footings appear feasible provided that the foundation subgrades are prepared in accordance with this report. Recommendations for shallow spread footings are provided below.

4.6.1 Seismic Criteria Summary

Seismic Criteria Summary	
Code Criteria	**Site Classification**
2021 International Building Code (IBC) [1]	C [2]
Latitude	47.442582
Longitude	-122.11884
S_S Spectral Acceleration for a Short Period	1.5g (Site Class C)
S_1 Spectral Acceleration for a 1-Second Period	0.5g (site Class C)
S_{MS} Maximum considered spectral response acceleration for a Short Period	1.66g (Site Class C)
S_{M1} Maximum considered spectral response acceleration for a 1-Second Period	0.68g (Site Class C)
S_{DS} Five-percent damped design spectral response acceleration for a Short Period	1.11g (Site Class C)
S_{D1} Five-percent damped design spectral response acceleration for a 1-Second Period	0.45g (Site Class C)
V_{S30} Average soil/bedrock shear wave velocity in the upper 100 feet in ft./sec. (estimated)	1,450 to 2,100 (Site Class C)

1. In general accordance with the *2021 International Building Code,* Section 1613.3.2 and *ASCE 7-22*, Chapter 20. IBC Site Class is based on the average characteristics of the upper 100 feet of the subsurface profile.
2. The explorations completed for this study extended to a maximum depth of about 12.5 feet below grade. ZGA therefore determined the Site Class assuming that very dense glacially consolidated soils and sedimentary bedrock extend to 100 feet as suggested by logs of nearby explorations and published geologic maps for the project area.

4.6.2 Foundation Subgrade Preparation

Finished floor and foundation subgrade elevations for the proposed storage building or retaining wall foundation subgrade elevations were not available as of the date of this report. We recommend that footings bear directly on at least medium dense to dense ESU-2 soils or on properly compacted structural fill extending down to ESU-2 soils. We recommend preparing footing subgrades in accordance with the *Subgrade Preparation* section. For areas where loose ESU-1 soils are over-excavated and replaced we recommend extending the excavation width beyond the edges of the footings a distance equal to the excavation depth. Replacement structural fill should be placed and compacted in accordance with the recommendations outlined in the *Fill placement and Compaction* section.

4.6.3 Allowable Bearing Pressure

Continuous and isolated column footings bearing on footing subgrades prepared as recommended above may be designed for a maximum allowable net bearing capacity of 3,000 psf. A one-third increase of the bearing pressure may be used for short-term dynamic loads such as wind and seismic forces. We recommend providing ZGA the opportunity to verify foundation subgrade conditions prior to form and reinforcing placement. We estimate that total settlement of foundations constructed as described herein will be less than 1 inch. Differential settlement is estimated to be about ½ inch or less in 40 feet.

4.6.4 Shallow Foundation Depth and Width

For frost protection, we recommend the bottom of all exterior footings bear at least 18 inches below the lowest adjacent outside grade, whereas the bottoms of interior footings should bear at least 12 inches below the surrounding slab surface level. We recommend that all continuous wall and isolated column footings be at least 12 and 24 inches wide, respectively.

4.6.5 Resistance to Lateral Loads

Lateral loads can be resisted by a combination of base friction and passive earth pressures acting on the face of footing elements. For footings founded as recommended above, we recommend using an ultimate base friction coefficient of 0.5. For footings backfilled with structural fill placed in accordance with this report, we recommend using an ultimate passive earth pressure value of 400 pounds per cubic foot (pcf). We recommend passive resistance be neglected within the upper 18 inches of embedment. The above values do not include safety factors. Appropriate safety factors or resistance factors should be used for design.

4.7 Below Grade Storage Tank

4.7.1 General

We understand that the project will include installation of a below-grade water storage tank that may be up to as deep as 15 feet below-grade. Dense to very dense ESU-3 soils were encountered between about 1-1/2 to 5 feet below grade at the boring B-4 location. Although boring B-4 was terminated at a depth of about 7-1/2 feet under refusal in very dense coarse glacial till soil, nearby boring B-1 disclosed very dense glacial till soils to just below the likely storage tank excavation lower elevation of about 561-1/2 feet. Therefore, we anticipate the below-grade storage tank will bear directly on dense to very dense ESU-3 soils.

4.7.2 Buoyancy Considerations

Given the presence of perched groundwater within the ESU-2 and ESU-3 soils when the borings were advanced, seasonal groundwater seepage and stormwater migration into the backfilled storage tank excavation should be expected. The contractor should be prepared to dewater excavations to the extent necessary to allow for installation of the tank foundation, tank, and backfill material.

We anticipate that the tank will likely be subject to buoyancy forces if the excavation is not equipped with a permanent drain. Potential buoyant forces acting on the tank may be calculated by multiplying the volume of the portion of the tank below the water table (in cubic feet) by 62.4 pcf. Buoyant forces may be resisted by a combination of the weight of the tank, the weight of the concrete slab or foundation to which it is attached, by pre-cast or cast-in-place concrete anchors installed in the excavation, and by the weight of backfill placed above the tank and above installing flanges on the tank base (if so equipped). Assuming that the tank cavity is primarily backfilled with pea gravel, we recommend considering a unit density of 100 pounds per cubic foot (pcf) for backfill above the excavation water elevation, and 38 pcf for backfill below the water level. For design purposes, we recommend considering that water within the tank excavation may be as high as approximately 573.5 feet, or about 3 feet below existing grade.

4.8 On-Grade Concrete Slabs

We anticipate that concrete slabs-on-grade will be used below the storage/maintenance building and as the base of the splash pad itself. The following sections summarize our recommendations for on-grade concrete slabs.

4.8.1 Subgrade Preparation

After removal of organic material and other items noted in the *Site Preparation* section of this report, we recommend that at least the upper 12 inches of material below the slab base be moisture conditioned (if needed) and compacted to a firm and unyielding condition and to a minimum 95 percent of the modified Proctor maximum dry density per ASTM D1557. If the slab is constructed above structural fill material, we recommend compacting the fill to at least 95 percent density per ASTM D 1557.

4.8.2 Capillary Break

To provide a capillary break and uniform slab bearing surface, we recommend that the maintenance building be underlain by a minimum 4-inch-thick layer of compacted crushed rock meeting the requirements of WSDOT Specification 9-03.9(3), Crushed Surfacing Top Course, with the modification of a maximum of 7 percent passing the U.S. No. 200 sieve. Alternatively, clean angular gravel such as No. 7 aggregate per WSDOT 9-03.1(4)C could be used for this purpose. Alternative capillary break materials should be submitted to the geotechnical engineer for review and approval before use. We recommend that the need for a capillary break below the splash pad slab be evaluated in light of the recommendations of the manufacturers that provide the splash pad piping and drainage hardware.

4.9 Retaining Walls

The site has a very gentle slope downward to the north. Consequently, it may be necessary to construct retaining walls at either the north or south sides of the splash pad (or both) to support relatively low cuts or fills needed to prepare a level surface for the splash pad. We anticipate that walls will be less than 6 feet tall. At the time this report was prepared, the types of wall had yet to be determined. We anticipate that segmental concrete block walls employing geogrid reinforced backfill would not be practical given the amount of plumbing that will be installed below the splash pad. As such, we have provided recommendations and design values below for cast-in-place (CIP) walls that will not employ reinforced backfill material. The foundation recommendations already provided in Section 4.5 are applicable to new retaining walls.

Retaining Wall Lateral Earth Pressure Recommendations	
Parameter	**Recommended Value**
Granular Wall Backfill Total Unit Weight, γ (pcf)	130
Friction Angle, Φ	36
Active Lateral Earth Pressure Coefficient, K_a (level backfill)	0.26
At-Rest Lateral Earth Pressure Coefficient, K_0 (level backfill)	0.41

The above-recommended lateral earth pressures assume that adequate drainage measures are provided to limit the potential for buildup of hydrostatic pressures. All backfilled walls should include a drainage aggregate zone extending a minimum of two feet from the back of wall for the full height of the wall and wide enough at the base of the wall to allow seepage to flow to the footing drain. The drainage aggregate should consist of material meeting the requirements of WSDOT 9-03.12(2), Gravel Backfill for Walls. A minimum 4-inch diameter, rigid, perforated PVC or HDPE drainpipe should be provided at the base of backfilled walls to collect and direct subsurface water to an appropriate discharge point. We recommend placing a non-woven geotextile, such as Mirafi 140N, or equivalent, around the free draining backfill material.

4.10 Stormwater Management Considerations

Since a stormwater infiltration feature location had not been determined when the field exploration took place, boring B-2 was advanced in the general vicinity that an infiltration feature may likely be located in the lawn immediately south of the sidewalk to the north of the playground and picnic shelter (and downslope of the splash pad). Low permeability glacially consolidated soils were encountered in boring B-2 at a depth of approximately 1.5 feet, perched groundwater was encountered at approximately 2.5 feet, and auger refusal was encountered at approximately 7 feet. These observations indicate a shallow perched groundwater table and a shallow low permeability soil horizon. Glacial till is typically considered a hydraulically restrictive layer not suitable for conventional stormwater infiltration. As such, we do not expect stormwater infiltration will be feasible at this site.

Per Section 5.2.1 *General Requirements for Infiltration Facilities* (Page 5-44 of the 2021 King County *Surface Water Design Manual*), a geotechnical professional is required to evaluate site conditions and provide *"… a written opinion… that sufficient permeable soil exists at the proposed facility location to allow construction of a property functioning infiltration facility".* Based on the conditions observed at the boring locations (the presence of dense to very dense lodgement glacial till and shallow perched groundwater) and the results of laboratory testing that indicate a relatively high fines content of the glacial till soils, it is our professional opinion that the site lacks the soil and groundwater conditions that would *"…allow construction of a property functioning infiltration facility"* per the *Manual.* Consequently, we offer the following:

- We recommend that the two field infiltration test pits included in our scope of services not be completed and that Otak and Parks consider stormwater infiltration infeasible for the proposed splash pad project.

- We recommend that Otak and Parks consider others means of stormwater management, such as directing surface water from the splash pad to the existing storm sewer system or directing water to a nearby location which would support dispersion.

4.11 Trail Recommendations

A new section of Hot Mix Asphalt (HMA) trail will be constructed to access the splash pad features. The trail will accommodate pedestrians as well as park service vehicles. We recommend a pavement section consisting of 2 inches of HMA above 4 inches of crushed surfacing. Our recommendations summarized below reference WSDOT Publication M41-10.

Subgrade Preparation and Compaction: We recommend constructing the new trail above at least medium dense and non-yielding native soil or structural fill compacted to at least 95 percent density.

HMA: We recommend that HMA conform to Section 9-02.1(4) for PG 58S-22 or PG 58H-22 *Performance Graded Asphalt Binder* as presented in the WSDOT Standard Specifications. We also recommend that the gradation of the asphalt aggregate conform to the aggregate gradation control points for a ½-inch mix as presented in WSDOT Specification 9-03.8(6), *HMA Proportions of Materials.*

Crushed Surfacing Top Course: We recommend that *Crushed Surfacing Top Course* (CSTC) conform to Section 9-03.9(3) of the WSDOT Standard Specifications.

Compaction and Paving: All base material should be compacted in accordance with WSDOT Specification 2-03.3(14)C *Compacting Earth Embankments – Method C*. We recommend that HMA be compacted to a minimum of 92 percent and a maximum of 96 percent of the theoretical maximum density. Placement and compaction of HMA should conform to the requirements of Section 5-04 of the WSDOT Standard Specifications. This includes weather limitations as specified in Section 5-04.3(1) and maximum nominal asphalt lift thickness as specified in Section 5-04.3(7), Table 6.

5.0 CLOSURE

The analysis and recommendations presented in this report are based, in part, on the explorations completed for this study. The number, location, and depth of the explorations were completed within the constraints of budget and site access so as to yield the information to formulate our recommendations. Project plans were in the preliminary stage at the time this report was prepared. We therefore recommend that ZGA be provided an opportunity to review the final plans and specifications when they become available in order to assess that the recommendations and design considerations presented in this report have been properly interpreted and implemented into the project design.

The performance of earthwork, structural fill, foundations, and temporary shoring depend greatly on proper site preparation and construction procedures. We recommend that Zipper Geo Associates, LLC be retained to provide geotechnical engineering services during the earthwork-related construction phases of the project. If variations in subsurface conditions are observed at that time, a qualified geotechnical engineer could provide additional geotechnical recommendations to the contractor and design team in a timely manner as the project construction progresses.

This report has been prepared for the exclusive use of Otak, and their agents, for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made. Site safety and excavation support are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Zipper Geo Associates, LLC reviews the changes and either verifies or modifies the conclusions of this report in writing.

GEOTECHNICAL ENGINEERING REPORT - DRAFT

PETROVITSKY PARK SPLASH PAD
16400 SE PETROVITSKY ROAD
KING COUNTY (RENTON), WASHINGTON

ZGA Project No. 2781.01
May 15, 2024

Prepared for:

Otak and King County Parks

Prepared by:

ZipperGeo
Geoprofessional Consultants

19019 36th Avenue W., Suite E
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ZipperGeo
Geoprofessional Consultants

ZGA Project No. 2781.01
May 15, 2024

Otak, Inc.
11241 Willows Road Northeast, Suite 200
Redmond, Washington 98052

Attention: Mr. Keith Bates, ASLA

Subject: Geotechnical Engineering Report - **DRAFT**
Petrovitsky Park Splash Pad
16400 SE Petrovitsky Road
King County (Renton), Washington
Otak Project no. 21250.B00

Dear Keith:

In accordance with your request, Zipper Geo Associates, LLC (ZGA) has completed the subsurface exploration and geotechnical engineering evaluation for the proposed Petrovitsky Park Splash Pad project. This report presents the findings of the subsurface exploration and geotechnical recommendations for the project. Our work was completed in general accordance with the scope of services described in our *Subconsultant Agreement* (dated April 4, 2024). We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further assistance, please contact us.

Sincerely,

Zipper Geo Associates, LLC

DRAFT

Martin R. Cross, LG
Project Geologist

DRAFT

Robert A. Ross, PE
Managing Principal

Distribution: Addressee (1 pdf)

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PETROVITSKY PARK SPLASH PAD
16400 SE PETROVITSKY ROAD
KING COUNTY (RENTON),WASHINGTON

Project No. 2781.01
May 15, 2024

1.0 INTRODUCTION

This report presents the surface and subsurface conditions encountered at the site and our geotechnical engineering recommendations for the proposed Petrovitsky Park Splash Pad. Our scope of services included reviewing readily available geologic data, a site reconnaissance, subsurface evaluation, laboratory testing, geotechnical engineering analysis, and preparation of this report. The project description, site conditions, and our geotechnical conclusions and recommendations are presented in the text of this report. Supporting data including detailed exploration logs and field exploration procedures and the results of laboratory testing are presented as appendices.

1.1 Site Description

Petrovitsky Park comprises one irregularly-shaped parcel (King County Parcel No. 362305-9007) in the Fairwood neighborhood of unincorporated King County, Washington. The site is located generally northeast of the SE Petrovitsky Road and Parkside Way SE intersection. The property encompasses approximately 88 acres and is bordered by forested land to the north, residential development to the east, SE Petrovitsky Road and residential development to the south, and Ridgewood Elementary School and residential developments to the west.

1.2 Project Understanding

We understand the proposed project will include construction of the new splash pad amenity. The improvements are planned for construction adjacent to the existing play areas and covered picnic shelter as shown on the Site and Exploration Plan, Figure 1. The splash pad will include a holding/recirculation tank located below the splash pad, a maintenance and chemical storage building to the south of the existing picnic shelter, low retaining walls around the pad, and a new asphalt-paved walkway. We understand that the excavation for the holding tank may be as deep as about 15 feet below grade. The storage building will be one story and likely of light metal frame and skin construction.

2.0 SURFACE CONDITIONS

The location of the proposed splash pad consists of open lawn that gently slopes downward at approximately 7 percent towards the north to northeast to an asphalt-paved walkway that runs east-west and above cut slope descending to the ball fields to the north. According to the survey of the *Petrovitsky Park Existing Conditions* (dated April 17, 2024) provided to us by Otak, ground surface elevations range from a high of about 579 feet at the south of the existing play area and picnic shelter to about 570 feet along the asphalt walk to the north. A stormwater management pond is located a short distance east of the picnic shelter. Review of historical aerial photographs suggests that the pond holds water most of the year.

3.0 SUBSURFACE CONDITIONS

3.1 General

In order to characterize subsurface soil and groundwater conditions at the project site, we reviewed readily available published surficial geologic mapping and advanced four (4) geotechnical borings on February 8, 2024. The following sections describe our understanding of subsurface soil and groundwater conditions based on the mapping and our borings.

3.2 Published Surficial Geologic Mapping

The publication *Surficial Geologic Map of the Maple Valley Quadrangle, King County, Washington* (USGS, Map MF-2287, 1995) indicates that the subject site and surrounding area have been mapped as containing glacial till deposits (Qgt). Glacial till is a glacially consolidated and heterogenous soil consisting of clay, silt, sand, gravel, cobbles, and boulders in varying amounts. Glacial till is colloquially termed “hardpan” in western Washington due to its compact and dense nature resulting from the deposit forming below the overburden of historical glaciers. As a result of the soil density and generally high fines content (the soil fraction passing the US No. 200 sieve), glacial till is generally characterized by a low permeability. Given the developed nature of the site, fill material is likely present as well.

3.3 Soil Conditions

The borings were completed at the locations of the proposed splash pad features approximately as shown on Figure 1. The borings were advanced to depths of approximately 7 to 12-¾ feet. Subsurface conditions disclosed by the borings were generally consistent with the published mapping. In general, each boring encountered a shallow fill horizon underlain by native glacial till that extended to the borings’ termination depths.

Soils were visually described during recovery in general accordance with the *Explanation of Exploration Logs* provided in Appendix A. Detailed descriptive logs of the subsurface explorations and the procedures utilized in the subsurface exploration program are presented in Appendix A. Generalized descriptions of subsurface soil conditions observed in specific areas of the site are presented below. Please refer to the exploration logs in Appendix A for a more detailed description of the conditions encountered at the exploration locations. Stratification boundaries on the boring logs represent the approximate depth of changes in soil types, although the transition between materials may have been gradual. If variations become apparent during construction, it may be necessary to re-evaluate the recommendations of this report.

For purposes of describing soil conditions observed in the borings, and for reference in other sections of this report, soils with similar engineering characteristics were grouped together into Engineering Stratigraphic Units (ESUs). The following paragraphs provide our interpretation of ESUs disclosed by the borings. ESUs are ordered in a top-down stratigraphic sequence. Shallow surficial conditions such as topsoil and gravel surfacing are not described as separate ESUs below, and the reader is referred to the boring logs attached in Appendix A for information regarding shallow surficial conditions.

ESU-1 Fill: Soils interpreted to be fill were observed at each boring location extending from below the grass and topsoil to about 1-½ to 3 feet below existing grade. ESU-1 soils were observed to generally consist of loose to medium dense, dark brown to brown, silty sand and sandy silt with a varying gravel content and trace organics.

ESU-2 Weathered Glacial Till: We observed soils interpreted to be weathered glacial till at the locations of borings B-1, B-3, and B-4 below the fill, extending to about 4 to 5 feet below grade. ESU-2 soils generally consisted of medium dense to dense, brown to light gray with some oxidation mottling, silty sand with a varying gravel content. Given the relatively high fines content and density of these soils, unweathered till typically has a relatively low permeability.

ESU-3 Unweathered Glacial Till: We observed soils interpreted to be unweathered glacial till below the weathered glacial till (borings B-1, B-3, an B-4) or below the fill (B-2), extending to the termination of each boring between 7 to 12-¾ feet below grade. ESU-3 soils generally consisted of dense to very dense, light brown to gray, sand with a high gravel and silt content. Given the relatively high fines content and density of these soils, unweathered till typically has a relatively low permeability.

3.4 Groundwater Conditions

Perched groundwater was encountered within the ESU-2 (B-3) and ESU-3 (B-1, B-2) soils at approximately 2-½ to 7-½ feet below existing grade. We have interpreted that the perched groundwater encountered at the time of drilling develops due to the hydraulically restrictive nature of the glacial till. It should be noted that groundwater and soil moisture content conditions will likely vary seasonally and in response to precipitation events, land use, irrigation, and other factors. In general, seasonal high groundwater in western Washington occurs toward the end of the local wet season, typically around the end of May.

For discussion purposes, it should be noted that the subject site within the likely depth of excavations necessary for construction does not support a “groundwater table”. Rather, sandy horizons and zones within and above the nature glacial till will tend to wet to saturated during the wetter time of year. These areas of perched groundwater may vary considerably in depth and be laterally discontinuous.

3.5 Summary of Laboratory Testing

Laboratory testing was completed on select soil samples recovered from the borings. The results of moisture content testing are shown on the boring logs in Appendix A. The results of grain size analysis are presented in Appendix B.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 General

Based on our understanding of the proposed project, our review of the documents referenced herein, our site reconnaissance, subsurface explorations, and laboratory testing results, we have concluded that the proposed project appears feasible from the geotechnical perspective. Our general geotechnical considerations are summarized below.

- The site-characteristic medium dense to very dense glacial till soils are well suited for support of the proposed splash pad elements including slabs, building and wall foundations, and the below-grade storage tank.

- The low permeability of the unweathered glacial till is such that the excavation housing the below-grade storage tank may fill with water following construction (unless it is drained). We recommend constructing the tank with measures intended to counteract possible buoyant forces acting on the tank following construction.

- The glacial till soils have a high fines content. Consequently, they should be considered moisture-sensitive and scheduling grading and foundations construction for the dryer summer and early fall months will reduce, but may not eliminate, the likelihood of grading delays due to wet weather and the need for select import fill materials.

- The low permeability of the glacial till soils and the development of a seasonal shallow perched groundwater condition is such that stormwater infiltration at the splash pad location is not feasible from the geotechnical perspective, in our opinion.

The following sections present specific geotechnical recommendations. Our recommendations are based on the observed soil conditions at specific exploration locations. Differing soil conditions than those observed at the boring locations may become evident during construction. Our recommendations are further based on the assumption that earthwork for site grading, utilities, foundations, and slabs will be monitored by a ZGA representative.

4.2 Geologically Hazardous Areas

In general accordance with Chapter 21A.24 of the King County Code (KCC), we utilized existing site plans, site observations, County mapping, online mapping applications, and readily available geologic maps and publications to determine the presence of regulated geologically hazardous areas in the project vicinity. Our conclusions regarding geologic hazard critical areas are summarized below, with italics indicating KCC definitions and our responses following.

Erosion Hazard

Per KCC 21A.06.415, an erosion hazard is "*…an area underlain by soils that is subject to severe erosion when disturbed. These soils include, but are not limited to, those classified as having a severe to very severe erosion hazard according to the United States Department of Agriculture Soil Conservation Service, the 1990 Snoqualmie Pass Area Soil Survey, the 1973 King County Soils Survey or any subsequent revisions or addition* *by or to these sources such as any occurrence of River Wash ("Rh") or Coastal Beaches ("Cb") and any of the following when they occur on slopes inclined at fifteen percent or more:*

The Alderwood gravely sandy loam ("AgD");

The Alderwood and Kitsap soils ("AkF");

The Beausite gravely sandy loam ("BeD" and "BeF");

The Kitsap silt loam ("KpD");

The Ovall gravely loam ("OvD" and "OvF");

The Ragnar fine sandy loam ("RaD"); and

The Ragnar-Indianola Association ("RdE")".

The Natural Resource Conservation Service (formerly the Soil Conservation Service) has mapped the proposed splash pad location as mantled by the Alderwood Gravelly Sandy Loam, 0 – 8 percent slopes, (AgB). These soils do not meet the KCC criteria for an erosion hazard.

Steep Slope Hazard

The KCC characterizes regulated steep slope hazard areas as described below:

An area on a slope of forty percent inclination or more within a vertical elevation change of at least ten feet. For the purpose of this definition, a slope is delineated by establishing its toe and top and is measured by averaging the inclination over at least ten feet of vertical relief. Also for the purpose of this definition:

- *The "toe" of a slope means a distinct topographic break in slope that separates slopes inclined at less than forty percent from slopes inclined at forty percent or more. Where no distinct break exists, the "toe" of a slope is the lower most limit of the area where the ground surface drops ten feet or more vertically within a horizontal distance of twenty five feet; and*

- *The "top" of a slope is a distinct topographic break in slope that separates slopes inclined at less than forty percent from slopes inclined at forty percent or more. Where no distinct break exists, the "top" of a slope is the upper-most limit of the area where the ground surface drops ten feet or more vertically within a horizontal distance of twenty-five feet.*

Based on our review of mapped site topography and field measurements, the cut slope extending below and north-northwest of the splash pad location toward field 1 includes segments inclined at or exceeding

40 percent and with 10 or more feet of relief. As such, these areas slope meet the KCC criteria for regulated steep slopes.

Based on KCC 21A.24.310, development standards for steep slope hazard areas, a buffer is required from all edges of steep slope hazard areas, and the size of the buffer is based upon a critical area report prepared by a geotechnical engineer. Because grading of the regulated steep slope or the area immediately above is not planned, stormwater from the splash pad area will be contained, and none of the steep slope vegetation (mowed grass) will be disturbed, and as the slope was created through previous legal activity, it is our opinion that a buffer is not necessary for the regulated steep slope north-northwest of the splash pad provided that proper erosion control measures are in place during construction.

Landslide Hazard

The KCC characterizes landslide hazard areas as follows:

An area subject to severe risk of landslide, such as:

A. An area with a combination of:

- *Slopes steeper than fifteen percent of inclination;*
- *Impermeable soils, such as silt and clay, frequently interbedded with granular soils, such as sand and gravel; and*
- *Springs or ground water seepage;*

B. An area that has shown movement during the Holocene epoch, which is from ten thousand years ago to the present, or that is underlain by mass wastage debris from that epoch;

C. Any area potentially unstable as a result of rapid stream incision, stream bank erosion or undercutting by wave action;

D. An area that shows evidence of or is at risk from snow avalanches; or

E. An area located on an alluvial fan, presently or potentially subject to inundation by debris flows or deposition of stream-transported sediments.

Published mapping and our explorations and observations confirm that the site and the constructed cut slopes to the north of the splash pad are comprised of dense to very dense native glacial till. We have not observed evidence of groundwater seepage from the slopes during multiple site visits over the course of a year, and the slopes do not contain evidence of continual or periodic seepage. Consequently, we have concluded that the north cut slopes do not meet the criteria for a landslide hazard.

Coal Mine Hazard

KCC characterizes coal mine hazard areas into three distinct categories, as defined below:

- *Declassified coal mine areas are those areas where the risk of catastrophic collapse is not significant and that the hazard assessment report has determined do not require special engineering or architectural recommendations to prevent significant risks of property damage. Declassified coal mine areas typically include, but are not limited to, areas underlain or directly affected by coal mines at depths of more than three hundred feet as measured from the surface;*

- *Moderate coal mine hazard areas are those areas that pose significant risks of property damage that can be mitigated by implementing special engineering or architectural recommendations. Moderate coal mine hazard areas typically include, but are not limited to, areas underlain or directly affected by abandoned coal mine workings from a depth of zero, which is the surface of the land, to three hundred feet or with overburden-cover-to-seam thickness ratios of less than ten to one depending on the inclination of the seam; and*

- *Severe coal mine hazard areas are those areas that pose a significant risk of catastrophic ground surface collapse. Severe coal mine hazard areas typically include, but are not limited to, areas characterized by unmitigated openings such as entries, portals, adits, mine shafts, air shafts, timber shafts, sinkholes, improperly filled sinkholes and other areas of past or significant probability for catastrophic ground surface collapse; or areas characterized by , overland surfaces underlain or directly affected by abandoned coal mine workings from a depth of zero, which is the surface of the land, to one hundred fifty feet.*

Based on our review of on-line coal mine mapping provided by the Department of Natural Resources and King County iMap, underground coal mining is reported to have occurred below and north of fields 3 and 4 and also at and to the east of the splash pad location. The USGS survey map *Preliminary Geologic Map and Brief Description of the Coal Fields of King County, Washington* (1945) indicates that mining occurred at depths of approximately 250 to 435 below the ground surface in the New Lake Youngs Mine near the splash pad. The overburden-cover-to seam thickness ratio of the apparent shallowest workings is reported as approximately 35:1. Based on the distance below the ground surface that mining occurred and the overburden-cover-to seam thickness, this condition meets the criteria for a declassified coal mine hazard, in our opinion. In accordance with KCC21A.24.210.B, all alterations are allowed within declassified coal mine areas.

Seismic Hazard

KCC 21A.06.1045 defines a seismic has as "…*an area subject to severe risk of earthquake damage from seismically induced settlement or lateral spreading as a result of soil liquefaction in an area underlain by cohesionless soils of low density and usually in association with a shallow groundwater table*". The site is underlain by dense to very dense glacial till in turn underlain by sedimentary bedrock and lacks significant groundwater. Consequently, the risk of liquefaction occurring during a seismic event is very low, and the site does not meet the criteria per the seismic hazard definition.

4.3 Site Preparation

4.3.1 Erosion Control Measures

We expect site preparation will begin with installation of erosion control measures. Stripped surfaces and soil stockpiles are typically a source of runoff sediments. We recommend that silt fences, berms, and/or swales be installed around the downslope side of stripped areas and stockpiles, in order to capture runoff water and sediment. If earthwork occurs during wet weather, we recommend that all stripped surfaces be covered with straw to reduce runoff erosion, whereas soil stockpiles should be protected with anchored plastic sheeting. Where recommend the use of biodegradable straw, coir, or compost wattles as an alternative to conventional plastic and steel silt fence in order to reduce the use of material that is impossible or difficult to re-use or recycle.

4.3.2 Temporary Drainage

Stripping, excavation, grading, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and provide proper control of erosion. The site should be graded to prevent water from ponding in construction areas and/or flowing into and/or over excavations. Exposed grades should be crowned, sloped, and smooth-drum rolled at the end of each day to facilitate drainage if inclement weather is forecasted. Accumulated water must be removed from subgrades and work areas immediately and prior to performing further work in the area. Equipment access may be limited, and the amount of soil rendered unfit for use as structural fill may be greatly increased if drainage efforts are not accomplished in a timely manner.

4.3.3 Clearing, Grubbing and Stripping

In preparation for grading, we recommend removal of all existing surficial vegetation. Following clearing of surficial vegetation, organic-rich topsoil (soils containing more than 4 percent organic material by weight) should be stripped from any locations that are to receive structural fill, footing and slab subgrades, and utility trenches. These materials should be wasted away from the improvement area or removed from the project site. Based on our observations, the thickness of surficial organic material requiring removal may be on the order of about 3 inches, but variation in this depth should be expected.

4.3.4 Existing Fill Removal

Site preparation is recommended to include selective removal of existing undocumented fill material containing substantial organics or deleterious debris and underlying relic organic topsoil. Please note that the nature of fill is such that its extent, composition, and thickness can vary over relatively short distances. Fill depth and composition was observed to vary across the site from about 1-½ to 3 feet. These materials should be evaluated during construction and removed as necessary under the observation of ZGA representative. Our representative will identify unsuitable materials that should be removed or may be re-used as structural fill. The resultant excavations should be backfilled in accordance with the subsequent recommendations for structural fill placement and compaction.

4.3.5 Subgrade Preparation and Protection

Once site preparation is complete, all areas that are at design subgrade elevation or areas that will receive new structural fill should be compacted to a firm and unyielding condition if possible. In the event the exposed subgrade becomes unstable, yielding, or unable to be compacted due to high moisture conditions, we recommend that the materials be over-excavated and replaced with structural fill compacted in accordance with the recommendations detailed in Section 4.4 – *Structural Fill Materials, Placement, and Compaction*.

4.3.6 Freezing Conditions

If earthwork takes place during freezing conditions, all exposed subgrades should be allowed to thaw and then be compacted prior to placing subsequent lifts of structural fill, pouring concrete, or paving. Alternatively, the frozen material could be stripped from the subgrade to expose unfrozen soil. The frozen soil should not be re-used as structural fill until allowed to thaw and adjusted to the proper moisture content; please note that this may not be feasible during winter months.

4.4 Structural Fill Materials, Placement, and Compaction

Structural fill includes any material placed below foundations, slabs, and pavement sections, within utility trenches, and behind retaining walls. Prior to the placement of structural fill, all surfaces to receive fill should be prepared as previously recommended in the *Site Preparation* section.

4.4.1 Re-use of Site Soils as Structural Fill

The suitability for re-use of site soils as structural fill depends on the composition and moisture content of the soil. Soils observed at the boring locations generally consisted of sand with a relatively high silt and varying gravel content. As the fines content increases, soil becomes increasingly sensitive to small changes in moisture content. Soils containing more than about 5 percent fines cannot be consistently compacted to the appropriate levels when the moisture content is more than approximately 2 percent above or below the optimum moisture content (per ASTM D1557). The optimum moisture content is the moisture content which results in the greatest compacted dry density with a specified compactive effort.

During dry weather, we expect site soils will be suitable for re-use as structural fill. However, some moisture conditioning consisting of drying may be required if grading occurs during wet weather or under wet site conditions. Due to the relatively high fines content of the glacial till soils, these soils should be considered highly moisture sensitive. From a compositional standpoint, the non-organic site soils are considered adequate for use as structural fill. However, the feasibility of using the on-site soils as structural fill depends greatly on the weather conditions at the time of placement and compaction. If site soils are planned for re-use, they should be protected from an increase in moisture content during periods of wet weather. At a minimum, we recommend stockpiles of excavated material to be used as structural fill be covered with plastic sheeting if rain is forecast.

We recommend that site soils used as structural fill have less than 3 percent organics on a dry weight basis, have no woody debris greater than ½-inch in diameter, and contain no other deleterious materials. We recommend that all pieces of organic material greater than ½-inch in diameter be picked out of the fill before it is placed and compacted. Deleterious debris includes waste building materials, organics, trash, and asphalt and, if encountered, it should be removed from the soil prior to its re-use as structural fill.

4.4.2 Imported Structural Fill

Imported structural fill may be required. For general purposes, we recommend imported fill meet the following requirements:

- During Extended Periods of Dry Weather: Common Borrow Options 1 or 2 per Section 9-03.14(3) of the 2022 WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* (Publication M41-10). The on-site soil would be classified as Common Borrow.

- During Wet Weather: Gravel Borrow per Section 9-03.14(1) of the WSDOT Standard Specs.

It should be noted that Common Borrow typically contains a significant fraction of fines (silt and clay). During wet weather, Common Borrow may likely become too wet for re-use as structural fill.

4.4.3 Moisture Content

The suitability of soil for use as structural fill will depend on the prevailing weather at the time of construction, the moisture content of the soil, and the fines content (that portion passing the U.S. No. 200 sieve) of the soil. Imported fills should be delivered to the site at a moisture content within ±2 percent of optimum. This is important as there will be minimal room on this site to moisture condition imported fill materials should they arrive at the site wet or dry of optimum.

4.4.4 Fill Placement and Compaction

We recommend that structural fill be placed in horizontal lifts of a thickness that can be compacted to the recommended levels with the equipment available (typically 8 to 12 inches). Our recommendations for soil compaction are summarized in the following table. We recommend that a ZGA representative be present during grading so that an adequate number of density tests may be conducted as structural fill placement occurs.

Recommend Soil Compaction Levels	
Location	**Minimum Percent Compaction***
Fill below foundations	95
Fill below concrete slabs exclusive of above the below-grade storage tank	95
Below-grade storage tank excavation cavity	Per tank manufacturer recommendation
Cast-in-place retaining walls	92 - 95
Upper two feet of utility trench backfill	95
Utility trenches below two feet	90
Landscape areas	90 or as required by Parks
* ASTM D1557 Modified Proctor Maximum Dry Density	

4.5 Temporary and Permanent Slopes

Temporary excavations are expected to construct the below-grade storage tank, pump house foundations, and for installation of underground utilities. Temporary excavation slope stability is a function of many factors, including:

- The presence and abundance of groundwater;
- The type and density of the various soil strata;
- The depth of cut;
- Surcharge loadings adjacent to the excavation; and
- The length of time the excavation remains open.

It is exceedingly difficult under the variable circumstances to pre-establish a safe and “maintenance-free” temporary cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe temporary slope configurations since the contractor is continuously at the job site, able to observe the nature and condition of the cut slopes, and able to monitor the subsurface materials and groundwater conditions encountered. Unsupported vertical slopes or cuts deeper than 4 feet are not recommended. The cuts should be adequately sloped, shored, or supported to prevent injury to personnel from local sloughing and spalling. The excavation should conform to applicable Federal, State, and Local regulations.

According to Chapter 296-155 of the Washington Administrative Code (WAC), the contractor should make a determination of excavation side slopes based on classification of soils encountered at the time of excavation. Temporary cuts may need to be constructed at flatter angles based upon the soil moisture, soil density, and groundwater conditions at the time of construction. Adjustments to the slope angles should be determined by the contractor at that time.

Installation of the below-grade storage tank will require a substantial excavation. The existing fill and medium dense weathered glacial till (ESU-1 and ESU-2), which extended to depths of about 1.5 to 5 feet, respectively, meet the criteria for Type C soils per Chapter 296-155 WAC, *Part N, Excavation, Trenching, and Shoring.* Temporary excavation slopes as deep as 20 feet in these soils may generally be planned with an inclination no steeper than 1.5H:1V (Horizontal: Vertical). The underlying dense to very dense native glacial till (ESU-3) meets the criteria for Type A soil and temporary excavations as deep as 20 feet may be designed with an inclination no steeper than 0.75H:1V. The locations of existing facilities, such as the picnic shelter, relative to the extent of temporary excavations should be taken into account during the design process. The feasibility of these temporary excavation slope inclinations should be verified during excavation, and it should be recognized that conditions disclosed during excavation may warrant the use of shallower temporary excavation slope inclinations. Otherwise, temporary excavation shoring may be required.

We recommend designing permanent cut or fill slopes constructed in native or properly compacted fill soils at a 2H:1V inclination or flatter. All permanent cut and fill slopes should be adequately protected from erosion both temporarily and permanently.

4.6 Shallow Foundation Recommendations

The project will include construction of a small storage building and possibly low cast-in-place concrete retaining walls. Based on the soil conditions and our analyses, conventional shallow spread continuous and column footings appear feasible provided that the foundation subgrades are prepared in accordance with this report. Recommendations for shallow spread footings are provided below.

4.6.1 Seismic Criteria Summary

Seismic Criteria Summary	
Code Criteria	**Site Classification**
2021 International Building Code (IBC) [1]	C [2]
Latitude	47.442582
Longitude	-122.11884
S_S Spectral Acceleration for a Short Period	1.5g (Site Class C)
S_1 Spectral Acceleration for a 1-Second Period	0.5g (site Class C)
S_{MS} Maximum considered spectral response acceleration for a Short Period	1.66g (Site Class C)
S_{M1} Maximum considered spectral response acceleration for a 1-Second Period	0.68g (Site Class C)
S_{DS} Five-percent damped design spectral response acceleration for a Short Period	1.11g (Site Class C)
S_{D1} Five-percent damped design spectral response acceleration for a 1-Second Period	0.45g (Site Class C)
V_{S30} Average soil/bedrock shear wave velocity in the upper 100 feet in ft./sec. (estimated)	1,450 to 2,100 (Site Class C)

1. In general accordance with the *2021 International Building Code,* Section 1613.3.2 and *ASCE 7-22*, Chapter 20. IBC Site Class is based on the average characteristics of the upper 100 feet of the subsurface profile.
2. The explorations completed for this study extended to a maximum depth of about 12.5 feet below grade. ZGA therefore determined the Site Class assuming that very dense glacially consolidated soils and sedimentary bedrock extend to 100 feet as suggested by logs of nearby explorations and published geologic maps for the project area.

4.6.2 Foundation Subgrade Preparation

Finished floor and foundation subgrade elevations for the proposed storage building or retaining wall foundation subgrade elevations were not available as of the date of this report. We recommend that footings bear directly on at least medium dense to dense ESU-2 soils or on properly compacted structural fill extending down to ESU-2 soils. We recommend preparing footing subgrades in accordance with the *Subgrade Preparation* section. For areas where loose ESU-1 soils are over-excavated and replaced we recommend extending the excavation width beyond the edges of the footings a distance equal to the excavation depth. Replacement structural fill should be placed and compacted in accordance with the recommendations outlined in the *Fill placement and Compaction* section.

4.6.3 Allowable Bearing Pressure

Continuous and isolated column footings bearing on footing subgrades prepared as recommended above may be designed for a maximum allowable net bearing capacity of 3,000 psf. A one-third increase of the bearing pressure may be used for short-term dynamic loads such as wind and seismic forces. We recommend providing ZGA the opportunity to verify foundation subgrade conditions prior to form and reinforcing placement. We estimate that total settlement of foundations constructed as described herein will be less than 1 inch. Differential settlement is estimated to be about ½ inch or less in 40 feet.

4.6.4 Shallow Foundation Depth and Width

For frost protection, we recommend the bottom of all exterior footings bear at least 18 inches below the lowest adjacent outside grade, whereas the bottoms of interior footings should bear at least 12 inches below the surrounding slab surface level. We recommend that all continuous wall and isolated column footings be at least 12 and 24 inches wide, respectively.

4.6.5 Resistance to Lateral Loads

Lateral loads can be resisted by a combination of base friction and passive earth pressures acting on the face of footing elements. For footings founded as recommended above, we recommend using an ultimate base friction coefficient of 0.5. For footings backfilled with structural fill placed in accordance with this report, we recommend using an ultimate passive earth pressure value of 400 pounds per cubic foot (pcf). We recommend passive resistance be neglected within the upper 18 inches of embedment. The above values do not include safety factors. Appropriate safety factors or resistance factors should be used for design.

4.7 Below Grade Storage Tank

4.7.1 General

We understand that the project will include installation of a below-grade water storage tank that may be up to as deep as 15 feet below-grade. Dense to very dense ESU-3 soils were encountered between about 1-1/2 to 5 feet below grade at the boring B-4 location. Although boring B-4 was terminated at a depth of about 7-1/2 feet under refusal in very dense coarse glacial till soil, nearby boring B-1 disclosed very dense glacial till soils to just below the likely storage tank excavation lower elevation of about 561-1/2 feet. Therefore, we anticipate the below-grade storage tank will bear directly on dense to very dense ESU-3 soils.

4.7.2 Buoyancy Considerations

Given the presence of perched groundwater within the ESU-2 and ESU-3 soils when the borings were advanced, seasonal groundwater seepage and stormwater migration into the backfilled storage tank excavation should be expected. The contractor should be prepared to dewater excavations to the extent necessary to allow for installation of the tank foundation, tank, and backfill material.

We anticipate that the tank will likely be subject to buoyancy forces if the excavation is not equipped with a permanent drain. Potential buoyant forces acting on the tank may be calculated by multiplying the volume of the portion of the tank below the water table (in cubic feet) by 62.4 pcf. Buoyant forces may be resisted by a combination of the weight of the tank, the weight of the concrete slab or foundation to which it is attached, by pre-cast or cast-in-place concrete anchors installed in the excavation, and by the weight of backfill placed above the tank and above installing flanges on the tank base (if so equipped). Assuming that the tank cavity is primarily backfilled with pea gravel, we recommend considering a unit density of 100 pounds per cubic foot (pcf) for backfill above the excavation water elevation, and 38 pcf for backfill below the water level. For design purposes, we recommend considering that water within the tank excavation may be as high as approximately 573.5 feet, or about 3 feet below existing grade.

4.8 On-Grade Concrete Slabs

We anticipate that concrete slabs-on-grade will be used below the storage/maintenance building and as the base of the splash pad itself. The following sections summarize our recommendations for on-grade concrete slabs.

4.8.1 Subgrade Preparation

After removal of organic material and other items noted in the *Site Preparation* section of this report, we recommend that at least the upper 12 inches of material below the slab base be moisture conditioned (if needed) and compacted to a firm and unyielding condition and to a minimum 95 percent of the modified Proctor maximum dry density per ASTM D1557. If the slab is constructed above structural fill material, we recommend compacting the fill to at least 95 percent density per ASTM D 1557.

4.8.2 Capillary Break

To provide a capillary break and uniform slab bearing surface, we recommend that the maintenance building be underlain by a minimum 4-inch-thick layer of compacted crushed rock meeting the requirements of WSDOT Specification 9-03.9(3), Crushed Surfacing Top Course, with the modification of a maximum of 7 percent passing the U.S. No. 200 sieve. Alternatively, clean angular gravel such as No. 7 aggregate per WSDOT 9-03.1(4)C could be used for this purpose. Alternative capillary break materials should be submitted to the geotechnical engineer for review and approval before use. We recommend that the need for a capillary break below the splash pad slab be evaluated in light of the recommendations of the manufacturers that provide the splash pad piping and drainage hardware.

4.9 Retaining Walls

The site has a very gentle slope downward to the north. Consequently, it may be necessary to construct retaining walls at either the north or south sides of the splash pad (or both) to support relatively low cuts or fills needed to prepare a level surface for the splash pad. We anticipate that walls will be less than 6 feet tall. At the time this report was prepared, the types of wall had yet to be determined. We anticipate that segmental concrete block walls employing geogrid reinforced backfill would not be practical given the amount of plumbing that will be installed below the splash pad. As such, we have provided recommendations and design values below for cast-in-place (CIP) walls that will not employ reinforced backfill material. The foundation recommendations already provided in Section 4.5 are applicable to new retaining walls.

Retaining Wall Lateral Earth Pressure Recommendations	
Parameter	**Recommended Value**
Granular Wall Backfill Total Unit Weight, γ (pcf)	130
Friction Angle, Φ	36
Active Lateral Earth Pressure Coefficient, K_a (level backfill)	0.26
At-Rest Lateral Earth Pressure Coefficient, K_0 (level backfill)	0.41

The above-recommended lateral earth pressures assume that adequate drainage measures are provided to limit the potential for buildup of hydrostatic pressures. All backfilled walls should include a drainage aggregate zone extending a minimum of two feet from the back of wall for the full height of the wall and wide enough at the base of the wall to allow seepage to flow to the footing drain. The drainage aggregate should consist of material meeting the requirements of WSDOT 9-03.12(2), Gravel Backfill for Walls. A minimum 4-inch diameter, rigid, perforated PVC or HDPE drainpipe should be provided at the base of backfilled walls to collect and direct subsurface water to an appropriate discharge point. We recommend placing a non-woven geotextile, such as Mirafi 140N, or equivalent, around the free draining backfill material.

4.10 Stormwater Management Considerations

Since a stormwater infiltration feature location had not been determined when the field exploration took place, boring B-2 was advanced in the general vicinity that an infiltration feature may likely be located in the lawn immediately south of the sidewalk to the north of the playground and picnic shelter (and downslope of the splash pad). Low permeability glacially consolidated soils were encountered in boring B-2 at a depth of approximately 1.5 feet, perched groundwater was encountered at approximately 2.5 feet, and auger refusal was encountered at approximately 7 feet. These observations indicate a shallow perched groundwater table and a shallow low permeability soil horizon. Glacial till is typically considered a hydraulically restrictive layer not suitable for conventional stormwater infiltration. As such, we do not expect stormwater infiltration will be feasible at this site.

Per Section 5.2.1 *General Requirements for Infiltration Facilities* (Page 5-44 of the 2021 King County *Surface Water Design Manual*), a geotechnical professional is required to evaluate site conditions and provide *"… a written opinion… that sufficient permeable soil exists at the proposed facility location to allow construction of a property functioning infiltration facility".* Based on the conditions observed at the boring locations (the presence of dense to very dense lodgement glacial till and shallow perched groundwater) and the results of laboratory testing that indicate a relatively high fines content of the glacial till soils, it is our professional opinion that the site lacks the soil and groundwater conditions that would *"…allow construction of a property functioning infiltration facility"* per the *Manual.* Consequently, we offer the following:

- We recommend that the two field infiltration test pits included in our scope of services not be completed and that Otak and Parks consider stormwater infiltration infeasible for the proposed splash pad project.

- We recommend that Otak and Parks consider others means of stormwater management, such as directing surface water from the splash pad to the existing storm sewer system or directing water to a nearby location which would support dispersion.

4.11 Trail Recommendations

A new section of Hot Mix Asphalt (HMA) trail will be constructed to access the splash pad features. The trail will accommodate pedestrians as well as park service vehicles. We recommend a pavement section consisting of 2 inches of HMA above 4 inches of crushed surfacing. Our recommendations summarized below reference WSDOT Publication M41-10.

Subgrade Preparation and Compaction: We recommend constructing the new trail above at least medium dense and non-yielding native soil or structural fill compacted to at least 95 percent density.

HMA: We recommend that HMA conform to Section 9-02.1(4) for PG 58S-22 or PG 58H-22 *Performance Graded Asphalt Binder* as presented in the WSDOT Standard Specifications. We also recommend that the gradation of the asphalt aggregate conform to the aggregate gradation control points for a ½-inch mix as presented in WSDOT Specification 9-03.8(6), *HMA Proportions of Materials.*

Crushed Surfacing Top Course: We recommend that *Crushed Surfacing Top Course* (CSTC) conform to Section 9-03.9(3) of the WSDOT Standard Specifications.

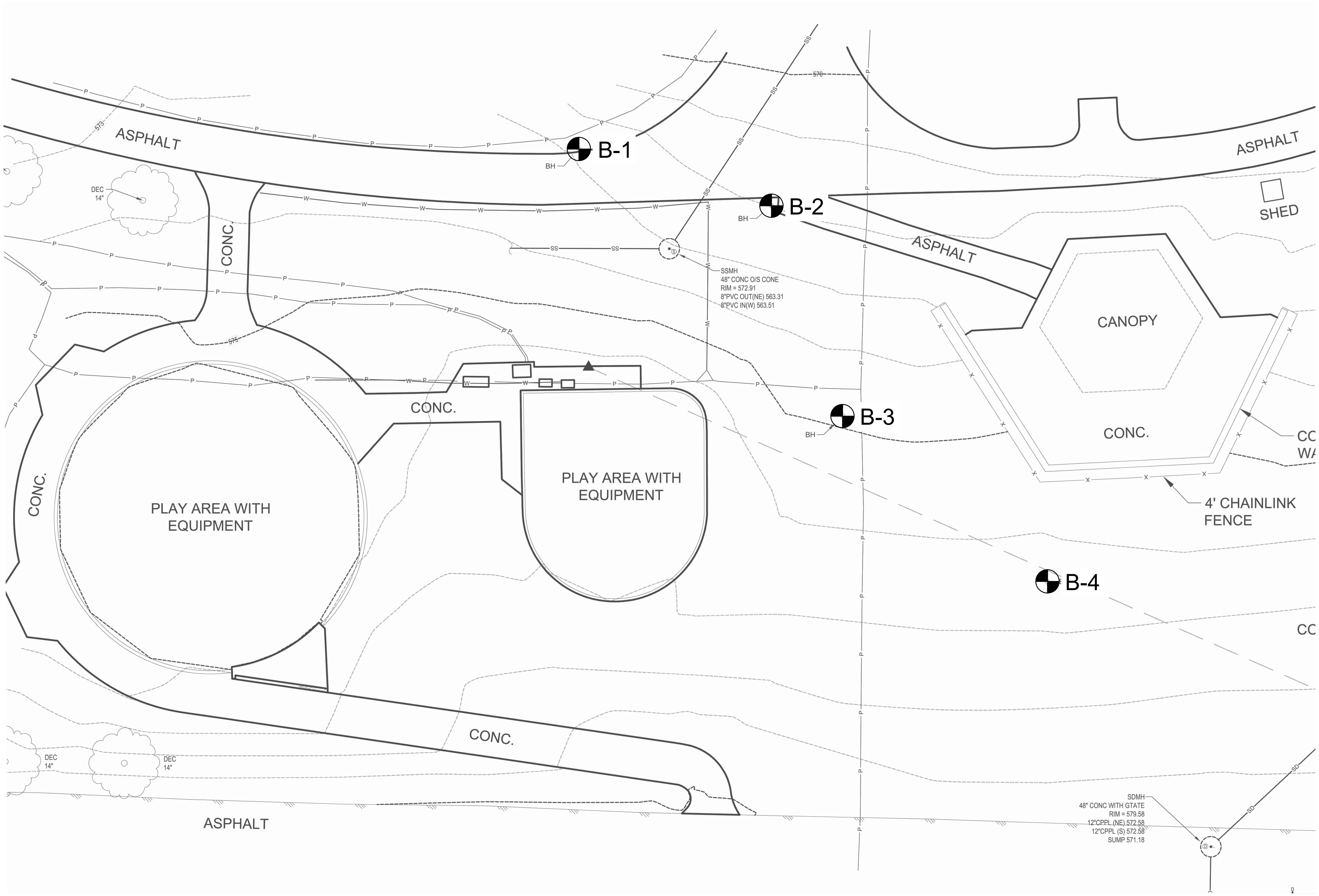
Compaction and Paving: All base material should be compacted in accordance with WSDOT Specification 2-03.3(14)C *Compacting Earth Embankments – Method C*. We recommend that HMA be compacted to a minimum of 92 percent and a maximum of 96 percent of the theoretical maximum density. Placement and compaction of HMA should conform to the requirements of Section 5-04 of the WSDOT Standard Specifications. This includes weather limitations as specified in Section 5-04.3(1) and maximum nominal asphalt lift thickness as specified in Section 5-04.3(7), Table 6.

5.0 CLOSURE

The analysis and recommendations presented in this report are based, in part, on the explorations completed for this study. The number, location, and depth of the explorations were completed within the constraints of budget and site access so as to yield the information to formulate our recommendations. Project plans were in the preliminary stage at the time this report was prepared. We therefore recommend that ZGA be provided an opportunity to review the final plans and specifications when they become available in order to assess that the recommendations and design considerations presented in this report have been properly interpreted and implemented into the project design.

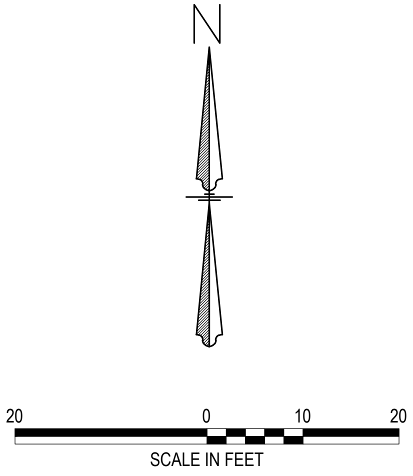
The performance of earthwork, structural fill, foundations, and temporary shoring depend greatly on proper site preparation and construction procedures. We recommend that Zipper Geo Associates, LLC be retained to provide geotechnical engineering services during the earthwork-related construction phases of the project. If variations in subsurface conditions are observed at that time, a qualified geotechnical engineer could provide additional geotechnical recommendations to the contractor and design team in a timely manner as the project construction progresses.

This report has been prepared for the exclusive use of Otak, and their agents, for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made. Site safety and excavation support are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Zipper Geo Associates, LLC reviews the changes and either verifies or modifies the conclusions of this report in writing.



LEGEND

◑ B-1	BORING NUMBER AND APPROXIMATE LOCATION



PETROVITSKY PARK SPLASH PAD
16400 SE PETROVITSKY ROAD
Renton, King County, Washington

SITE AND EXPLORATION PLAN

DATE: MAY 2024
Job No. 2781.01

Zipper Geo Associates, LLC
19019 36th Ave. W.,Suite E
Lynnwood, WA

FIGURE
1
SHT.1of 1

REFERENCE: PETROVITSKY PARK SPLASH PAD/COOLING CENTER, EXISTING CONDITIONS, SHEET G2, PREPARED BY KING COUNTY, DELIVERED APRIL 17, 2024

APPENDIX A
FIELD EXPLORATION PROCEDURES AND LOGS

FIELD EXPLORATION PROCEDURES AND LOGS

Our field exploration program for this project included completing a visual reconnaissance of the site and advancing four borings (B-1 through B-4). The approximate exploration locations are presented on Figure 1, the *Site and Exploration Plan*. Exploration locations were determined in the field using steel and fiberglass tapes by measuring distances from existing site features. The ground surface elevation at each exploration location was interpolated from the topographic survey *Petrovitsky Park Existing Conditions*, provided to us by Otak on 17 April 2024. As such, the exploration locations and elevations should be considered accurate to the degree implied by the measurement methods. The following sections describe our procedures associated with the explorations. Descriptive logs of the explorations are enclosed in this appendix.

Boring Procedures

The borings were advanced using a track-mounted drill rig operated by an independent drilling company (Geologic Drill Explorations, Inc.) working under subcontract to ZGA. The borings were advanced using hollow stem auger drilling methods. A ZGA geologist continuously observed the borings, logged the subsurface conditions encountered, and obtained representative soil samples. All samples were stored in moisture-tight containers and transported to our laboratory for further evaluation and testing. Samples were generally obtained by means of the Standard Penetration Test at 2.5-foot intervals throughout the drilling operation, although bulk samples were collected from approximately the upper 2 feet of borings B-1 and B-2 while using a posthole digger. B-2 was sampled continuously.

The Standard Penetration Test (ASTM D 1586) procedure consists of driving a standard 2-inch outside diameter steel split spoon sampler 18 inches into the soil with a 140-pound hammer free falling 30 inches. The number of blows required to drive the sampler through each 6-inch interval is recorded, and the total number of blows struck during the final 12 inches is recorded as the Standard Penetration Resistance, or "blow count" (N value). If a total of 50 blows are struck within any 6-inch interval, the driving is stopped and the blow count is recorded as 50 blows for the actual penetration distance. The resulting Standard Penetration Resistance values indicate the relative density of granular soils and the relative consistency of cohesive soils.

The enclosed boring logs describe the vertical sequence of soils and materials encountered in each boring, based primarily upon our field classifications. Where a soil contact was observed to be gradational, our logs indicate the average contact depth. Where a soil type changed between sample intervals, we inferred the contact depth. Our logs also graphically indicate the blow count, sample type, sample number, and approximate depth of each soil sample obtained from the boring. If groundwater was encountered in a borehole, the approximate groundwater depth and date of observation are depicted on the log.

EXPLANATION OF EXPLORATION LOGS

SOIL DESCRIPTION: Soil descriptions presented on the borings logs are based on visual observations. Soil descriptions include density (coarse-grained soils) or consistency (fine-grained soils), moisture, color, major soil type, and grain size modifiers and should not be interpreted to suggest laboratory or field testing unless indicated on the logs. Soil descriptions include the following: Density/consistency, moisture, color, grain size modifier (adjective implying 31-49 percent), major soil type (CAPITALIZED implying 50+ percent), minor grain size modifier (some implying 6-12 percent, with implying 13-30 percent, and trace implying 0-5 percent), descriptive modifiers (i.e. roots, fill debris, cemented, etc.), and interpreted general geologic description. Descriptions may also include comments describing geologic properties such as dilatancy, toughness, structure, plasticity, and angularity of coarse-grained particles. Additional information regarding geologic properties is presented in the report text as applicable.

DENSITY/CONSISTENCY: Soil density/consistency in borings is related to the blow count number in blows per foot using the sampling method indicated on the logs. Soil density/consistency in test pits is related to a “Field Test” as described below. Soil consistency in test pits or borings may be augmented by field Torvane or Pocket Penetrometer testing.

Coarse-Grained Soils

Density Descriptor	SPT (# blows/ft)	Field Test
Very Loose	0 – 4	Easily penetrated with ½ -inch steel rod pushed by hand.
Loose	5 – 10	Difficult to penetrate with ½ - inch steel rod pushed by hand.
Medium Dense	11 – 30	Easily penetrated a foot with ½-inch steel rod driven with 5-lb hammer.
Dense	31 – 50	Difficult to penetrate a foot with ½-inch steel rod driven with 5-lb hammer.
Very Dense	>50	Penetrated only a few inches with ½-inch steel rod driven with 5-lb hammer.

Fine-Grained Soils

Consistency Descriptor	SPT (# blows/ft)	Torvane	Pocket Penetrometer	Field Test
		Undrained shear strength (tsf)	Unconfined Compressive Strength (tsf)	
Very Soft	0 – 2	<0.125	<0.25	Easily penetrates several inches by thumb.
Soft	3 – 4	0.125 – 0.25	0.25 – 0.5	Easily penetrates one inch by thumb.
Medium Stiff	5 – 8	0.25 – 0.5	0.5 – 1.0	Penetrated over ½ inch by thumb with moderate effort.
Stiff	9 – 15	0.5 – 1.0	1.0 – 2.0	Indented by thumb but penetrated only with great effort.
Very Stiff	16 – 30	1.0 – 2.0	2.0 – 4.0	Readily indented by thumbnail.
Hard	>30	>2.0	>4.0	Indented by thumbnail with difficult effort.

MOISTURE

Descriptor	Field Test
Dry	Absence of moisture, dusty, dry to the touch.
Damp	Too low to achieve compaction
Moist	Appears near optimum moisture content for compaction
Wet	Too wet to achieve compaction
Saturated	Below the groundwater table, visible free moisture.

MAJOR SOIL TYPE: Coarse-grained soils with over 50% of the material retained on the U.S. No. 200 sieve. Coarse-grained soils include boulders, cobbles, gravels and sands. Fine-grained soils with over 50% of the material passing the U.S. No. 200 sieve. Fine-grained soils include silts and clays.

GRAIN SIZE

Descriptor	Sieve Size	Grain Size
Boulder	>12”	>12”
Cobble	3 – 12”	3 – 12”
Gravel	3” – #4	3” – 0.19”
Sand	>#4 – #200	<0.19” – >0.0029”
Silt/Clay	Passing #200	<0.0029”

GRAIN SIZE MODIFIERS

Descriptor	Approximate Percentage
Trace	0 – 5
Some	6 – 12
With	13 – 30
Adjective (silty, clayey, sandy, gravelly)	31 – 50

BORING LOG: B-1

B-1

Logged By: KRN
Location: See Figure 1
Elevation: 573 ft
Drill Date: 2024-02-08

Location: 16400 SE Petrovitsky Rd
Drilling Company: Geologic Drill Partners
Drill Rig: Mini Track Rig
Drilling Method: Auger (Hollow-Stem)
Reviewer: DCW

Borehole Diameter: 6
Hammer Type: Cathead

Depth (FT)	Lithologic Description	Symbol	Samples No.	Samples Type	Standard Penetration Test (SPT) 0 10 20 30 40 50 60	(SPT)	Moisture (%)	Fines (%)	Testing
	Ground Surface EL 573 ft								
0	Approximately 2 to 3 inches of grass sod and topsoil over medium dense, moist, dark brown, silty SAND, with fine roots (Fill) ESU-1		S-1	▽		17	17		
	1.5 ft EL 571.5 ft								
	Dense, moist, light brown to light gray, silty SAND to sandy SILT, some gravel, slight mottling/iron staining on small 1-inch silty lens (Weathered Till) ESU-2		S-2	▽		34	12		
	4 ft EL 569 ft								
5	Very dense, moist, light brown to brown, gravelly SAND, with silt (Unweathered Till) ESU-3		S-3	▽	●	50/6	8		
	7 ft EL 566 ft								
▽ 08 Feb 2024	Very dense, wet, light brown to brown, SAND, with silt and gravel		S-4	▽	●	50/5	13	22	GSA
10	Very dense, moist, gray, gravelly SAND, with silt		S-5	▽	●	50/6	8		
	11.5 ft EL 561.5 ft								
	Very dense, wet, gray, sandy SILT, with gravel		S-6	▽	●	50/3	12		
	12.5 ft EL 560.5 ft								
13	Boring terminated at approximately 12.5 feet due to auger refusal. Perched groundwater observed at approximately 7.5 and 11.5 feet at time of drilling.								

Depth (FT): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20

ZipperGeo
Geoprofessional Consultants
19019 36th Ave. W, Suite E
Lynnwood, WA 98036

PROJECT TITLE: Petrovitsky Park Splash Pad
PROJECT NO.: 2781.01
CLIENT: Otak
SHEET: 1 of 1

BORING LOG: B-2

B-2

Logged By: KRN
Location: See Figure 1
Elevation: 572 ft
Drill Date: 2024-02-08

Location: 16400 SE Petrovitsky Rd
Drilling Company: Geologic Drill Partners
Drill Rig: Mini Track Rig
Drilling Method: Auger (Hollow-Stem)
Reviewer: DCW

Borehole Diameter: 6
Hammer Type: Cathead

Depth (FT)	Lithologic Description	Symbol	Samples No.	Samples Type	Standard Penetration Test (SPT) 0 10 20 30 40 50 60	(SPT)	Moisture (%)	Fines (%)	Testing
	Ground Surface EL 572 ft								
0–1	Approximately 2 to 3 inches of grass sod and topsoil over loose to medium dense, wet, brown gravelly silty SAND (Fill) ESU-1		S-1	▽			12		
	1.5 ft EL 570.5 ft					60			
2	▽ 08 Feb 2024 Dense to very dense, wet, light brown, silty SAND (Unweathered Till) ESU-3		S-2	▽			11		
3						35			
4			S-3	▽			11	20	GSA
						74			
5			S-4	▽			8.5	17	GSA
6					●	50/5			
			S-5	▽			8		
7	7 ft EL 565 ft								
8	Boring terminated at approximately 7 feet due to auger refusal. Perched groundwater observed at approximately 2.5 feet at time of drilling.								
9									
10									
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									

ZipperGeo
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PROJECT TITLE: Petrovitsky Park Splash Pad
PROJECT NO.: 2781.01
CLIENT: Otak
SHEET: 1 of 1

RSLog / 1Zipper Geo Geotechnical Soil Log (no piezometer) / zipper-geo-associates / admin / May 06, 2024 10:51 AM

BORING LOG: B-3

B-3

Logged By: KRN
Location: See Figure 1
Elevation: 575 ft
Drill Date: 2024-02-08

Location: 16400 SE Petrovitsky Rd
Drilling Company: Geologic Drill Partners
Drill Rig: Mini Track Rig
Drilling Method: Auger (Hollow-Stem)
Reviewer: DCW

Borehole Diameter: 6
Hammer Type: Cathead

Depth (FT)	Lithologic Description	Symbol	Samples No.	Samples Type	Standard Penetration Test (SPT) 0 10 20 30 40 50 60	(SPT)	Moisture (%)	Fines (%)	Testing
	Ground Surface EL 575 ft								
0	Approximately 2 to 3 inches of grass sod and topsoil over loose, wet, dark brown, silty SAND, abundant fine roots (Fill) ESU-1		S-1	▽		10	31		
1.5	1.5 ft EL 573.5 ft								
	08 Feb 2024 ▽								
	Medium dense, wet to saturated, light brown, silty SAND, some gravel, some iron staining and mottling present (Weathered Till) ESU-2		S-2	▽		25	17	38	GSA
5	5 ft EL 570 ft								
	Very dense, saturated, light brown, gravelly silty SAND (Unweathered Till) ESU-3		S-3	▽		34	12		
			S-4	▽	●	75/11	9		
			S-5	▽	●	50/4			
10.5	10.5 ft EL 564.5 ft								
	Boring terminated at approximately 10.5 feet due to auger refusal. Perched groundwater observed at approximately 2.5 feet at time of drilling.								

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SHEET: 1 of 1

BORING LOG: B-4

B-4

Logged By: KRN
Location: See Figure 1
Elevation: 576.5 ft
Drill Date: 2024-02-08

Location: 16400 SE Petrovitsky Rd
Drilling Company: Geologic Drill Partners
Drill Rig: Mini Track Rig
Drilling Method: Auger (Hollow-Stem)
Reviewer: DCW

Borehole Diameter: 6
Hammer Type: Cathead

Ground Surface EL 576.5 ft

Depth (FT)	Lithologic Description	Samples No.	Samples Type	(SPT)	Moisture (%)	Fines (%)	Testing
0	Approximately 2 to 3 inches of grass sod and topsoil over loose, wet, dark brown, silty SAND to sandy SILT, with organics and woody fibers, roots (Fill) ESU-1	S-1	▽	6	35		
3 ft EL 573.5 ft	Medium dense, moist, gray to light brown with mottling, silty SAND with gravel (Weathered Till) ESU-2	S-2	▽	26	15		
5 ft EL 571.5 ft	Very dense, moist to wet, light brown to gray, gravelly silty SAND (Unweathered Till) ESU-3	S-3	▽	65	11		
		S-4	▽	● 50/5	9		
7.5 ft EL 569 ft	Boring terminated at approximately 7.5 feet due to auger refusal. Perched groundwater observed at approximately 7 feet at time of drilling.						

▽ 08 Feb 2024 (at 7 ft)

Standard Penetration Test (SPT): 0 10 20 30 40 50 60

Depth (FT): 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20

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APPENDIX B

LABORATORY TESTING PROCEDURES AND RESULTS

LABORATORY PROCEDURES AND RESULTS

A series of laboratory tests were performed during the course of this study to evaluate the index and geotechnical engineering properties of the subsurface soils. Descriptions of the types of tests performed are given below.

Visual Classification

Samples recovered from the exploration locations were visually classified in the field during the exploration program. Representative portions of the samples were carefully packaged in moisture tight containers and transported to our laboratory where the field classifications were verified or modified as required. Visual classification was generally done in accordance with ASTM D 2488. Visual soil classification includes evaluation of color, relative moisture content, soil type based upon grain size, and accessory soil types included in the sample. Soil classifications are presented on the exploration logs in Appendix A.

Moisture Content Determinations

Moisture content determinations were performed on representative samples obtained from the explorations in order to aid in identification and correlation of soil types. The determinations were made in general accordance with the test procedures described in ASTM D 2216. The results are shown on the exploration logs in Appendix A.

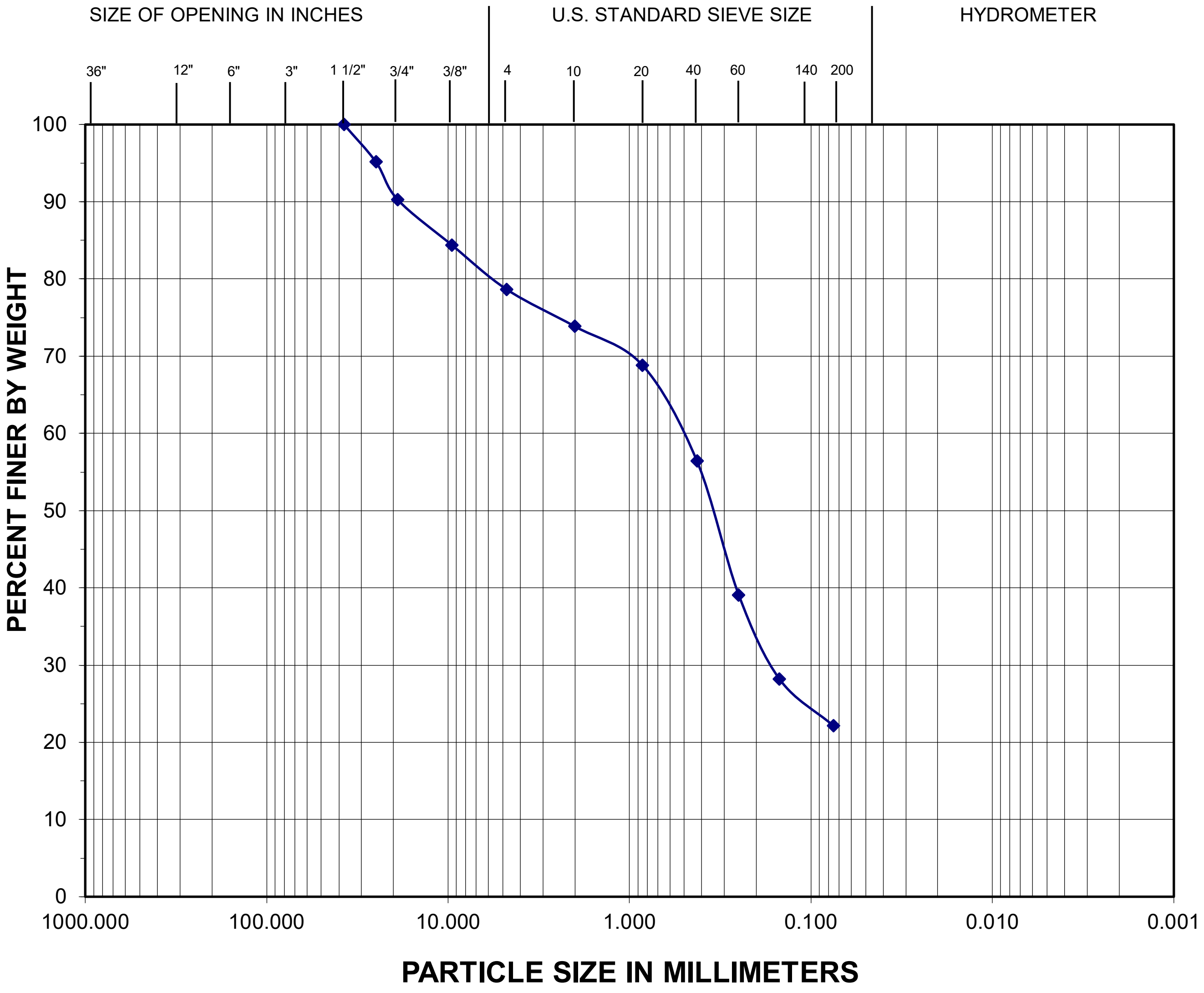
Grain Size Analysis

A grain size analysis indicates the range in diameter of soil particles included in a particular sample. Grain size analyses were performed on representative samples in general accordance with ASTM D 6913. The results of the grain size determinations for the samples were used in classification of the soils, and are presented in this appendix.

GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-1	S-4	7.5	12.7	22.1	SAND, with silt and gravel

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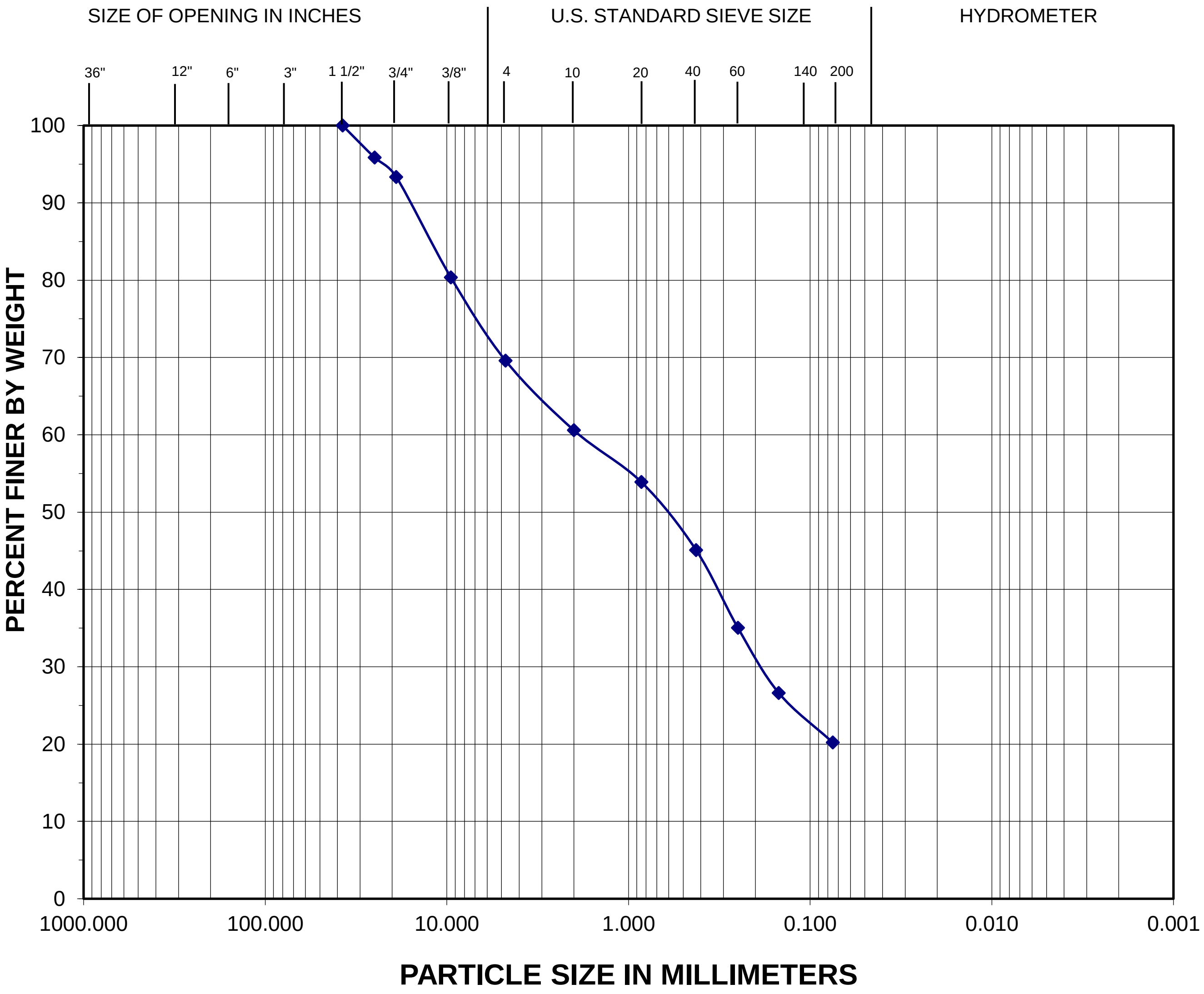
GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-2	S-3	3	10.7	20.2	SAND, with gravel and silt

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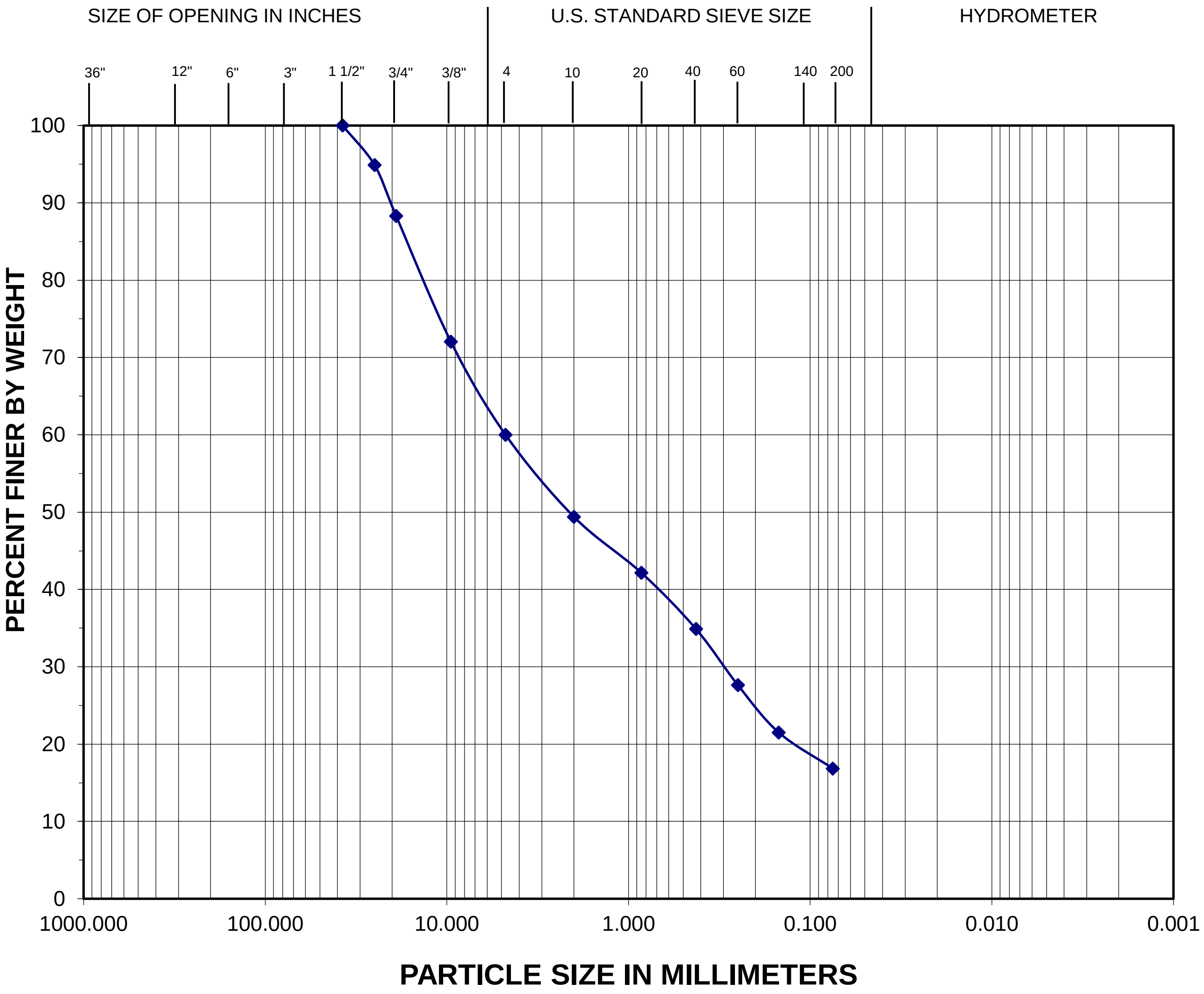
GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-2	S-4	4.5	8.5	16.8	Gravelly SAND, with silt

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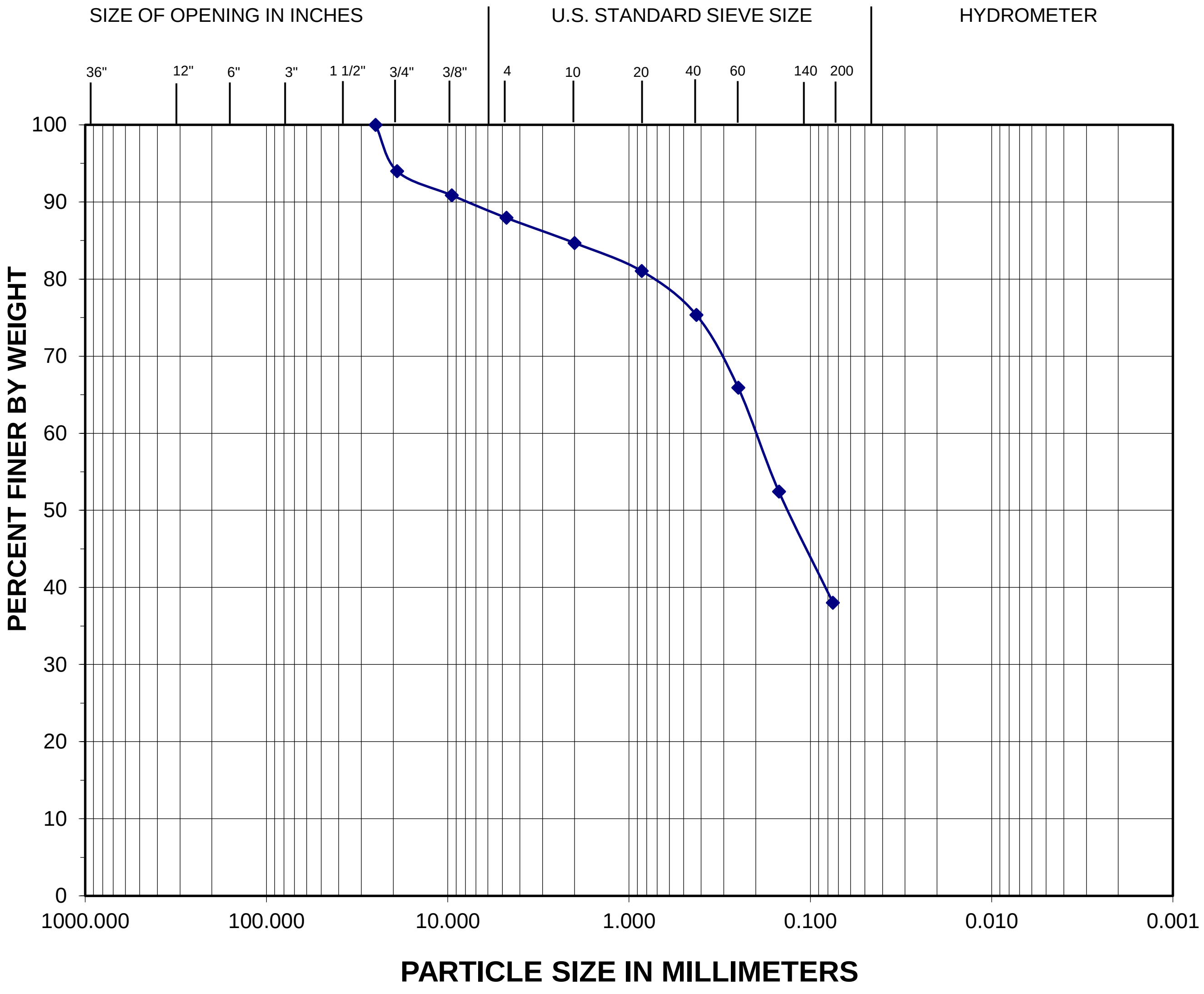
GRAIN SIZE ANALYSIS

Test Results Summary

ASTM D6913



		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	

Comments:

Exploration	Sample	Depth (feet)	Moisture (%)	Fines (%)	Description
B-3	S-2	2.5	17.4	38.0	Silty SAND, some gravel

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DATE OF TESTING: 2/8/2024

PROJECT NAME:

Petrovitsky Park Splash Pad