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Subsurface Exploration, Geologic Hazard, Preliminary Geotechnical Engineering, and Preliminary Infiltration Report

SAMMAMISH 18TH ASSEMBLAGE - Parcels 2625069-033, -048, and -090

King County, Washington
King County Application No. PLAT18-0009

Prepared For:

TOLL BROS., INC.

Project Nos. 180096E001 and 180351E001
September 24, 2018

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September 24, 2018
Project Nos. 180096E001 and 180351E001

Toll Bros., Inc.
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Attention: Mr. Jeff Peterson

Subject: Subsurface Exploration, Geologic Hazard, Preliminary Geotechnical Engineering, and Preliminary Infiltration Report
Sammamish 18th Assemblage - Parcels 2625069-033, -048, and -090
King County, Washington
King County Application No. PLAT18-0009

Dear Mr. Peterson:

We are pleased to present the enclosed copy of the subject report. This report summarizes the results of our subsurface exploration, geologic hazard, preliminary geotechnical engineering, and preliminary infiltration studies and offers recommendations for the design and development of the proposed project.

We have enjoyed working with you on this study and are confident that the recommendations presented in this report will aid in the successful completion of your project. If you should have any questions, or if we can be of additional help to you, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

Kurt D. Merriman, P.E.
Senior Principal Engineer

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SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, PRELIMINARY GEOTECHNICAL ENGINEERING, AND PRELIMINARY INFILTRATION REPORT

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I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of our subsurface exploration, geologic hazard, preliminary geotechnical engineering, and preliminary infiltration study for the proposed new residential subdivision known as the Sammamish 18th Assemblage. The site location is shown on the "Vicinity Map," Figure 1. The approximate locations of explorations completed for this study are shown on the "Site and Exploration Plan," Figure 2. This report is based on a site plan by D. R. Strong Consulting Engineers titled "Delappe Sheehan, Preliminary Subdivision Layout, Infiltration Vault Option," dated June 28, 2018.

Interpretive exploration logs are included in the Appendix. The conclusions and recommendations contained in this report should be reviewed and modified, or verified, if project plans change substantially.

1.1 Purpose and Scope

The purpose of this study was to provide subsurface data to be used in the design of the project. Our study included a review of selected geologic literature, observation of the excavation of exploration pits, and geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow groundwater. Geotechnical engineering studies were completed to formulate our recommendations for site preparation and grading, the type of suitable foundations and floors, allowable foundation soil bearing pressure, anticipated foundation and floor settlement, pavement recommendations, and drainage considerations. This report summarizes our current fieldwork and offers recommendations for development based on our present understanding of the project. We recommend that we be allowed to review any revisions to project plans and update the recommendations in this report as needed.

1.2 Authorization

This report has been prepared for the exclusive use of Toll Bros., Inc. and its agents for specific application to this project. Our work was performed in accordance with our scope of work and cost proposals, dated March 26, 2018 and July 2, 2018. We were authorized to proceed by means of consultant agreements.

Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering and engineering geology practices in effect in this area at the time our report was prepared. No other warranty, express or implied, is made.

2.0 PROJECT AND SITE DESCRIPTION

The subject site consists of three residential parcels located at 24403, 24407, and 24515 NE 18th Street in Sammamish, Washington (King County Parcels 2625069-033, -048, and -090). The roughly rectangular-shaped site is approximately 5.4 acres in area and bounded on the north by NE 18th Street, to the east and south by single-family residential development, and to the west by 244th Avenue NE. The properties are developed with three existing, single-family residences and various small, auxiliary buildings. The unbuilt areas are vegetated with grass lawns areas, landscaping shrubs and trees, and stands of mature conifer trees. Topography generally slopes downward to the southeast, south, and southwest from a high point near the property boundary between the middle and eastern parcel. Slopes generally range in inclination from gentle to moderate, with an isolated steeper slope on the western parcel. Overall vertical site relief is on the order of 36 feet.

We understand that the development will consist of constructing 32 new single-family residences, a stormwater facility, interior streets, sidewalks, and other miscellaneous site improvements.

3.0 SUBSURFACE EXPLORATION, TESTING, AND FIELD INVESTIGATION

Our field studies included excavating nine exploration pits, drilling two exploration borings, installing one well and one piezometer, performing one field infiltration test, several grain-size distribution tests, and water level monitoring. On the “Site and Exploration Plan” (Figure 2), the locations of the exploration pits EP-1 through EP-5, infiltration test pit IT-1, and exploration borings EB-1W and EB-2 were surveyed by Toll Bros., Inc., and the locations of exploration pits EP-6 through EP-9 were measured from existing site features. Interpretive exploration logs are presented in the Appendix.

The various types of sediments, as well as the depths where characteristics of the sediments changed, are indicated on the exploration logs presented in the Appendix. The depths indicated on the logs where conditions changed may represent gradational variations between sediment types in the field.

The conclusions and recommendations presented in this report are based on the explorations completed for this study. The number, locations, and depths of our explorations were completed within site and budgetary constraints. Because of the nature of exploratory work below ground, extrapolation of subsurface conditions between field explorations is necessary. It should be noted that differing subsurface conditions may sometimes be present due to the random nature of deposition and the alteration of topography by past grading and/or filling. The nature and extent of any variations between the field explorations may not become fully evident until construction. If variations are observed at that time, it may be necessary to re-evaluate specific recommendations in this report and make appropriate changes.

3.1 Exploration Pits

Associated Earth Sciences, Inc. (AESI) observed the excavation of nine exploration pits and one infiltration test pit; all explorations pits were excavated with track-mounted excavator. Exploration pits EP-1 through EP-4 were excavated on March 2nd by excavator provided by Toll Bros., Inc., exploration pit EP-5 was excavated on March 27th by excavator subcontracted to AESI, and exploration pits EP-6 through EP-9 were excavated on July 16th by excavator subcontracted to AESI. The pits permitted direct, visual observation of subsurface conditions. Materials encountered in the exploration pits were studied and classified in the field by an engineer from our firm. All exploration pits were backfilled after examination and logging. Selected samples were then transported to our laboratory for further visual classification and testing, as necessary.

3.2 Piezometer

AESI installed one groundwater piezometer in exploration pit EP-1 in an area proposed for stormwater infiltration; see Figure 2. The piezometer consisted of a 10-foot long, 1¼-inch polyvinyl chloride (PVC) pipe with hand-drilled perforations in the lower 2½ feet set at the bottom of the exploration pit before backfilling. The piezometer extended about 2½ inches above the backfilled ground surface, and the end was covered with a slip cap.

3.3 Infiltration Testing

In order to evaluate stormwater infiltration feasibility and to obtain a representative infiltration rate, one infiltration test, IT-1, was completed at the location shown on Figure 2. The test was conducted using the small-scale Pilot Infiltration Test (PIT) procedure outlined in the 2016 *King County Surface Water Design Manual* (KCSWDM). Exploration pit EP-5 and infiltration test pit IT-1 were excavated on March 27th by excavator subcontracted to AESI. A continuous water source was supplied to the test by a water truck subcontracted to AESI.

The infiltration test procedure consisted of excavating a test pit with a relatively flat bottom to the test depth of 5 feet. A staff gauge was installed in the test base to measure water level rise (head) to the nearest 0.01 feet. Water was introduced to the test area through a hose attached to a digital, propeller-type, flow meter assembly with instantaneous flow rate and total flow volume readouts. The discharge hose was equipped with a diffuser to minimize turbulence and scouring of the test bottom. Water was discharged into the test area for a "soaking period" of at least 6 hours to saturate the receptor soils in the immediate vicinity of the pit. After the soaking period, the "constant head" period of the test began and water was discharged into the test area at a constant rate for about 1 hour. After constant head period, the "falling head" period began and water flow into the test area was shut off.

Readings of the instantaneous flow rate, total flow volume, and water level were recorded at approximately 5- to 15-minute intervals throughout the soaking and constant head periods of the test. The water level and wetted area were recorded frequently as the water receded during the falling head period of the test. Infiltration test data were recorded by hand in the field and subsequently transferred to electronic spreadsheets. Infiltration test data sheets are included in the Appendix.

Upon completion of the infiltration test, the test pit was deepened to document the types of soils through which the water infiltrated and identify any soil layers that would restrict the downward flow of water. During excavation of the test pit after testing, AESI observed seepage at depths of about 6.5 feet and 9 feet below the ground surface. We interpret this seepage as return flow from infiltration testing due to perching on lower-permeability layers beneath the test depth. Pooling was observed at a depth of 11 feet below the ground surface and was interpreted as shallow groundwater not related to testing.

A summary of test results and stormwater infiltration recommendations are presented in the "Infiltration Feasibility" section of this report.

3.4 Exploration Borings

Exploration borings EB-1W and EB-2 were drilled on March 28 and March 29, 2018, by track-mounted, hollow-stem auger drill subcontracted to AESI. During the drilling process, samples were obtained at 2.5- and 5-foot intervals. The borings were continuously observed and logged by a field engineer from our firm. The interpretive exploration logs presented in the Appendix are based on the field logs, drilling action, and observation of the samples collected.

Disturbed, but representative samples were obtained by using the Standard Penetration Test (SPT) procedure in accordance with *American Society for Testing and Materials* (ASTM) D-1586. This test and sampling method consists of driving a standard, 2-inch outside-diameter, split-barrel sampler a distance of 18 inches into the soil with a 140-pound hammer free-falling a distance of 30 inches. The number of blows for each 6-inch interval is recorded, and the number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance ("N") or blow count. If a total of 50 blows are recorded at or before the end of one 6-inch interval, the blow count is recorded as the number of blows for the corresponding number of inches of penetration. The resistance, or N-value, provides a measure of the relative density of granular soils or the relative consistency of cohesive soils. These values are plotted on the attached boring logs.

The samples obtained from the split-barrel sampler were classified in the field and representative portions placed in water-tight containers. The samples were then transported to our laboratory for further visual classification and geotechnical laboratory testing, as necessary.

The various types of soil and groundwater elevations, as well as the depths where soil and groundwater characteristics changed, are indicated on the exploration boring logs presented in the Appendix of this report. The locations of our explorations were approximated by measuring from known site features.

3.5 Monitoring Well

Following drilling, one groundwater monitoring well was installed in exploration boring EB-1W to allow for longer-term monitoring of groundwater levels below the site. The well consisted of a 2-inch-diameter, PVC Schedule-40 well casing with threaded connections. The lower 10 feet of the well was a finely slotted (0.010-inch machine slot) well screen to permit water inflow. The annular space around the well screen was backfilled with silica sand, and the upper portion of annulus was sealed with bentonite grout and chips. A steel, flush-mount monument was set in concrete over the top of the wellhead for protection. The as-built configuration of the well is illustrated on the associated boring log included in the Appendix. The well was developed with a plastic Mini Typhoon pump and $^{3}/_{8}$-inch, inside-diameter poly tube assembly. The entire length of the well screen was surged incrementally from the top down at a rate of about 1 minute per foot of screen. Following surging, approximately 35 gallons of water were pumped from the well in EB-1W.

A water level data logger was installed in the well on March 30, 2018. We will collect continuous water level measurements for the next several months and up to 1 year, depending on the need for water level data during design development. Off-site groundwater depths and elevations were also obtained from an adjacent project. The data is considered provisional, but is included for reference. Off-site well locations are shown on Figure 1.

3.6 Water Level Monitoring

Several water level monitoring stations have been installed on or near the subject property. Off-site monitoring station locations are shown on Figure 1. Table 1 summarizes the monitoring stations and seasonal high water level data collected to date.

Table 1
Seasonal High Water Depth And Elevation at On-Site and Off-Site Monitoring Stations

Water Level Monitoring Station	Surface Water Elevation (ft)	Depth to Groundwater (ft)	Groundwater Elevation (ft)	Date
EB-1W, onsite	n/a	10.17	333.37	April 18, 2018
WP-1, onsite	n/a	8.87	332.33	April 25, 2018
MW-1, Mystic Lake	n/a	37.59	334.44	April 29, 2013
MW-1, 16th Street	n/a	2.46	336.73	April 29, 2013
Staff Gauge, 16th Street	337	n/a	n/a	May 22, 2017
Mystic Lake	354	n/a	n/a	May 22, 2017
Allen Lake	343	n/a	n/a	May 22, 2017
B-1, Terra	n/a	3.48	334.45	January 25, 2018
B-2, Terra	n/a	28.43	346.79	January 12, 2018
B-101, Terra	n/a	19.50	330.85	May 1, 2018

ft=feet

4.0 SUBSURFACE CONDITIONS

Subsurface conditions at the project site were inferred from the field explorations accomplished for this study, visual reconnaissance of the site, and review of selected geologic literature. The general distribution of geologic units is shown on the exploration logs. The explorations generally encountered native sediments, consisting of Vashon recessional outwash, ice-contact deposits, Vashon lodgement till, and Vashon advance outwash, overlain by thin deposits of topsoil and fill soils locally.

4.1 Stratigraphy

The following sections present more detailed subsurface information organized from the youngest (shallowest) to the oldest (deepest) sediment types.

Topsoil/ Forest Duff

A surficial soil layer consisting of soft, dark brown, sandy silt with some gravel and abundant organics was encountered in all explorations except EP-1, EP-2, and EP-7. This soil is interpreted as topsoil and was observed in thicknesses ranging from approximately 3 to 18 inches.

Due to their high organic content, topsoil materials are not considered suitable for foundation, roadway, or slab-on-grade floor support, or for use in a structural fill.

Existing Fill

Fill soils (those not naturally placed) were encountered in exploration pits EP-1, EP2, EP-6, and EP-7 to depths of 1 feet to 2 feet. Artificial fill should be expected in other developed areas of the site, such as around existing utilities and foundations. Observed fill was variable in nature, but generally consisted of gravel or silt with some sand and organic material such as roots and decaying particles. Waste materials, including wires and plastic, were observed in EP-6.

Due to the variable organic and waste content, existing fill is not considered suitable for foundation support or reuse in structural fills.

Vashon Recessional Outwash

Underlying the topsoil or fill in explorations EB-1, EB-2, EB-4, EB-5, IT-1, and EB-1W, sediments encountered typically consisted of medium dense, gravelly, medium to coarse sand with silt interbeds interpreted as Vashon recessional outwash. Recessional outwash was observed to depths of approximately 2.5 to 7 feet below the ground surface. In all explorations, the upper 1.5 to 2 feet of recessional outwash was weathered, characterized by an orange or tan color, a loose to medium density, higher silt content, and the presence of fine roots. Vashon recessional outwash sediments were deposited by meltwater streams flowing from the receding Vashon glacier approximately 10,000 years ago. The weathered condition was created by natural processes of freeze-thaw and bioturbation by roots and animals.

Recessional outwash is typically suitable for support of light to moderately loaded foundations and, where it occurs in sufficient, unsaturated thickness, may serve as a receptor for surface water infiltration.

Vashon Ice-Contact Deposit

Sediments encountered below the recessional outwash in explorations EP-1, EP-2, EP-5, IT-1, and EB-1W, and below the topsoil in EB-2, generally consisted of medium dense to very dense, silty, gravelly, medium to coarse sand with occasional interbeds of cleaner and siltier sands. We interpret these sediments to have been deposited in close contact with debris-laden glacial ice during the Vashon Stade of the Fraser Glaciation, approximately 12,500 to 17,500 years ago. Ice-contact deposits were encountered to depths of 15 to 22 feet below the ground surface in EB-1W and EB-2, respectively, and extended beyond the maximum depths explored of 10 to 12 feet in EP-1, EP-2, EP-5, and IT-1.

With proper preparation, the ice-contact deposits are suitable for foundation support due to their dense configuration. The high fines content and low permeability of these sediments are not favorable for stormwater infiltration.

Vashon Lodgement Till

Underlying the topsoil and fill in explorations EP-3 and EP-6 through EP-9, and underlying the recessional outwash in EP-4, we encountered medium dense to very dense, very silty, gravelly fine sand with occasional cobbles and boulders interpreted as Vashon-age lodgement till. In these explorations, lodgement till was encountered to the full depths explored of 6 to 8.5 feet below the ground. In all explorations except EP-6, the upper 1.5 to 2 feet of lodgement till was weathered, characterized by an orange or tan color, a loose to medium dense consistency, and the presence of fine roots. The lodgement till was deposited at the base of an active ice sheet and was subsequently compacted by the weight of the overlying glacial ice.

Excavations into the lodgement till should be prepared to encounter large cobbles and boulders at random. Lodgement till typically possesses high-strength and low-compressibility attributes that are favorable for support of foundations, floor slabs, and paving, with proper preparation. Lodgement till soils are typically low permeability due to their high silt content and consolidated consistency and are therefore not considered suitable for infiltration.

Vashon Advance Outwash

Underlying the ice-contact deposits in EB-1W and EB-2, very dense, coarse sand with silty and gravelly interbeds extended to full depths explored of 31.5 and 34 feet, respectively. We interpret these sediments as Vashon advance outwash. Vashon advance outwash sediments were deposited by flowing water from the base of the southward advancing Vashon glacial front. Subsequent to deposition, the advance outwash was overridden by approximately 3,000 feet of glacial ice, resulting in the very dense condition of this unit. In both explorations, this unit was saturated.

In an undisturbed condition, the advance outwash is suitable for support of light to heavy foundation loads and for roadway subbase material. Due to the presence of groundwater in this unit, the advance outwash is not considered suitable for stormwater infiltration at the project site.

4.2 Published Geologic Literature

We reviewed the published *Geologic Map of the Issaquah 7.5' Quadrangle*, King County, Washington, by D. B. Booth and J.P. Minard, 1992. This map indicates that the site is underlain by recessional outwash and lodgement till sediments and our interpretations of subsurface conditions onsite generally agree.

4.3 Laboratory Testing

As a part of our study of the infiltration potential of the site soils, we completed two laboratory grain-size analyses. We performed grain-size analyses on representative samples of the recessional outwash collected at a depth of about 5 feet in IT-1, and the Vashon ice-contact deposits collected from depths of about 7.5 to 8 feet in EP-1 and 9 to 10 feet in EP-2. Copies of the grain-size analyses reports are included in the Appendix.

4.4 Hydrology

Our understanding of the hydrologic conditions at the site is based on our explorations and water level monitoring data, including:

- Groundwater encountered in our borings at the time of our explorations;
- Water level data from the data logger in our on-site well EB-1W, last download July 19, 2018;
- Manual water level measurements in our piezometer WP-1;
- Groundwater monitoring records from the well MW-1 and surface water monitoring records from the staff gauge in the Northeast 16th Street Assemblage about ¼ mile to the south of the site;
- Groundwater monitoring records from the well MW-1 and surface water records in the Mystic Lake development about $^{1}/_{10}$ mile to the southwest of the site;
- Surface water elevations of Allen Lake about $^{3}/_{4}$ mile to the south of the site; and
- Off-site water level data obtained for other projects.

Intermittent perched groundwater or “interflow” is expected to accumulate at the upper surface of fine-grained native sediments, such as glacial till, and minor amounts of seepage may be expected seasonally. Interflow would tend to flow downslope closely following the existing topography and recharge underlying aquifers. It should be noted that the depth and occurrence of groundwater seepage may vary in response to changes in season, amount of precipitation, on- and off-site land use, and other factors. Explorations for this study were conducted on March 2nd, 27th, and 28th, and July 16th of 2018 and did not encounter perched groundwater.

Groundwater seepage was encountered within the advance outwash deposits at about 15.5 feet in EB-1W and at about 21 feet in EB-2. This groundwater is interpreted to represent the shallow aquifer underlying the site and vicinity. Groundwater within the advance outwash may interact with surface water features near the site, including the wetland within the 16th Street Assemblage to the south. Recharge to the unconfined aquifer is primarily through rainfall and from surface water features. Recharge also occurs due to infiltration of interflow and surface runoff from surrounding till uplands, which infiltrates along the upland margins into the surrounding outwash deposits.

Table 1 presents seasonal high groundwater data of the shallow aquifer from the site and nearby projects. Based on this data, we interpret the regional groundwater flow direction in the project area is to the north. Continuous groundwater level data collected from March 30 to July 19, 2018, indicates seasonal high groundwater for the 2017–2018 winter season near the proposed infiltration facility was 334.5 feet elevation.

II. GEOLOGIC HAZARDS AND MITIGATIONS

The following discussion of potential geologic hazards is based on the geologic, slope, and shallow groundwater conditions as observed and discussed herein.

5.0 LANDSLIDE AND STEEP SLOPE HAZARDS AND MITIGATIONS

King County Municipal Code (KCMC) 21A.06.680 defines landslide hazard areas as areas "subject to severe risk of landslide, such as:

A. An area with a combination of:

1. Slopes steeper than fifteen percent of inclination;
2. Impermeable soils, such as silt and clay, frequently interbedded with granular soils, such as sand and gravel; and
3. Springs or groundwater seepage;

B. An area that has shown movement during the Holocene epoch, which is from 10,000 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. (Ord. 15051 § 70, 2004: Ord. 10870 § 176, 1993)"

Based on our observations of the surface and subsurface conditions encountered in our explorations, it is our opinion that the site is not considered a landslide hazard area.

KCMC 21A.06.1230 defines a steep slope hazard area as "an area on a slope of forty percent inclination or more within a vertical elevation change of at least ten feet." Based on review of the King County iMap web application, a slope in the northeast quadrant of the west lot (Parcel No. 2625069033) is mapped as a potential steep slope hazard area. The previously referenced survey shows the slope is inclined up to about 45 percent, with vertical elevation change ranging from about 10 to 16 feet. Based on these conditions, this slope is considered a steep slope hazard area.

The steep slope appears to have been made by grading associated with the development of the existing driveway. Evidence of slope movement, such as colluvium, fans, or pistol-butted trees, was not observed. Our explorations at similar elevations as the steep slope area (EP-2 and EP-3) and above the steep slope encountered glacially consolidated sediments at relatively shallow depth. Based on these conditions, it is our opinion that the existing steep slope hazard is low.

We understand project plans include regrading of this steep slope to a level or gently inclined configuration. In order to mitigate steep slope hazards if steep slopes are not flattened by grading, we recommend the following for any remaining steep slopes:

- Establishment of a buffer from the top and toe of slope extending a distance equal to at least the height of the slope in which no development is permitted.
- Collection and direction of surface water away from the top of the steep slope.
- Maintenance of slope-stabilizing vegetation on the steep slope and within the steep slope buffers.

It is our opinion that, with implementation of these mitigations and the recommendations contained in this report, the risk posed to the project by the steep slope hazards is very low. However, it must be understood that no recommendations or engineering design can yield a guarantee of stable slopes. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner. No detailed slope stability analysis was completed for this project and none is warranted, in our opinion.

6.0 SEISMIC HAZARDS AND MITIGATIONS

Earthquakes occur regularly in the Puget Lowland. Most of these events are small and are not felt by people. However, large earthquakes do occur, as evidenced by the 2001, 6.8-magnitude event; the 1965, 6.5-magnitude event; and the 1949, 7.2-magnitude event. The 1949 earthquake appears to have been the largest in this region during recorded history and was centered in the Olympia area. Evaluation of earthquake return rates indicates that an earthquake of the magnitude between 5.5 and 6.0 is likely within a given 20-year period.

Generally, there are four types of potential geologic hazards associated with large seismic events: 1) surficial ground rupture, 2) seismically induced landslides, 3) liquefaction, and 4) ground motion. The potential for each of these hazards to adversely impact the proposed project is discussed below.

6.1 Surficial Ground Rupture

The nearest known major faults to the project site are the Seattle Fault Zone located approximately 2 miles to the south of the project and the South Whidbey Fault Zone located approximately 4 miles to the north. Recent studies of the Seattle Fault Zone indicate that it is an active fault zone capable of generating surface ruptures. The Seattle Fault Zone is not well understood and is an area of active research. According to the U.S. Geological Survey (USGS) studies, the recurrence interval of movements along the Seattle Fault Zone is unknown but is speculated to be on the order of 1,100 years. The South Whidbey Island Fault Zone is also thought to be an active fault zone capable of producing surface ruptures, though research of this fault zone near the project is preliminary and limited. Due to the distance from the site to the known fault zones and the suspected long recurrence interval for the Seattle Fault Zone, it is our opinion that the risk posed to the project during the expected life of the structures by surface faulting is low.

6.2 Seismically Induced Landslides

The medium dense to very dense glacially consolidated sediments underlying the site typically present a low risk of landsliding under the topographic conditions at the site. Provided that the recommendations presented in this report are properly followed, it is our opinion that the risk of damage to the proposed structures by landsliding under both static and seismic conditions is low. Landslide hazards are discussed further in the “Landslide and Steep Slope Hazards and Mitigations” section of this report.

6.3 Liquefaction

Liquefaction is a process through which unconsolidated soil loses strength as a result of vibrations, such as those which occur during a seismic event. During normal conditions, the weight of the soil is supported by both grain-to-grain contacts and by the fluid pressure within the pore spaces of the soil below the water table. Extreme vibratory shaking can disrupt the grain-to-grain contact, increase the pore pressure, and result in a temporary decrease in soil shear strength. The soil is said to be liquefied when nearly all of the weight of the soil is supported by pore pressure alone. Liquefaction can result in deformation of the sediment and settlement of overlying structures. Areas most susceptible to liquefaction include those areas underlain by non-cohesive silt and sand with low relative densities, accompanied by a shallow water table.

Our explorations generally encountered medium dense or denser sediments at relatively shallow depths. Where looser deposits were encountered, these occurred in relatively thin layers and significant groundwater was not observed within them. Based on these conditions, it is our opinion that the potential risk of damage to the proposed development by liquefaction is low.

6.4 Ground Motion/Seismic Site Class (2015 *International Building Code*)

Structural design of the buildings should follow 2015 *International Building Code* (IBC) standards. We recommend that the project be designed in accordance with Site Class "D," as defined in IBC Table 20.3-1 of *American Society of Civil Engineers* (ASCE) *7 - Minimum Design Loads for Buildings and Other Structures*.

7.0 EROSION HAZARD AND MITIGATION

Erosion hazard areas are defined, in part, by KCMC 21A.06.415 as "an area underlain by soils that is subject to severe erosion when disturbed." The western portion of the site is mapped on the USDA Web Soil Survey as EvC, Everett very gravelly sandy loam, and the eastern portion of the site is mapped as Agc, Alderwood gravelly sandy loam; both are rated "moderate" erosion hazard. Our exploration encountered soils that contained high percentages of silt, such as the lodgement till, near the surface or at shallow depth. These soils are considered susceptible to erosion when disturbed or exposed to surface water. Based on these conditions, it is our opinion that during construction the risk posed to the project by erosion is moderate.

In order to mitigate erosion hazard, project plans should include implementation of temporary erosion controls in accordance with local standards of practice. Control methods should include limiting earthwork to seasonally drier periods, typically April 30 to October 31, use of perimeter silt fences, and straw mulch in exposed areas. Removal of existing vegetation should be limited to those areas that are required to construct the project, and new landscaping and vegetation with equivalent erosion mitigation potential should be established as soon as possible after grading is complete. During construction, surface water should be collected as close as possible to the source to minimize silt entrainment that could require treatment or detention prior to discharge. Timely implementation of permanent drainage control measures should also be a part of the project plans and will help reduce erosion and generation of silty surface water onsite.

III. PRELIMINARY DESIGN RECOMMENDATIONS

8.0 INTRODUCTION

Our exploration indicates that, from a geotechnical engineering standpoint, the proposed project is feasible provided the recommendations contained herein are properly followed. Bearing consisting of medium dense to dense native sediments was encountered in all explorations at relatively shallow depth. Infiltration of stormwater appears feasible in the northwest area of the site. The following report sections provide recommendations regarding site preparation, grading, foundations, floor support, drainage, paving, and stormwater vault construction.

9.0 SITE PREPARATION

Site preparation of building and paving areas should include removal of all existing buildings, foundations, buried utilities, grass, trees, brush, debris, and any other deleterious materials. Existing fill should be removed. We installed one groundwater observation well for this study. If the existing observation well is not compatible with future site development plans, it should be decommissioned in accordance with *Washington Administrative Code* (WAC) Section 173-160 by a Washington State licensed well driller. Buried utilities should be removed from foundation areas, and should be abandoned in place or removed from below planned new paving. Any depressions below planned final grades should be backfilled with structural fill, as discussed under the "Structural Fill" section of this report.

Existing topsoil should be stripped from structural areas. The observed in-place depth of topsoil and grass at the exploration locations is presented on the exploration logs in the Appendix. After stripping, remaining roots and stumps should be removed from structural areas. All native soils disturbed by stripping and grubbing operations should be recompacted as described below for structural fill.

Erosion and surface water control should be established around the clearing limits to satisfy local requirements.

9.1 Temporary Cut Slopes

In our opinion, stable construction slopes should be the responsibility of the contractor and should be determined during construction. For estimating purposes, however, temporary, unsupported cut slopes can be planned at 1.5H:1V (Horizontal:Vertical) in unsaturated topsoil, weathered native soils, recessional outwash, and existing fill. Temporary slopes of 1H:1V can be planned in unsaturated, unweathered, lodgement till, ice-contact, and advance outwash sediments. Permanent cut slopes in medium dense, native sediments or structural fill must not exceed a 2H:1V inclination.

These slope angles are for areas where groundwater seepage is not present at the faces of the slopes, which may require temporary dewatering in the form of pumped sumps or other measures. If ground or surface water is present when the temporary excavation slopes are exposed, flatter slope angles may be required. As is typical with earthwork operations, some sloughing and raveling may occur, and cut slopes may have to be adjusted in the field. In addition, WISHA/OSHA regulations should be followed at all times.

9.2 Site Disturbance

The topsoil, existing fill, ice-contact, and lodgement till soils contain fine-grained material, which makes them moisture-sensitive and subject to disturbance when wet. The contractor must use care during site preparation and excavation operations so that the underlying soils are not softened. If disturbance occurs, the softened soils should be removed and the area brought to grade with structural fill.

9.3 Winter Construction

The topsoil, existing fill, ice-contact, and lodgement till soils contain substantial proportions of silt and are considered highly moisture-sensitive. Soils excavated onsite suitable for reuse will likely require drying during favorable dry weather conditions to allow such reuse in structural fill applications. Care should be taken to seal all earthwork areas during mass grading at the end of each workday by grading all surfaces to drain and sealing them with a smooth-drum roller. Stockpiled soils that will be reused in structural fill applications should be covered whenever rain is possible.

If winter construction is expected, crushed rock fill could be used to provide construction staging areas. The stripped subgrade should be observed by the geotechnical engineer, and should then be covered with a geotextile fabric, such as Mirafi 500X or equivalent. Once the fabric is placed, we recommend using a crushed rock fill layer at least 10 inches thick in areas where construction equipment will be used.

10.0 STRUCTURAL FILL

All references to structural fill in this report refer to subgrade preparation, fill type, placement, and compaction of materials, as discussed in this section. If a percentage of compaction is specified under another section of this report, the value given in that section should be used.

For backfill of buried utilities in the right-of-way, the backfill should be placed and compacted in accordance with King County codes and standards.

After stripping, planned excavation, and any required overexcavation have been performed to the satisfaction of the geotechnical engineer/engineering geologist, the surface of the exposed ground should be recompacted to a firm and unyielding condition. If the subgrade contains too much moisture, adequate recompaction may be difficult or impossible to obtain, and should probably not be attempted. In lieu of recompaction, the area to receive fill should be blanketed with washed rock or quarry spalls to act as a capillary break between the new fill and the wet subgrade. Where the exposed ground remains soft and further overexcavation is impractical, placement of an engineering stabilization fabric may be necessary to prevent contamination of the free-draining layer by silt migration from below.

After recompaction of the exposed ground is approved, or a free-draining rock course is laid, structural fill may be placed to attain desired grades. Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer/engineering geologist, placed in maximum 8-inch loose lifts, with each lift being compacted to 95 percent of ASTM D-1557. The top of the compacted fill should extend horizontally outward a minimum distance of 3 feet beyond the locations of the perimeter footings or roadway edges before sloping down at a maximum angle of 2H:1V.

The contractor should note that any proposed fill soils should be evaluated by AESI prior to their use in fills. This would require that we have a sample of the material at least 72 hours in advance to perform a Proctor test and determine its field compaction standard.

Soils in which the amount of fine-grained material (smaller than the No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. The underlying lodgement till soils are estimated to contain substantially more than 5 percent fine-grained material. Use of moisture-sensitive soil in structural fills should be limited to favorable dry weather and dry subgrade conditions. Construction equipment traversing the site when the soils are wet can cause considerable disturbance.

If fill is placed during wet weather or if proper compaction cannot be obtained, a select, import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil, with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction, and at least 25 percent retained on the No. 4 sieve.

In order to reuse excavated on-site soils in structural fill applications, it will be necessary to moisture-condition wet site soils by aeration and drying during favorable dry weather conditions. Alternatives to drying site soils include using imported granular soils suitable for use in structural fill, or possibly treating wet soils with Portland cement.

11.0 INFILTRATION FEASIBILITY

The feasibility of stormwater infiltration depends upon the presence of a suitable receptor soil of sufficient thickness, extent, permeability, and vertical separation from groundwater. Of the soils encountered onsite, only the outwash deposits are typically permeable enough for stormwater infiltration. Due to the presence of groundwater within the advance outwash encountered in EB-1W and EB-2, this deposit is not considered a suitable infiltration receptor. The recessional outwash observed in EP-2 and EP-4 was present in relatively thin deposits of about 1.5 feet thick each and is not considered a suitable infiltration receptor. Recessional outwash encountered in EP-1 in the northwest area of the site and in EP-5, IT-1, and EB-1W in the southwest area of the site was observed in unsaturated thicknesses of about 3 to 5 feet, and therefore may be suitable for stormwater infiltration.

11.1 Laboratory Test Results

Laboratory grain size analyses were performed on selected soil samples; see Table 2 for details. Complete test results are presented in the Appendix.

Table 2
Sample Source, Depth Below Ground Surface (bgs), and Grain-Size Description

Sample Source	Sample Depth (feet bgs)	Description
EP-1	7.5 to 8	Very sandy GRAVEL, trace silt
EP-2	9 to 10	Very gravelly, silty, SAND
IT-1	5	Very sandy GRAVEL, some silt

11.2 Infiltration Test Results

Infiltration testing was performed in the southwest area of the site as described in the "Subsurface Exploration" section of this report. A summary of test results is presented in Table 3.

Table 3
Summary of Field Infiltration Test Data

Test No.	Test Depth (feet)	Total Water Used (Gallons)	Wetted Area (square feet)	Constant Head Flow Rate (gallons/ minute)	Field-based Constant Head Infiltration Rate (inches/ hour)	Field-based Falling Head Infiltration Rate (inches/ hour)
IT-1	5	662	13.5	2.1	14.9	15.1

11.3 Preliminary Design Infiltration Rate

Based on the subsurface conditions at the site and the results of our field and laboratory testing, we recommend a shallow infiltration facility, such as the proposed bottomless vault, that uses the recessional outwash deposit in the northwest or southwest areas of the site as a stormwater receptor. Sufficient vertical separation must be maintained between the bottom of the facility and the seasonal high groundwater. Because the infiltration receptor sediments are thin and the anticipated seasonal high groundwater is shallow, the effect of groundwater mounding on stormwater facility sizing is currently unknown. Subsurface conditions are expected to vary within the footprint of a future infiltration facility. Actual subsurface conditions are expected to vary from those encountered at IT-1 and may result in lower design infiltration rates and a larger facility footprint than currently envisioned. Additional site specific infiltration testing and a groundwater mounding analysis will be needed to determine final facility sizing as described in our May 30, 2018 technical memorandum.

As described in the 2016 (KCSWDM), the design infiltration rate for infiltration facilities is derived using the correction factors for testing, facility geometry, and plugging, per the following formula:

$I_{design} = I_{measured} \times F_{testing} \times F_{geometry} \times F_{plugging}$

where I_{design} and $I_{measured}$ are the design and measured infiltration rates, respectively. As stated in the "Infiltration Test Results" section, the field infiltration rate was approximately 15 inches per hour (in/hr).

The correction factor $F_{testing}$ accounts for uncertainties in the testing methods. This factor is defined as 0.50 for small-scale PITs.

The $F_{geometry}$ correction factor accounts for the influence of facility geometry and depth to the water table or impervious strata on the field-based infiltration rate. The KCSWDM states that this factor must be between 0.25 and 1.0, as determined by the following equation:

$F_{geometry}$ = 4 D/W + 0.05

Where D = Depth from the bottom of the proposed facility to the maximum wet season water table or nearest impervious layer, whichever is less; and

W = Width of the facility.

Based on the preliminary site plan identified earlier, we understand D will be equal to 3 feet and W will be equal to approximately 80 feet. Therefore, $F_{geometry}$ can be taken as the minimum required 0.25.
The plugging factor ($F_{plugging}$) is based on the grain size of the materials tested. For coarse sands to cobbles, such as those encountered in EP-1 at 7.5 feet and IT-1 at 5 feet, $F_{plugging}$ can be taken as 1.0. For medium sands, $F_{plugging}$ can be taken as 9.0. If the facility is preceded by a water quality treatment best management practice (BMP), the plugging factor is 1.0.

The resulting preliminary design infiltration rate is therefore 1.9 inches per hour. This rate is for planning purposes only. Site-specific infiltration testing is recommended within the footprint of the proposed infiltration facility.

12.0 FOUNDATIONS

It is not recommended that foundations be placed directly on the topsoil or existing fill. Therefore, overexcavation and replacement of these deposits or extension of the foundation to native soil is recommended. Spread footings may be used for building support when they are founded on approved structural fill placed as described above, or on undisturbed, medium dense or denser natural soils that are prepared as recommended in this report. Suitable foundation bearing soils were generally observed at 2 to 5 feet below existing grade, but may be deeper in some areas near existing foundations and utilities. One should refer to exploration logs in the Appendix for the observed depth to bearing soils at the exploration pit locations.

For residential structures, footings may be designed for an allowable foundation soil bearing pressure of 2,500 pounds per square foot (psf), including both dead and live loads. An increase of one-third may be used for short-term wind or seismic loading. Perimeter footings should be buried at least 18 inches into the surrounding soil for frost protection. However, all foundations must penetrate to the prescribed bearing strata, and no foundations should be constructed in or above loose, organic, or existing fill soils.

Anticipated settlement of footings founded as recommended should be on the order of ¾ inch or less, with differential settlement of ½ inch or less. However, disturbed material not removed from footing trenches prior to footing placement could result in increased settlements. All footing areas should be inspected by AESI prior to placing concrete to verify that the foundation subgrades are undisturbed and construction conforms to the recommendations contained in this report. Such inspections may be required by King County. Perimeter footing drains should be provided as discussed under the "Drainage Considerations" section of this report.

It should be noted that the area bounded by lines extending downward at 1H:1V from any footing must not intersect another footing or intersect a filled area that has not been compacted to at least 95 percent of ASTM D-1557. In addition, a 1.5H:1V line extending down and away from any footing must not daylight because sloughing or raveling may eventually undermine the footing. Thus, footings should not be placed near the edges of steps or cuts in the bearing soils.

13.0 FLOOR SUPPORT

If crawl-space floors are used, an impervious moisture barrier should be provided above the soil surface within the crawl space. Slab-on-grade floors may be used over medium dense or denser native soils, or over structural fill placed as recommended in the "Site Preparation" and "Structural Fill" sections of this report. Slab-on-grade floors should be cast atop a minimum of 4 inches of washed pea gravel or washed crushed "chip" rock with less than 3 percent passing the U.S. No. 200 sieve to act as a capillary break. The floors should also be protected from dampness by covering the capillary break layer with an impervious moisture barrier at least 10 mils in thickness.

We recommend that an AESI representative be allowed to monitor all floor slab construction to verify suitable conditions. Our monitoring services would include probing of subgrade soils, observation and testing of underslab fill layers, and a check of layer thicknesses. Drainage should be provided for all slabs as discussed under the "Drainage Considerations" section of this report.

14.0 DRAINAGE CONSIDERATIONS

Groundwater was encountered in our explorations and depending upon the time of year that construction is performed, may be encountered in proposed excavations. Therefore, prior to site work and construction, the contractor should be prepared to provide temporary drainage and subgrade protection, including during utility installation, as necessary.

All perimeter footings, slabs, basement walls, and retaining walls should be provided with a drain at the footing or slab subgrade elevation. Drains should consist of rigid, perforated, PVC pipe surrounded by washed pea gravel. The level of the perforations in the pipe should be set downward and at the bottom of the footing at all locations, and the drain collectors should be constructed with sufficient gradient to allow gravity discharge away from the buildings. In addition, all foundation walls taller than 3 feet should be lined with a minimum, 12-inch-thick, washed gravel blanket drain provided to within 1 foot of finish grade that ties into the footing drain. A prefabricated drainage mat is not an acceptable alternative to the gravel blanket drain unless the entire excavation backfill consists of free-draining structural fill. Roof and surface runoff should not discharge into the footing drain system, but should be handled by a separate, rigid, tightline drain.

In planning, exterior grades adjacent to foundations should be sloped downward away from the structures to achieve surface drainage. These recommendations apply to conventional shallow foundation walls and landscape walls less than about 4 feet tall. One should refer to the following section for walls up to 10 feet tall.

15.0 CAST-IN-PLACE RETAINING WALLS AND BASEMENT WALLS

All backfill behind foundation walls or around foundation units should be placed as per our recommendations for structural fill and as described in this section of the report. Horizontally backfilled walls that are free to yield laterally at least 0.1 percent of their height may be designed using an equivalent fluid pressure equal to 35 pounds per cubic foot (pcf). Fully restrained, horizontally backfilled, rigid walls that cannot yield should be designed for an equivalent fluid pressure of 50 pcf. Walls with sloping backfill up to a maximum gradient of 2H:1V should be designed using an equivalent fluid pressure of 55 pcf for yielding conditions or 75 pcf for fully restrained conditions. If parking areas are adjacent to walls, a surcharge equivalent to 2 feet of soil should be added to the wall height in determining lateral design forces.

As required by the 2015 IBC, retaining wall design should include a seismic surcharge pressure in addition to the equivalent fluid pressures presented above. Considering the site soils and the recommended wall backfill materials, we recommend a seismic surcharge pressure of 5H and 10H psf, where H is the wall height in feet for the “active” and “at-rest” loading conditions, respectively. The seismic surcharge should be modeled as a rectangular distribution with the resultant applied at the midpoint of the walls.

The lateral pressures presented above are based on the conditions of a uniform backfill consisting of excavated on-site soils, or imported structural fill compacted to 90 percent of ASTM D-1557. A higher degree of compaction is not recommended, as this will increase the pressure acting on the walls. A lower compaction may result in settlement of the slab-on-grade or other structures supported above the walls. Thus, the compaction level is critical and must be tested by our firm during placement. Surcharges from adjacent footings or heavy construction equipment must be added to the above values. Perimeter footing drains should be provided for all retaining walls, as discussed under the "Drainage Considerations" section of this report.

It is imperative that proper drainage be provided so that hydrostatic pressures do not develop against the walls. This would involve installation of a minimum 1-foot-wide blanket drain to within 1 foot of finish grade for the full wall height using imported, washed gravel against the walls. If situations exist where a footing drain is not feasible for a foundation wall or retaining wall, the wall should be designed for saturated lateral earth pressures and a hydrostatic surcharge. We should be allowed to offer situation-specific recommendations if this situation arises. The use of drainage improvements as recommended herein does not alleviate the need for waterproofing where finished spaces are planned on the interior side of basement walls. Backfilled walls with finished interior space should be waterproofed in accordance with recommendations of the building designer.

15.1 Passive Resistance and Friction Factors

Lateral loads can be resisted by friction between the foundation and the native soils or supporting structural fill soils, and by passive earth pressure acting on the buried portions of the foundations. The foundations must be backfilled with structural fill and compacted to at least 95 percent of the maximum dry density to achieve the passive resistance provided below. We recommend the following allowable design parameters:

- Passive equivalent fluid = 250 pcf
- Coefficient of friction = 0.35

16.0 PAVEMENT RECOMMENDATIONS

We understand that the proposed project will include construction of a paved internal road and driveways. Pavement areas should be prepared in accordance with the "Site Preparation" section of this report. If the stripped native soil or existing fill pavement subgrade can be compacted to a firm and unyielding condition as determined by the geotechnical engineer/engineering geologist, no additional overexcavation is required. Soft or yielding areas should be overexcavated to provide a suitable subgrade and backfilled with structural fill.

The pavement sections included in this report section are for streets and parking areas onsite and are not applicable to right-of-way improvements. If any new paving of public streets is required, we should be allowed to offer situation-specific recommendations.

The exposed ground should be recompacted to 95 percent of ASTM D-1557. If required, structural fill may then be placed to achieve desired subbase grades. Upon completion of the recompaction and structural fill, a pavement section consisting of 2½ inches of asphaltic concrete pavement (ACP) underlain by 4 inches of 1¼-inch crushed surfacing base course is the recommended minimum in areas of planned passenger car driving and parking. In heavy traffic areas, a minimum pavement section consisting of 3 inches of ACP underlain by 2 inches of $^{5}/_{8}$-inch crushed surfacing top course and 4 inches of 1¼-inch crushed surfacing base course is recommended. The crushed rock courses must be compacted to 95 percent of the maximum density, as determined by ASTM D-1557. All paving materials should meet gradation criteria contained in the current Washington State Department of Transportation (WSDOT) Standard Specifications.

Depending on construction staging and desired performance, the crushed base course material may be substituted with asphalt treated base (ATB) beneath the final asphalt surfacing. The substitution of ATB should be as follows: 4 inches of crushed rock can be substituted with 3 inches of ATB, and 6 inches of crushed rock may be substituted with 4 inches of ATB. ATB should be placed over a native or structural fill subgrade compacted to a minimum of 95 percent relative density, and a 1½- to 2-inch thickness of crushed rock to act as a working surface. If ATB is used for construction access and staging areas, some rutting and disturbance of the ATB surface should be expected. The general contractor should remove affected areas and replace them with properly compacted ATB prior to final surfacing.

17.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

This report is based on a site plan that was current at the time this report was written. We are available to provide additional geotechnical consultation as the project design develops and possibly changes from that upon which this report is based. We recommend that AESI perform a geotechnical review of the plans prior to construction. In this way, our earthwork and foundation recommendations may be properly interpreted and implemented in the design.

We are also available to provide geotechnical engineering and monitoring services during construction. The integrity of the foundations for residences and of retaining walls depends on proper site preparation and construction procedures. In addition, engineering decisions may have to be made in the field in the event that variations in subsurface conditions become apparent. Construction monitoring services are not part of the current scope of work. If these services are desired, please let us know, and we will prepare a cost proposal.

We have enjoyed working with you on this study and are confident these recommendations will aid in the successful completion of your project. If you should have any questions or require further assistance, please do not hesitate to call.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

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Senior Staff Engineer

Curtis J. Koger, L.G., L.E.G., L.Hg.
Senior Principal Geologist/Hydrogeologist

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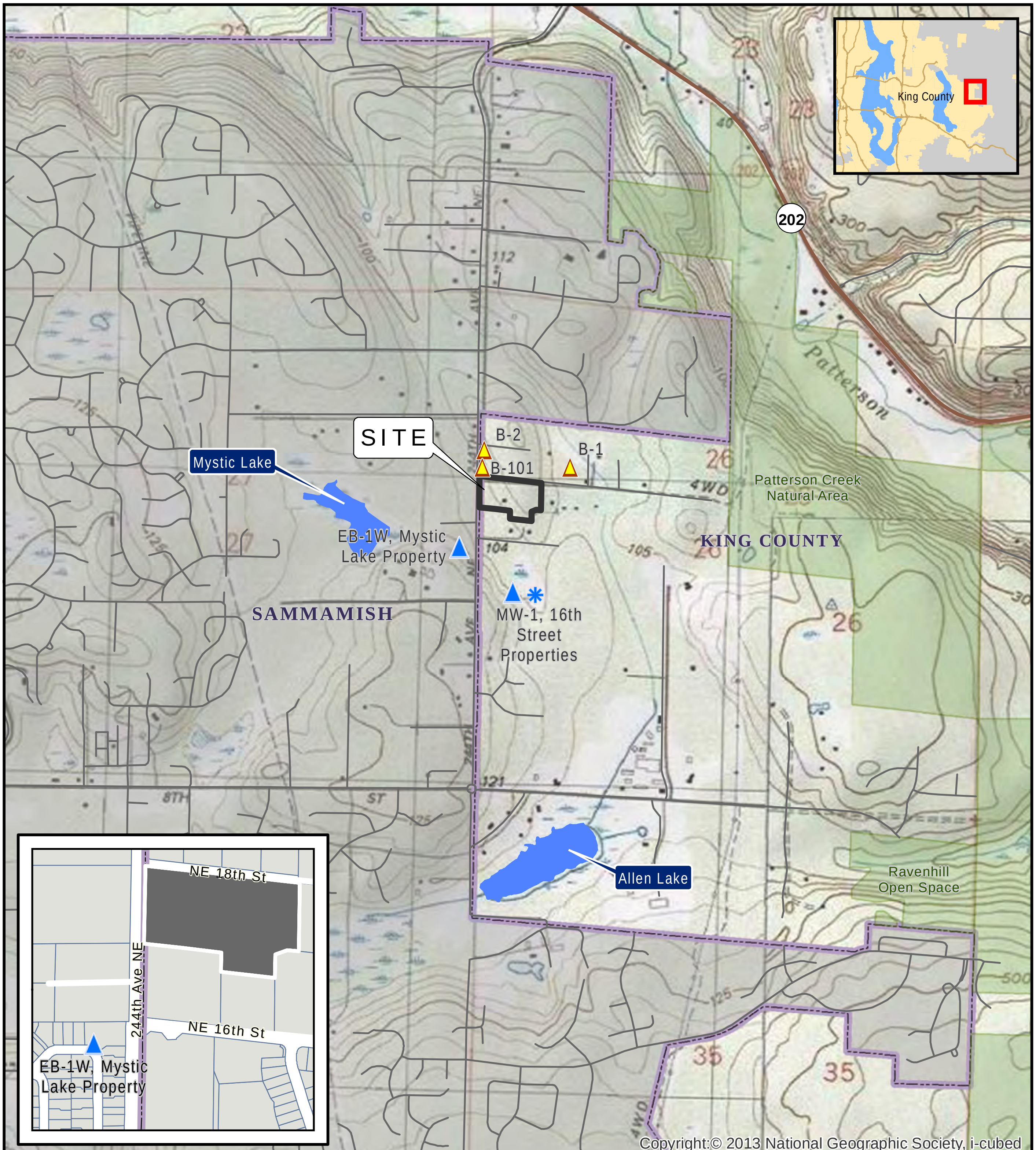
REGISTERED
PROFESSIONAL ENGINEER
9/24/18

Kurt D. Merriman, P.E.
Senior Principal Engineer

Attachments: Figure 1: Vicinity Map
Figure 2: Site and Exploration Plan
Appendix: Exploration Logs
Laboratory Test Results
Infiltration Test Data



MONITORING WELL, AESI

MONITORING WELL, TERRA ASSOC. 2017

STAFF GAUGE

DATA SOURCES / REFERENCES:
USGS: 7.5' SERIES TOPOGRAPHIC MAPS, ESRI/I-CUBED/NGS 2013
KING CO: STREETS, PARCELS, CITY LIMITS 1/18
TERRA ASSOC. REPORT KENSINGTON ENCLAVE, 12/17, REVISED 7/27/18

LOCATIONS AND DISTANCES SHOWN ARE APPROXIMATE

N

0 750 1500

FEET

NOTE: BLACK AND WHITE REPRODUCTION OF THIS COLOR ORIGINAL MAY REDUCE ITS EFFECTIVENESS AND LEAD TO INCORRECT INTERPRETATION

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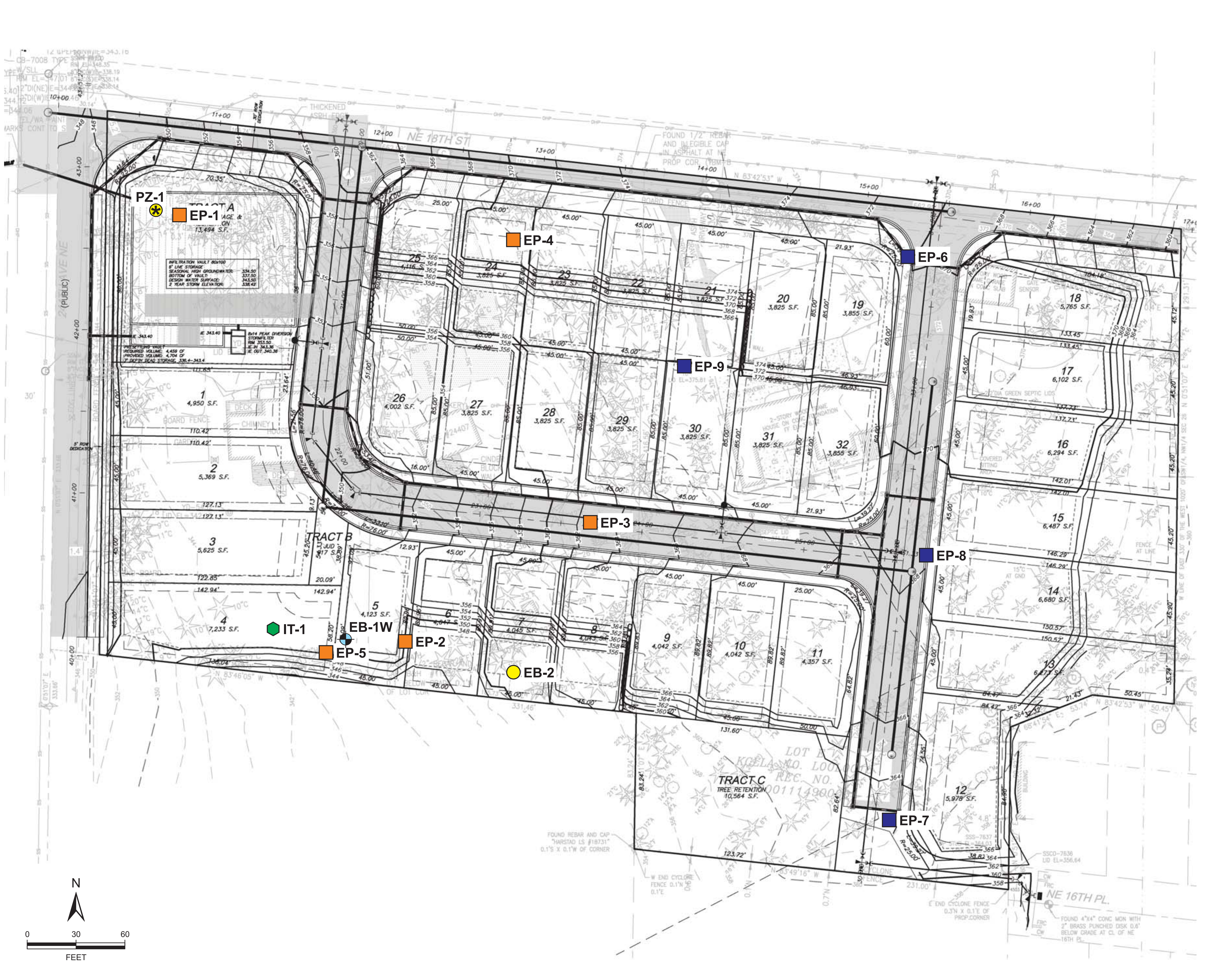
VICINITY MAP

SHEEHAN PROPERTY
KING COUNTY, WASHINGTON

PROJ NO.	DATE:	FIGURE:
180351E001	3/18	1

Document Path: G:\GIS_Projects\aaY2018\180351 Sheehan Property\mxd\180351E001 F1 VM_Sheehan.mxd



LEGEND:

- EB EXPLORATION BORING - MARCH 2018
- EBW MONITORING WELL - MARCH 2018
- EP EXPLORATION PIT - MARCH 2018
- IT INFILTRATION TEST - MARCH 2018
- PIEZOMETER - MARCH 2018
- EP EXPLORATION PIT - JULY 2018

CONTOUR INTERVAL = 2'

NOTE: LOCATION AND DISTANCES SHOWN ARE APPROXIMATE.

NOTES:

1. BASE MAP REFERENCE: D.R. STRONG CONSULTING ENGINEERS, DELAPPE SHEEHAN / PRELIMINARY SUBDIVISION LAYOUT INFILTRATION VAULT OPTION, SHEET 1 OF 1, 6/28/18

BLACK AND WHITE REPRODUCTION OF THIS COLOR ORIGINAL MAY REDUCE ITS EFFECTIVENESS AND LEAD TO INCORRECT INTERPRETATION.

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SITE AND EXPLORATION PLAN

DELAPPE SHEEHAN ASSEMBLAGE
KING COUNTY, WASHINGTON

PROJ NO.	DATE:	FIGURE:
180351E001	7/18	2

180351 Delappe Sheehan \ 180351E001 F2 S-E Plan.cdr

APPENDIX

Coarse-Grained Soils - More than 50%[1] Retained on No. 200 Sieve	Gravels - More than 50%[1] of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines[5]	GW	Well-graded gravel and gravel with sand, little to no fines
			GP	Poorly-graded gravel and gravel with sand, little to no fines
		≥12% Fines[5]	GM	Silty gravel and silty gravel with sand
			GC	Clayey gravel and clayey gravel with sand
	Sands - 50%[1] or More of Coarse Fraction Passes No. 4 Sieve	≤5% Fines[5]	SW	Well-graded sand and sand with gravel, little to no fines
			SP	Poorly-graded sand and sand with gravel, little to no fines
		≥12% Fines[5]	SM	Silty sand and silty sand with gravel
			SC	Clayey sand and clayey sand with gravel
Fine-Grained Soils - 50%[1] or More Passes No. 200 Sieve	Silts and Clays Liquid Limit Less than 50		ML	Silt, sandy silt, gravelly silt, silt with sand or gravel
			CL	Clay of low to medium plasticity; silty, sandy, or gravelly clay, lean clay
			OL	Organic clay or silt of low plasticity
	Silts and Clays Liquid Limit 50 or More		MH	Elastic silt, clayey silt, silt with micaceous or diatomaceous fine sand or silt
			CH	Clay of high plasticity, sandy or gravelly clay, fat clay with sand or gravel
			OH	Organic clay or silt of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

Terms Describing Relative Density and Consistency

	Density	SPT[2] blows/foot
Coarse-Grained Soils	Very Loose	0 to 4
	Loose	4 to 10
	Medium Dense	10 to 30
	Dense	30 to 50
	Very Dense	>50
	Consistency	**SPT[2] blows/foot**
Fine-Grained Soils	Very Soft	0 to 2
	Soft	2 to 4
	Medium Stiff	4 to 8
	Stiff	8 to 15
	Very Stiff	15 to 30
	Hard	>30

Test Symbols

G = Grain Size
M = Moisture Content
A = Atterberg Limits
C = Chemical
DD = Dry Density
K = Permeability

Component Definitions

Descriptive Term	Size Range and Sieve Number
Boulders	Larger than 12"
Cobbles	3" to 12"
Gravel	3" to No. 4 (4.75 mm)
Coarse Gravel	3" to 3/4"
Fine Gravel	3/4" to No. 4 (4.75 mm)
Sand	No. 4 (4.75 mm) to No. 200 (0.075 mm)
Coarse Sand	No. 4 (4.75 mm) to No. 10 (2.00 mm)
Medium Sand	No. 10 (2.00 mm) to No. 40 (0.425 mm)
Fine Sand	No. 40 (0.425 mm) to No. 200 (0.075 mm)
Silt and Clay	Smaller than No. 200 (0.075 mm)

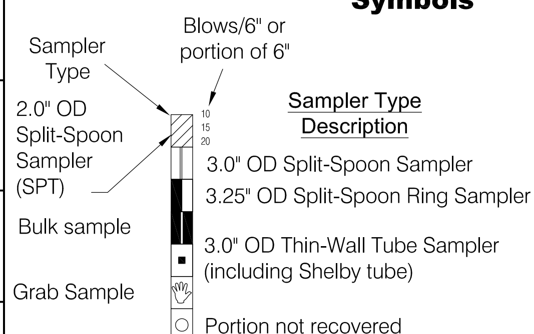
(3) Estimated Percentage

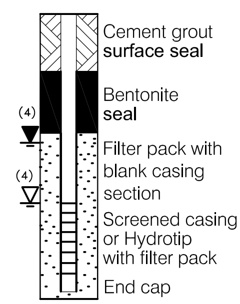
Component	Percentage by Weight
Trace	<5
Some	5 to <12
Modifier (silty, sandy, gravelly)	12 to <30
Very *modifier* (silty, sandy, gravelly)	30 to <50

Moisture Content

Dry - Absence of moisture, dusty, dry to the touch
Slightly Moist - Perceptible moisture
Moist - Damp but no visible water
Very Moist - Water visible but not free draining
Wet - Visible free water, usually from below water table

Symbols





(1) Percentage by dry weight
(2) (SPT) Standard Penetration Test (ASTM D-1586)
(3) In General Accordance with Standard Practice for Description and Identification of Soils (ASTM D-2488)
(4) Depth of ground water
ATD = At time of drilling
Static water level (date)
(5) Combined USCS symbols used for fines between 5% and 12%

Classifications of soils in this report are based on visual field and/or laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field or laboratory testing unless presented herein. Visual-manual and/or laboratory classification methods of ASTM D-2487 and D-2488 were used as an identification guide for the Unified Soil Classification System.

blocks \ dwg \ log_key.dwg LAYOUT: Layout 4 -2014 Qty Chng

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EXPLORATION LOG KEY

FIGURE A1

LOG OF EXPLORATION PIT NO. EP-1

Depth (ft)

This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplfication of actual conditions encountered.

DESCRIPTION

Fill

Loose, very moist, dark brown, silty, SAND, some gravel; areas of (5/8 inch) minus crushed rock; abundant roots (SM).

Weathered Vashon Recessional Outwash

Loose to medium dense, moist to very moist, reddish brown, gravelly, silty, SAND; abundant roots (SM).

Vashon Recessional Outwash

Medium dense, moist, grayish tan, very gravelly, fine to medium SAND; contains pockets of clayey, gravelly, sand (SP/SM).

Sand becomes medium to coarse grained, trace silt (SP).

Vashon Ice-Contact

Dense, moist, grayish tan, very gravelly, silty, SAND; cemented (SM).

Dense, moist, tan gray, very gravelly, medium to coarse SAND, trace silt; contains thin (<3 inches thick) silty lenses from 7 to 7.5 feet (till-like) (SP).

Becomes wet.

Bottom of exploration pit at depth 10 feet
Slow seepage at 10 feet. No caving.

Note: Installed a 1.25 inch piezometer to 10 feet; piezometer hand drilled (perforated) 7.5 to 10 feet.
Top of casing at 2.5 inches above ground surface.
Depth to water in piezometer is 9.4 feet below top of casing.

LOG OF EXPLORATION PIT NO. EP-2

This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplfication of actual conditions encountered.

Depth (ft)	DESCRIPTION
	Fill Loose to medium dense, very moist, dark brown, gravelly, very silty, SAND (SM).
1	**Weathered Vashon Recessional Outwash** Loose to medium dense, very moist, reddish brown, gravelly, silty, SAND; abundant roots (SM).
2	
3	**Vashon Ice-Contact** Medium dense, moist, grayish tan, very gravelly, medium to coarse SAND, some silt; scattered cobbles (SM/SP).
4	
5	Becomes denser.
6	Trace silt. Contains pockets/lenses of silty, very gravelly, SAND up to 12 inches thick (till-like).
7	
8	Some silt (SP/SM). Silty, till-like lenses becomes more frequent below 8 feet.
9	
10	
11	Bottom of exploration pit at depth 10 feet No seepage. No caving.
12	
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20	

Eiseles Delappe Assemblage
King County, WA

Logged by: NS

Approved by: JHS

associated earth sciences incorporated

Project No. 180096E001

3/2/18

LOG OF EXPLORATION PIT NO. EP-3

This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplfication of actual conditions encountered.

Depth (ft)	DESCRIPTION
	Forest Duff / Topsoil - 3 inches
1	**Weathered Vashon Lodgement Till** Loose to medium dense, moist, reddish brown, very silty, gravelly, SAND; abundant roots; wet at base (SM).
2	**Vashon Lodgement Till** Very dense, moist, grayish tan, very silty, fine SAND; nonstratified (SM).
3	
4	Becomes very moist below 4 feet.
5	
6	
7	
8	Bottom of exploration pit at depth 7 feet Slow, discontinuous seepage at ~1.5 feet. No caving.
9	
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KCTP3 180096.GPJ March 23, 2018

Eiseles Delappe Assemblage
King County, WA

Logged by: NS

Approved by: JHS

associated earth sciences incorporated

Project No. 180096E001

3/2/18

LOG OF EXPLORATION PIT NO. EP-4

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplfication of actual conditions encountered. **DESCRIPTION**
	Forest Duff / Topsoil - 4 inches
1	**Weathered Vashon Recessional Outwash** Loose to medium dense, very moist, reddish brown, gravelly, silty, SAND; abundant roots (SM).
2	**Vashon Recessional Outwash**
3	Medium dense, moist, tan gray, very gravelly, medium to coarse SAND; contains thin (<1 inch thick) lenses of silty, fine sand (SP).
4	**Vashon Lodgement Till** Very dense, moist, grayish tan, very silty, gravelly, SAND; scattered cobbles and boulders; nonstratified (SM).
5	
6	
7	
8	Bottom of exploration pit at depth 7 feet No seepage. No caving.
9	
10	
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19	
20	

KCTP3 180096.GPJ March 23, 2018

Eiseles Delappe Assemblage
King County, WA

Logged by: NS
Approved by: JHS

associated earth sciences incorporated

Project No. 180096E001

3/2/18

LOG OF EXPLORATION PIT NO. IT-1

This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplfication of actual conditions encountered.

Depth (ft)	DESCRIPTION
1	**Topsoil / Forest Duff** Loose, moist, dark brown, very silty, medium SAND, some gravel; abundant organics (SM).
2	**Weathered Vashon Recessional Outwash** Loose, moist, orangish brown, silty, fine SAND, some roots, trace gravel (SM).
3 4 5	**Vashon Recessional Outwash** Loose to medium dense, moist, gray with faint iron oxide staining, gravelly, medium to coarse SAND, trace silt; occasional cobbles; faintly bedded with gravel (~1 to 2 inches thick) every 6 inches to 1 foot (SW).
6 7 8 9 10 11	**Vashon Ice-Contact** Clasts of silty, coarse SAND; unsorted. Difficult digging, very hard in eastern wall of pit, some bedding (3 to 6 inches thick) of alternating silty, SAND and sandy, SILT. Occasional roots up to 7 feet. Medium dense to dense, very moist, light brown, very silty, coarse SAND (SM).
12 13 14 15 16 17 18 19 20	Bottom of exploration pit at depth 11 feet Slow seepage at 6.5 feet and slow to moderate seepage at 9.5 feet interpreted as return flow from infiltration test; pooling at 11 feet. Minor to moderate caving 2.5 to 5.5 feet. Infiltration test performed at 5 feet.

Eisles Delappe Assemblage
King County, WA

Logged by: NS

Approved by: CJK

associated earth sciences incorporated

Project No. 180096E002

3/27/18

LOG OF EXPLORATION PIT NO. EP-5

This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplfication of actual conditions encountered.

Depth (ft)	DESCRIPTION
	Topsoil
	Loose, moist, dark brown, silty, SAND, trace gravel; abundant organics (SM).
1	**Weathered Vashon Recessional Outwash**
	Loose, moist, orangish brown, very silty, SAND, some gravel; abundant roots (SM).
2	**Vashon Recessional Outwash**
3	Loose, moist, gray with iron oxide, gravelly, medium to coarse SAND, trace silt (SP). Loose to medium dense.
4	Excavator notes tighter at 4 feet, trace to some silt.
5	Excavator notes medium dense at 5 feet.
6	Silty beds (1 to 2 inches thick); clasts of medium dense, moist, light brown, SILT, some sand to sandy. Till-like clasts of very sandy, SILT with heavy iron oxide staining. Increasing silt content.
7	**Vashon Ice-Contact**
8	Difficult digging. Medium dense to dense, moist, light brown, very silty, medium SAND, some gravel (SM); contains clasts of sandy, SILT (ML); unsorted.
9	Medium dense to dense, very moist, brownish gray, medium SAND, some silt, trace gravel; some lightly consolidated clasts (SM).
10	
11	Medium dense to dense, very moist, light grayish brown, very silty, medium SAND, some gravel; slightly tacky (SM).
12	
13	Bottom of exploration pit at depth 12 feet No seepage. Minor to moderate caving 2.5 to 4.5 feet, very minor caving 4.5 to 7 feet.
14	
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Eisles Delappe Assemblage
King County, WA

Logged by: NS

Approved by: CJK

associated earth sciences incorporated

Project No. 180096E002

3/27/18

Geologic & Monitoring Well Construction Log

associated earth sciences incorporated

Project Number	Well Number	Sheet
180096E002	EB-1W	1 of 1

Project Name	Eiseles Delappe Assemblage	Location	King County, WA
Elevation (Top of Well Casing)	343.18	Surface Elevation (ft)	343.36
Water Level Elevation	15.5	Date Start/Finish	3/28/18,3/29/18
Drilling/Equipment	Environmental Drilling / B53	Hole Diameter (in)	6 inches
Hammer Weight/Drop	140# / 30"		

Depth (ft)	Water Level	WELL CONSTRUCTION	Blows/ 6"	DESCRIPTION
0		Flush Mount Monument Concrete Seal 0 to 2 feet		**Topsoil / Forest Duff** Loose, moist, dark brown, very silty, SAND, trace gravel; abundant organics (SM).
2		Bentonite 2 to 17 feet	5 34 50/2"	**Vashon Recessional Outwash** Very dense, moist, gray with faint iron oxide, gravelly, silty, coarse SAND; contains broken rocks (SM).
5		2-inch I.D. Sch. 40 PVC well casing 0 to 20 feet	30 50/6"	Very dense, moist, gray, silty, gravelly, coarse SAND; difficult drilling with beds (1 inch thick) of light brown, sandy, silt, trace gravel (SM/ML).
7.5			22 26 34	**Vashon Ice-Contact** Very dense, moist, light brown, very silty, coarse SAND, some gravel; cobbly drill chatter with beds (<1 inch thick) of very moist, gray, coarse SAND, trace silt; broken gravel; lightly consolidated (SM).
10			24 36 50/3"	Very dense, moist, gray, brown, silty, medium SAND, trace gravel; occasional large broken gravel; massive (SM).
12.5			17 32 45	Very dense, very moist to wet, light brown, silty to very silty, medium SAND, trace to some gravel; occasional rounded gravel; one angular black fragment; slightly tacky (SM).
15	▼		28 50/3"	**Vashon Advance Outwash** Very dense, wet, dark gray, coarse SAND, some gravel to gravelly, some tan silt; massive (SP).
17.5		10/20 Colorado silica sand pack 17 to 30 feet	50/6"	Very dense, very wet, tan, silty, coarse SAND, some gravel (SM). Very dense, wet, gray, SAND, some silt, trace gravel (SP).
20		2-inch I.D. Sch. 40 PVC well screen 20 to 30 feet	47 50/3"	Very dense, wet, brown gray, coarse SAND, some silt, trace gravel; occasional interbeds (1 to 2 inches thick) of gravel (SP).
22			36 50/3"	Very dense, wet, gray, very gravelly, coarse SAND, trace silt; bedded (2 to 3 inches thick) with medium SAND, round, fine gravel, and occasional large gravel; some drill chatter (SP).
25			36 50/2"	Very dense, wet, gray, rounded, fine GRAVEL, some sand to sandy, trace silt; contains a thick bed of very coarse SAND in upper 1/3 of the spoon; driller notes blowcounts are overstated; looks pebbly???? (GP). Very dense, wey brown gray, gravelly, coarse SAND, trace to some silt (SP).
27.5			37 50/5"	2 inch thick bed of fine to medium SAND, trace to some silt. Moist, gray, silt clast in shoe.
30		Threaded end cap Well Tag # BKH-531	26 32 50/4"	Very dense, wet, brownish gray, very sandy, rounded, fine GRAVEL, trace silt; thick bed of medium to coarse SAND in the top 1/3 of spoon (GP).
31.5				Boring terminated at 31.5 feet. Well completed at 30 feet on 3/29/18. Groundwter encountered at 15.5 feet.
35				

Sampler Type (ST):

- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- No Recovery
- Ring Sample
- Shelby Tube Sample
- M - Moisture
- Water Level ()
- Water Level at time of drilling (ATD)

Logged by: NS
Approved by: CJK

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Exploration Log

Project Number	Exploration Number	Sheet
180096E002	EB-2	1 of 1

Project Name	Eiseles Delappe Assemblage	Ground Surface Elevation (ft)	349.97
Location	King County, WA	Datum	NAVD88
Driller/Equipment	Environmental Drilling / B53	Date Start/Finish	3/28/18,3/29/18
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	6 inches

Depth (ft)	Samples	DESCRIPTION	Water Level	Blows/6"	Blows/Foot
		Topsoil / Forest Duff			
		Vashon Ice-Contact			
5	S-1	Very dense, moist, light brownish gray, silty, coarse SAND, some gravel to gravelly; broken gravel near shoe; some till like clasts (SM).		16 50/6"	50/6"
10	S-2	Very dense, moist, light brown, very silty, gravelly, medium to coarse SAND; slightly plastic; rock fragments in shoe, blowcounts likely overstated; non-bedded (SM).		50/6"	50/6"
	S-3	Very dense, moist, light grayish brown, silty, medium SAND, some gravel to gravelly; "jumbled"; non-bedded (SM).		50/6"	50/6"
15	S-4	Very dense, very moist, light gray brown, silty to very silty, medium SAND, some gravel; lightly consolidated; slightly plastic; driller notes rock while driving hammer; non-bedded (SM).		42 50/3"	50/3"
	S-5	Very dense, very moist, brownish gray, silty, medium SAND, trace fine gravel; faintly bedded (<1 inch thick); fine rounded gravel (SM).		50/6"	50/6"
20	S-6	Very dense, wet, light brown, silty, medium SAND, trace gravel; occasional large gravel; massive (SP).	ATD	26 50/6"	50/6"
		Vashon Advance Outwash			
	S-7	Very dense, light brown and dark gray, medium to coarse SAND, trace to some silt, trace gravel; contains beds (1 inch thick) of rounded gravel and beds (3 inches thick) of fine to medium sand (SP-SM).		40 39 46	85
25	S-8	Very dense, wet, light brown and dark gray, slightly silty, very sandy, fine GRAVEL; faintly bedded (<1 inch thick) with alternating fine gravel and coarse sand; upper 1/3 of spoon is fairly clean medium sand, interpreted as slough (GP-GM).		5 16 50/5"	50/5"
	S-9	Very dense, wet, light brown and gray, slightly silty, medium to coarse SAND, trace gravel; driller suggests heave; contains a bed (4 inches thick) of silty, medium sand near bottom; black fragment in shoe (SP-SM).		5 7 50/3"	50/3"
30	S-10	Very dense, wet, dark gray, medium to coarse SAND, trace silt, trace gravel; contains bed (1 inch thick) of fine gravel near shoe (SP).		11 50/4"	50/4"
	S-11	Very dense, wet, dark gray, medium SAND, grading to coarse SAND to GRAVEL, trace silt (SP/GP).		39 50/2"	50/2"
35		Bottom of exploration boring at 34 feet Groundwter encountered at 21 feet.			

Sampler Type (ST):

- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- No Recovery
- Ring Sample
- Shelby Tube Sample
- M - Moisture
- Water Level ()
- Water Level at time of drilling (ATD)

Logged by: NS
Approved by: CJK

LOG OF EXPLORATION PIT NO. EP-6

Depth (ft)	This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplfication of actual conditions encountered. **DESCRIPTION**
	Forest Duff / Organic Debris - 6 inches
	Fill
1	Medium stiff, dry, light brown, gravelly, sandy, SILT; occasional cobbles and rubbish (wires, metal, plastic) (ML).
2	**Vashon Lodgement Till**
3	Dense, slightly moist, light olive with faint iron oxide, very silty, gravelly, fine SAND; occasional cobbles; lightly cemented; unsorted (SM).
	Roots 0 to 4 feet.
4	As above, stiff to hard, less to no iron oxide, occasional cobbles; cemented.
5	
6	
7	Bottom of exploration pit at depth 6 feet No seepage. No caving.
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9	
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15	
16	
17	
18	
19	
20	

KCTP3 180351.GPJ July 25, 2018

Sheehan Property
Sammamish, WA

Logged by: NS

Approved by: CJK

associated earth sciences incorporated

Project No. 180351E001

7/16/18

LOG OF EXPLORATION PIT NO. EP-7

This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplfication of actual conditions encountered.

Depth (ft)	DESCRIPTION
	Fill
1	Medium stiff, dry, light brown, gravelly, sandy, SILT (ML).
	Weathered Vashon Lodgement Till
2	Medium stiff, dry, light tan, gravelly, sandy, SILT; occasional cobbles; abundant roots (ML).
	Vashon Lodgement Till
3	Very dense, moist, light olive, very silty, gravelly, fine SAND; occasional cobbles; cemented; unsorted (SM).
4	Roots 0 to 4 feet.
5	
6	Increasing moisture content, darker olive color.
7	Increasing sand content.
8	Very dense, moist, silty, gravelly, fine SAND; cemented; unsorted (SM).
9	Bottom of exploration pit at depth 8.5 feet No seepage. No caving.
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KCTP3 180351.GPJ July 25, 2018

Sheehan Property
Sammamish, WA

Logged by: NS

Approved by: CJK

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Project No. 180351E001

7/16/18

LOG OF EXPLORATION PIT NO. EP-8

This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplfication of actual conditions encountered.

Depth (ft)	DESCRIPTION
	Forest Duff / Topsoil - 4 inches
	Weathered Vashon Lodgement Till
1	Soft, dry, orangish brown, gravelly, sandy, SILT; occasional cobbles; abundant roots (ML).
2	**Vashon Lodgement Till**
3	Medium dense, slightly moist, light olive, very silty, gravelly, fine SAND; occasional cobbles; occasional roots; cemented; unsorted (SM).
	Grades to hard.
4	Roots 0 to 3.5 feet.
5	Very dense, slightly moist, light gray, silty, gravelly, fine SAND; occasional to frequent cobbles; cemented; unsorted (SM).
6	Grades to moist.
7	Moist, as above.
8	Bottom of exploration pit at depth 7.5 feet No seepage. No caving.
9	
10	
11	
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13	
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17	
18	
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20	

Sheehan Property
Sammamish, WA

Logged by: NS
Approved by: CJK

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Project No. 180351E001

7/16/18

LOG OF EXPLORATION PIT NO. EP-9

This log is part of the report prepared by Associated Earth Sciences, Inc. (AESI) for the named project and should be read together with that report for complete interpretation. This summary applies only to the location of this trench at the time of excavation. Subsurface conditions may change at this location with the passage of time. The data presented are a simplfication of actual conditions encountered.

Depth (ft)	DESCRIPTION
	Forest Duff - 4 inches
	Weathered Vashon Lodgement Till / Fill
1	Eastern half of pit: Medium stiff, slightly moist, orangish brown, gravelly, sandy, SILT (ML).
	West half of pit: Lens of clean, well graded sand and fine gravel with occasional cut rock blocks.
2	**Vashon Lodgement Till**
	Roots 0 to 3 feet.
3	
4	Dense, moist, light gray, silty, gravelly, fine SAND; occasional cobbles; cemented; unsorted (SM).
5	
6	Occasional boulders.
7	As above.
8	Bottom of exploration pit at depth 7.5 feet No seepage. Minor caving 0 to ~2 feet.
9	
10	
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KCTP3 180351.GPJ July 25, 2018

Sheehan Property
Sammamish, WA

Logged by: NS

Approved by: CJK

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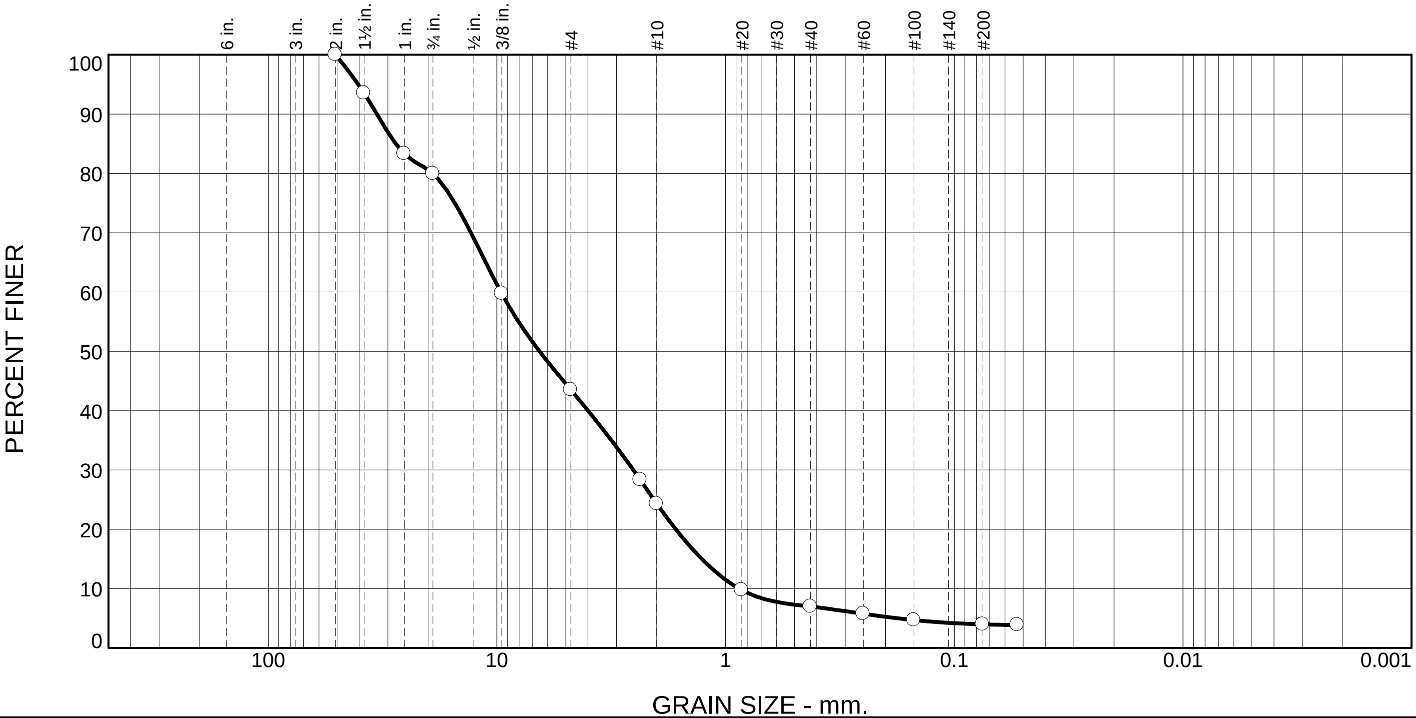
Project No. 180351E001

7/16/18

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	20.1	36.4	19.2	17.3	3.0	4.0	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
2	100.0		
1.5	93.5		
1	83.3		
.75	79.9		
.375	59.7		
#4	43.5		
#8	28.3		
#10	24.3		
#20	9.8		
#40	7.0		
#60	5.8		
#100	4.7		
#200	4.0		
#270	3.8		

* (no specification provided)

Material Description

very sandy, GRAVEL, trace silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=**

Classification

USCS (D 2487)= GP **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 33.5251 D_{85}= 27.7127 D_{60}= 9.6073
D_{50}= 6.5137 D_{30}= 2.5325 D_{15}= 1.2715
D_{10}= 0.8730 C_u= 11.00 C_c= 0.76

Remarks

Collected by: SNS

Date Received: 03/13/2018 **Date Tested:** 03/15/2018

Tested By: BN

Checked By: KDM

Title:

Location: Onsite
Sample Number: EP-1 **Depth:** 7.5'-8'
Date Sampled: 03/02/2018

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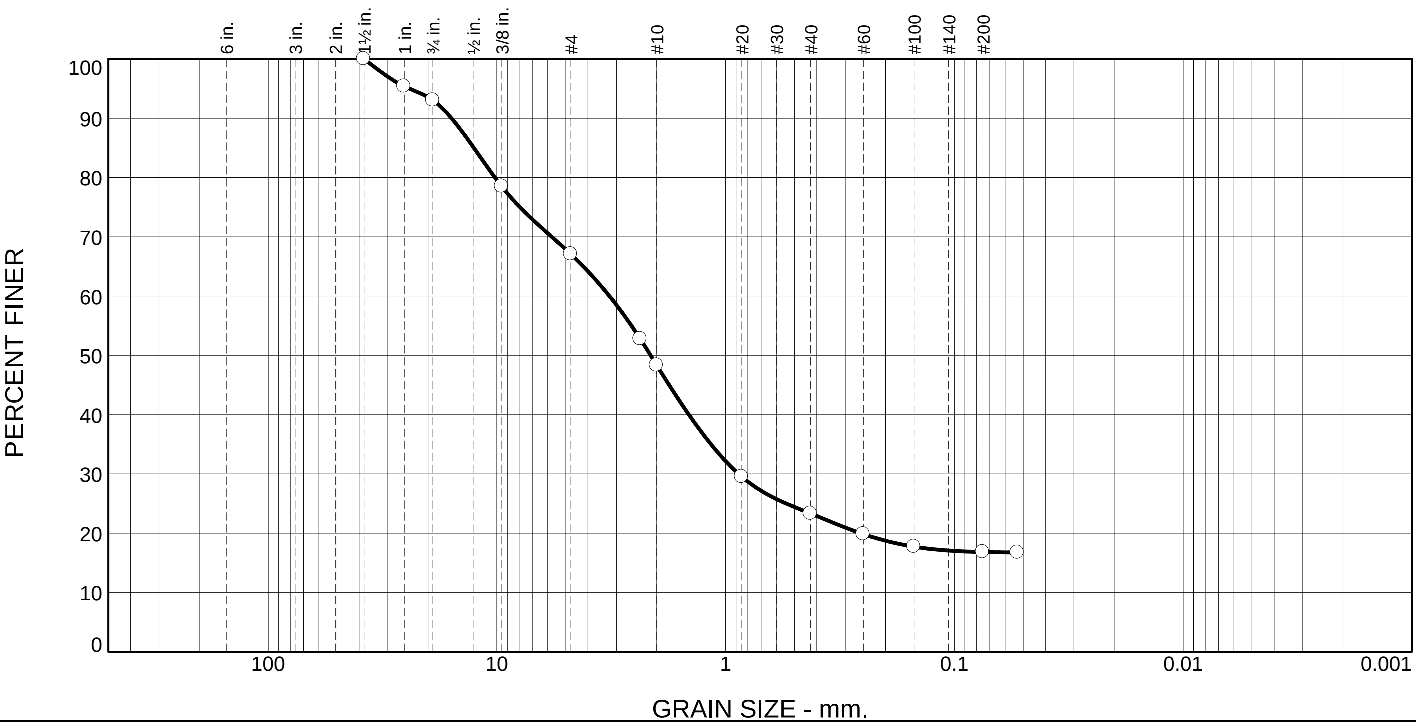
Client: Toll Bros. Inc
Project: Eiseles Dleappe Assemblge

Project No: 180096 E001 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	7.0	25.9	18.8	25.0	6.5	16.8	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5	100.0		
1	95.4		
.75	93.0		
.375	78.5		
#4	67.1		
#8	52.8		
#10	48.3		
#20	29.5		
#40	23.3		
#60	19.8		
#100	17.7		
#200	16.8		
#270	16.7		

* (no specification provided)

Material Description

very gravelly, silty, SAND

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=**

Classification

USCS (D 2487)= SM **AASHTO (M 145)=** A-1-b

Coefficients

D_{90}= 15.7744 D_{85}= 12.6043 D_{60}= 3.2197
D_{50}= 2.1285 D_{30}= 0.8785 D_{15}=
D_{10}= C_u= C_c=

Remarks

Collected by: SNS

Date Received: 03/13/2018 **Date Tested:** 03/15/2018

Tested By: BN

Checked By: KDM

Title:

Location: Onsite
Sample Number: EP-2 **Depth:** 9'-10'
Date Sampled: 03/02/2018

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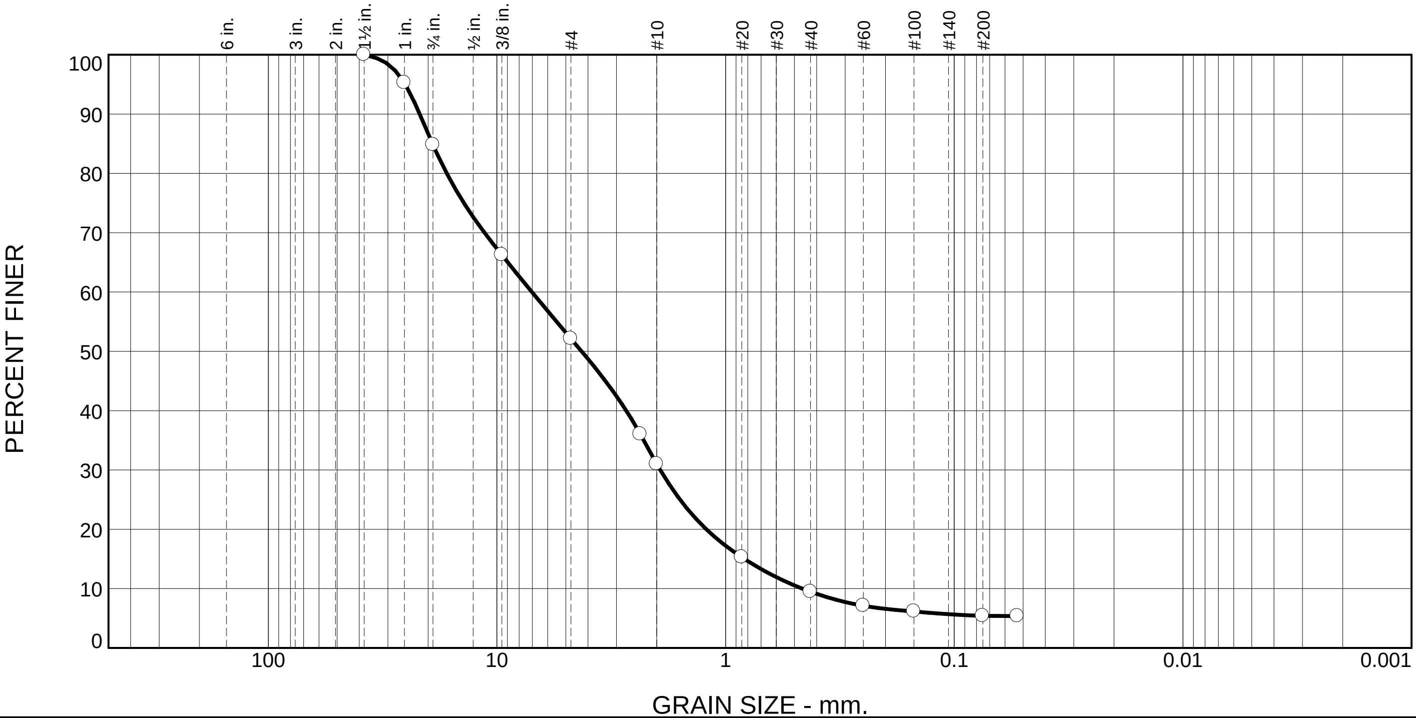
Client: Toll Bros. Inc
Project: Eiseles Dleappe Assemblge

Project No: 180096 E001 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	15.2	32.7	21.1	21.5	4.1	5.4	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
1.5	100.0		
1	95.3		
.75	84.8		
.375	66.3		
#4	52.1		
#8	36.0		
#10	31.0		
#20	15.3		
#40	9.5		
#60	7.1		
#100	6.1		
#200	5.4		
#270	5.4		

* (no specification provided)

Material Description

very sandy GRAVEL, some silt

Atterberg Limits (ASTM D 4318)

PL= NP **LL=** NV **PI=**

Classification

USCS (D 2487)= GW-GM **AASHTO (M 145)=** A-1-a

Coefficients

D_{90}= 21.7650 D_{85}= 19.1423 D_{60}= 7.0325
D_{50}= 4.2633 D_{30}= 1.9319 D_{15}= 0.8272
D_{10}= 0.4626 C_u= 15.20 C_c= 1.15

Remarks

Collected by: SNS

Date Received: 08/01/2018 **Date Tested:** 08/03/2018

Tested By: BN

Checked By: KDM

Title:

Location: Onsite
Sample Number: IT-5 **Depth:** 5'
Date Sampled: 03/27/2018

a s s o c i a t e d
e a r t h s c i e n c e s
i n c o r p o r a t e d

Client: Toll Bros. Inc
Project: Eiseles Dleappe Assemblge
Project No: 180096 E001 **Figure**

FIELD INFILTRATION TESTING DATA

Project Name	Eiseles Delappe Assemblage	**Water Source**	Water Truck
Project No.	180096E001	**Meter**	AESI FM XXXX
Date	3/27/2018	**Pit Area**	3ft x 4 1/2ft = 13 1/2 ft
Test No.	IT-1	**Test Depth**	5 ft
Performed By	SNS	**Test Material**	Recessional outwash

Time (24-hr)	**Totalizer (gallon)**	**Flow Rate (gpm)**	**Stage (ft)**	**Comments**
9:21:00	0.0	20.3	0.00	Flow on.
9:36:00	99.9	4.7	0.50	Decrease flow.
9:51:00	155.2	3.64	0.56	
10:08:00	214.0	3.36	0.60	Reduced flow
10:23:00	263.7	3.17	0.61	
10:38:00	307.8	3.06	0.62	
10:53:00	353.6	3.06	0.62	Minor sloughing at 3 1/2 to 4 1/2 feet.
11:08:00	399.5	3.04	0.63	
11:09:00	402.4	0	0.63	Switch flow meter. Water off.
11:13:00	402.4	2.02	0.54	Water on. Valve fully open.
11:28:00	432.1	2.03	0.50	
11:43:00	462.6	2.03	0.47	
12:01:00	494.8	2.03	0.44	
12:09:00		2.77		Pump on to increase pressure.
12:16:00	536.7	2.6	0.52	
12:31:00	575.7	2.38	0.54	
12:46:00	608.0	2.2	0.55	
13:01:00	640.8	2.21	0.55	
13:16:00	674.9	2.22	0.55	
13:33:00	711.8	2.23	0.55	
13:53:00	757.1	2.23	0.55	
13:58:00	767.2	2.22	0.55	Flow meter check with bucket. Flow stopped.
14:04:00	772.4	2.21	0.42	Flow on. Interim FH rate 15.6 iph
14:19:00	805.2	2.22	0.46	
14:35:00	840.9	2.22	0.48	
14:50:00	873.6	2.16	0.50	
15:05:00	905.6	2.13	0.50	
15:21:00	939.0	2.09	0.50	Begin last hour of constant head.
15:31:00	954.8	2.09	0.50	
15:41:00	980.9	2.09	0.50	
15:51:00	1001.7	2.09	0.50	
16:01:00	1022.6	2.09	0.50	
16:11:00	1043.6	2.09	0.50	
16:21:00	1064.1	2.09	0.50	Flow off, begin falling head
16:22:40			0.46	
16:24:10			0.44	
16:25:50			0.40	
16:27:45			0.36	Area decreasing, about 12.5 sf
16:29:50			0.32	
16:31:30			0.28	Area decreasing, about 11 sf
16:33:40			0.22	Area decreasing, about 10.5 sf
16:36:15			0.16	Wetted area not representative (22"x34" = 5.2 sf). Test terminated.