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Subsurface Exploration, Geologic Hazard, and Geotechnical Engineering Report

BEAR CREEK PARKWAY PROPERTY

Redmond, Washington

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Prepared for:

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I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of our subsurface exploration, geologic hazard, and geotechnical engineering study for the subject project. The site location is shown on the "Vicinity Map," Figure 1. The approximate locations of the existing site structures and the explorations completed for this study are shown on the "Site and Exploration Plan," Figure 2. Logs of the subsurface explorations and copies of laboratory test results completed for this study are included in Appendix A and Appendix B, respectively.

1.1 Purpose and Scope

The purpose of this study was to provide subsurface data to be used in the design and development of the subject project. This study included a review of selected available geologic literature, advancing six hollow-stem auger soil borings, and performing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow ground water. Geotechnical engineering studies were completed to establish recommendations for the type of suitable foundations, floor support, anticipated liquefaction-induced settlement, lateral earth pressures, and drainage considerations. A brief discussion on infiltration feasibility on-site is presented in our report, though additional grain-size or infiltration testing may be required by the City of Redmond to support a final infiltration design. This report summarizes our fieldwork and offers geotechnical engineering recommendations based on our present understanding of the property and potential future development. We recommend that we be allowed to review the recommendations presented in this report and revise them, if needed, as the project develops and a design is finalized.

1.2 Authorization

Authorization to proceed with this study was granted by a signed contract received from MainStreet Property Group, LLC (MSPG). Our work was completed in general accordance with our contract dated January 7, 2015. This report has been prepared for the exclusive use of MSPG and their agents for specific application to this project. Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering practices in effect in this area at the time our report was prepared. No other warranty, express or implied, is made.

2.0 SITE AND PROJECT DESCRIPTION

The project site is a commercial property comprised of six contiguous parcels and a portion of a seventh including Parcels 927070-0010, -0020, -0025, -0030, -0035, -0040, and a portion of Parcel 112505-9026. The addresses of existing commercial buildings located on the site include 15806, 15810, 15814, and 15904 Bear Creek Parkway. The eastern portion of the site appears to have been recently graded and slopes up toward the east. This area is vegetated by low field grass with areas of bare soil. The remainder of the site (west of Parcel 927070-0040) is developed and relatively flat to gently sloping down toward the west. The developed portion of the site contains four commercial buildings. The areas surrounding the existing buildings consist largely of asphalt pavement with one area of crushed rock surfacing located behind the building at the west end of the site (15806 Bear Creek Parkway).

Based on our current understanding, the project would consist of demolition of the existing structures and construction of one or more mixed-use buildings similar to other new construction in the downtown Redmond area.

3.0 SUBSURFACE EXPLORATION

Subsurface exploration completed for this project included advancing six hollow-stem auger soil borings to depths of approximately 21.5 to 41.5 feet. These borings are identified on Figure 2 as borings EB-1 through EB-6. Copies of exploration logs for these borings are included in Appendix A. The conclusions and recommendations presented in this report are based, in part, on the explorations completed for this study. The locations and depths of the explorations were completed within site and budget constraints.

3.1 Exploration Borings

Exploration borings EB-1 through EB-6 were completed using a truck-mounted, hollow-stem auger drill rig. During the drilling process, samples were generally obtained at 2.5- to 5-foot-depth intervals. The exploration borings were continuously observed and logged by an engineering geologist from our firm. The exploration logs presented in Appendix A are based on the field logs, drilling action, and inspection of the samples secured.

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 is recorded within 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 exploration boring logs.

The samples obtained from the split-barrel sampler were classified in the field and placed in watertight containers. The samples were then transported to our laboratory for further visual classification and laboratory grain-size analyses. The laboratory testing results are included in Appendix A.

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 applicable geologic literature.

As shown on the exploration logs included in Appendix A, natural sediments encountered at the site consist primarily of sand and gravel with minor quantities of silt. In the western portion of the site, the sand and gravel was overlain by medium stiff to stiff silt, with interbeds of organic silt and peat to depths of approximately 10 feet. The natural sediments were overlain by fill in the eastern portion of the site. The following section presents more detailed subsurface information organized from the youngest to the oldest sediment types.

Because of the nature of exploratory work below ground, extrapolation of subsurface conditions between field explorations is necessary. It should be noted that subsurface conditions differing from those indicated on the exploration logs may 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 the time of construction.

4.1 Stratigraphy

Fill

Fill soils (those not naturally placed) were encountered in borings EB-1, EB-2, and EB-5. The fill encountered in boring EB-1, located near the top of the elevated area near the east end of the site, generally consisted of medium dense, tan to brown sand with moderately high to high silt and gravel content. At this location the existing fill extended to a depth of approximately 10.5 feet. Fill encountered in boring EB-2, also located in the eastern portion of the site, generally consisted of crushed rock and quarry spalls. At this location, the fill was approximately 2.5 feet thick. Fill encountered in boring EB-5, located in the south-central

portion of the site, generally consisted of gravelly sand with abundant organic debris. The fill encountered in boring EB-5 extended to a depth of approximately 2 feet. The existing fill is not considered suitable for foundation support.

Holocene Alluvium

Sediments encountered below the surficial fill, asphalt pavement, or gravel surfacing generally consisted of medium dense, sandy gravel interbedded with gravelly sand with minor quantities of silt. Although we were unable to visualize maximum gravel size from the sampling method used, it is our experience that the sediments in this geologic unit typically contain coarse gravel and cobbles. In the western portion of the property, at the locations of borings EB-4 and EB-6, the sand and gravel was capped by medium stiff to stiff silt. At the location of boring EB-4, the silt contained interbeds of organic silt and peat. Similar sediments are noted on the boring log for well W-1, also located in the western portion of the site. Where encountered, the silt/peat facies of this unit extended to depths ranging from approximately 6 to 9 feet. Due to the compressible nature of the organic silt and peat, the silt/peat facies of this unit is not considered suitable for foundation support.

We interpret these sediments to be representative of alluvium deposited subsequent to the most recent glaciation of the project region. This geologic time period, known as the Holocene epoch, began approximately 12,000 years ago. Where present below the water table, these sediments appeared to be relatively free-draining. At the locations of borings EB-2, EB-4, and EB-6, the Holocene alluvium extended to depths ranging from approximately 18.5 to 20 feet. At the locations of borings EB-1, EB-3, and EB-5, the Holocene alluvium extended beyond the maximum depth explored of approximately 21.5 feet.

The less gravelly strata within the Holocene alluvium are moderately liquefiable and may result in settlement of buildings directly supported on this material during a strong seismic event. This is discussed further in the “Liquefaction” section of this report. The granular (sand and gravel) portion of the Holocene alluvium may be used in structural fill applications if it can be properly moisture-conditioned and compacted and is specifically allowed by project specifications. The fine-grained (silt and peat) facies of this unit is not recommended for use as structural fill.

Vashon Recessional Outwash

Sediments encountered below the Holocene alluvium in borings EB-2, EB-4, and EB-6 generally consisted of dense to very dense sand with minor quantities of silt and variable gravel content, interbedded with dense to very dense gravel with minor quantities of silt and variable sand content. We interpret these sediments to be representative of Vashon recessional outwash. The Vashon recessional outwash was deposited by meltwater streams that emanated from the retreating glacial ice during the Vashon Stade of the Fraser Glaciation approximately 12,000 to

15,000 years ago. Although is it likely that some of the blow counts obtained during drilling within this unit are overstated due to the presence of coarse gravel, blow counts obtained within the sandier strata within this unit that contained no coarse gravel were still indicative of dense soil conditions. This observation is consistent with the findings of a 2012 GeoEngineers study conducted for the City of Redmond that describes the recessional outwash in this area as "dense." The recessional outwash sediments appeared visually similar to the overlying Holocene alluvium and were primarily distinguishable from it by the elevated density of the outwash. Where encountered in our borings, the recessional outwash sediments extended beyond the maximum depths explored of approximately 21.5 to 41.5 feet.

Geologic Mapping

Review of the regional geologic map of the area titled *Geologic Map of King County*, compiled by Booth, Troost, and Wisher (2007) indicates that the area of the project site is underlain by Quaternary alluvium with Vashon recessional outwash and older, pre-Vashon-aged sediments exposed along the edges of the Sammamish River valley east of the site. Our interpretation of the sediments encountered in our explorations is consistent with the regional geologic map.

4.2 Hydrology

Ground water seepage was encountered in our explorations below depths of approximately 10.5 to 16.5 feet. One exception was boring EB-1, located near the topographic high point of the property near its east end, where no ground water was encountered within the maximum depth explored of approximately 21.5 feet.

Three existing monitoring wells were observed at the site. These are identified on Figure 2 as wells W-1 through W-3. One of these three wells (W-1) had been abandoned prior to our field study. No Washington Department of Ecology (Ecology) well tag was in the monument and we were not able to correlate it with a well log on file with Ecology. Well W-2 did not contain a Ecology well tag, but a tag inside the monument identified it as City of Redmond well #540000MW062. A copy of the log for this well, obtained from the City of Redmond, is included in Appendix A. Well W-3 did contain an Ecology well tag. A copy of the log for this well, obtained from Ecology, is included in Appendix A.

A summary of the ground water levels observed at the site is included in Table 1. Ground water levels measured in wells W-2 and W-3 are included in Table 1. As previously mentioned, well W-1 had been abandoned prior to our study.

Table 1
Summary of Ground Water Levels

Boring/Monitoring Well	Ground Surface Elevation (feet)	Ground Water Level (feet bgs)		Ground Water Elevation (feet)
		At Time of Drilling	Monitoring Well*	
EB-1	48	>21.5	---	<26.5
EB-2	41	16.5	---	24.5
EB-3	40	14.5	---	25.5
EB-4	38	10.5	---	27.5
EB-5	39	13.5	---	25.5
EB-6	38	12	---	26
W-2	42	---	16.7	25.3
W-3	38	---	10.4	27.6

Note: The ground water levels reported for monitoring wells W-2 and W-3 were measured on May 21, 2015.
Ground surface and ground water elevations shown are approximate and are inferred from the plotted locations of the explorations and topographic contours depicted on Figure 2. The topographic contours shown on Figure 2 are based on a topographic survey prepared by Contour Engineering, LLC and are based on the NAVD 88 datum.
Borings EB-1 through EB-4 were drilled on May 20, 2015; borings EB-5 and EB-6 were drilled on May 21, 2015.
feet bgs – feet below ground surface.

The ground water levels shown are interpreted to be representative of the alluvial aquifer, which is likely hydraulically connected to the Sammamish River, located approximately 200 yards west of the subject property.

It should be noted that ground water levels below the site vary in response to such factors as changes in season, precipitation, and off- and on-site land usage. Review of water level data recorded by the City of Redmond for well W-2 (City of Redmond Well ID #540000MW062) for the time period from August 14, 2012 through July 2, 2013 indicates that the water level in this well fluctuated 2.46 feet over the monitoring period. Specifically, a seasonal low ground water surface elevation of 25.01 feet (NAVD 88 datum) was recorded on August 14, 2012 and a seasonal high ground water elevation of 27.47 feet was recorded on February 8, 2013.

Given the gradation of the sediments underlying the site, we anticipate that significant dewatering efforts may be required to control ground water flow into any excavations extending below the water table.

4.3 Laboratory Testing

We completed mechanical grain-size analyses on eight samples collected from our borings. The results of these analyses are included in Appendix B.

5.0 INFILTRATION FEASIBILITY ASSESSMENT

Our assessment of infiltration feasibility includes general surface characterization, subsurface exploration, and grain-size testing. It is our understanding that the project is under the City of Redmond 2012 *Clearing, Grading, and Stormwater Management Technical Notebook* (SWMTN), and the 2005 Washington State Department of Ecology *Stormwater Management Manual for Western Washington* (Ecology Manual), which will provide the guidance for the site suitability criteria for infiltration.

Holocene alluvium consisting of stratified sand and gravel was encountered near the ground surface in the portion of the site from boring EB-5 extending east to boring EB-2. In our opinion, the unsaturated alluvial sand and gravel is a suitable receptor soil for storm water infiltration.

Explorations advanced west of boring EB-5 encountered up to approximately 10 feet of silt and peat overlying the granular alluvium. Due to the low permeability of the silt, infiltration west of boring EB-5 is not recommended. In the eastern portion of the site (the area of boring EB-1) the granular alluvial sediments are overlain by up to approximately 10 feet of fill. The natural alluvial sediments underlying the fill at the location of boring EB-1 appears suitable for infiltration.

Sieve analyses conducted on selected samples of the alluvial sand and gravel collected from borings EB-2, EB-3, and EB-5 indicate that these sediments classify as a “SAND” on the United States Department of Agriculture (USDA) soil textural triangle. Other samples of the alluvial sand and gravel recovered from these explorations during drilling appeared visually similar to the samples on which sieve analyses were conducted. The 2005 Ecology Manual recommends a long-term design infiltration rate no greater than 2 inches per hour (iph) into soil classified as sand. We anticipate that additional testing, area/facility-specific infiltration testing, and mounding analysis would support a higher design infiltration rate. It should also be noted that any infiltration system design would be dependent on depth to the seasonal high ground water table, as well as the depth of proposed improvements.

II. GEOLOGIC HAZARDS AND MITIGATIONS

The following discussion of potential geologic hazards is based on the geologic, slope, and ground water conditions, as observed and discussed herein. The discussion will be limited to slope stability, seismic, liquefaction, and erosion issues.

6.0 SEISMIC HAZARDS AND MITIGATIONS

Earthquakes occur regularly in the Puget Lowland. Most events are small and are usually not felt by people. However, large earthquakes do occur, as evidenced by the 1949, 7.2-magnitude event; the 2001, 6.8-magnitude event; and the 1965, 6.5-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 fault traces to the project site are the South Whidbey Island Fault Zone (SWIFZ) located approximately 2 miles to the northeast, and the Seattle Fault Zone located approximately 5 miles to the south.

A 2005 study by the U.S. Geological Survey (USGS) (Sherrod et al., 2005, *Holocene Fault Scarps and Shallow Magnetic Anomalies Along the Southern Whidbey Island Fault Zone near Woodinville, Washington*, Open-File Report 2005-1136, March 2005) reported that "strong" evidence of prehistoric earthquake activity has been observed along two fault strands thought to be part of the southeastward extension of the SWIFZ. The study suggests as many as nine earthquake events along the SWIFZ may have occurred within the last 16,400 years. The recognition of this fault splay is relatively new, and data pertaining to it are limited with the studies still ongoing. The recurrence interval of movement along this fault system is still unknown, although it is hypothesized to be in excess of one thousand years.

Studies of the Seattle Fault Zone by the USGS (e.g., Johnson et al., 1994, *Origin and Evolution of the Seattle Fault and Seattle Basin*, *Washington*, Geology, v. 22, pp. 71-74; and Johnson et al., 1999, *Active Tectonics of the Seattle Fault and Central Puget Sound Washington - Implications for Earthquake Hazards*, Geological Society of America Bulletin, July 1999, v. 111, n. 7, pp. 1042-

1053) have provided evidence of surficial ground rupture along a northern splay of the Seattle Fault. According to the USGS studies, the latest movement of this fault was about 1,100 years ago when about 20 feet of surficial displacement took place. This displacement can presently be seen in the form of raised, wave-cut beach terraces along Alki Point in West Seattle and Restoration Point at the south end of Bainbridge Island. The recurrence interval of movement along this fault system is still unknown, although it is hypothesized to be in excess of several thousand years.

Based on the distance between the site and the traces of these faults and the suspected long recurrence interval, the risk of surficial ground rupture along the Seattle Fault Zone and the SWIFZ is considered to be low during the expected life of the proposed structures and no mitigations are recommended.

6.2 Seismically Induced Landslides

It is our opinion that the risk of damage to the subject project by landsliding under either static or seismic conditions is low, provided the recommendations presented in this report are properly followed. Landslide hazards are discussed in further detail in the *Landslide Hazards and Mitigation* 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.

The subject site is situated in a designated Seismic Hazard Area according to the City of Redmond seismic hazard map. To assess the liquefaction risk, we performed a liquefaction hazard analysis for this site in accordance with guidelines published in Seed & Idriss, 1982; Seed et al., 1985; and Kramer, 1996. Our liquefaction analysis was completed with the aid of LiquefyPro computer software Version 5.8h (2009) by CivilTech Corporation. This program accepts input for SPT data, ground water levels, soil unit weight, and the depth and grain-size distribution of the sediments of concern to calculate seismically induced settlement. The liquefaction analysis was conducted based on the subsurface conditions encountered in the

explorations drilled at the site, particularly the grain-size analysis results and SPT blow count data from borings EB-2 and EB-6. The following assumptions were used during the analysis:

- Conservative estimated unit weights were based on conventional values for the soil types encountered;

- Estimates of silt content were based on laboratory testing of selected samples from borings EB-2 and EB-6, as well as comparison of sieve results to other, non-tested intervals;

- Given the historical seasonal ground water fluctuation recorded at the site, ground water levels 2 feet above the levels observed at the time of our field study were assumed for our liquefaction analysis.

- We used two different wet settlement analysis methods embedded in the software and obtained a range of liquefaction-induced settlement values;

- The design event was considered a magnitude 7.0 earthquake with a peak horizontal ground acceleration of 0.51g, in accordance with the 2012 *International Building Code* (IBC);

- Due to the elevated gravel content of some of the samples, some of the blow count data was suspected to be overstated. We conducted additional analyses in which blow counts were lowered to more typical values for both the portion of the alluvium and recessional outwash sediments encountered below the water table.

Based on the liquefaction analysis utilizing the soil and ground water data from borings EB-2 and EB-6, the subsurface conditions encountered at the site pose a moderate risk of liquefaction and associated liquefaction-induced settlement. The liquefaction-induced settlement calculated for the site ranges from less than 0.5 inches to approximately 2 inches.

In our opinion, the magnitude of the predicted liquefaction-induced settlement for the property is within tolerable limits given that our analysis is based on a very large, very rare seismic event. Per the 2012 IBC, a design event for the subject location consists of a magnitude 7.0 earthquake with a peak vertical ground acceleration of 0.51g. Actual seismic events and forces experienced at the site will likely be much lower and result in less settlement.

As discussed above in Section 6.0, earthquakes of magnitude 5.5 to 6.0 have a recurrence interval of about 20 years in the Puget Sound and would result in liquefaction-induced settlement less than the predicted 0.5 to 2 inches. Larger earthquake events have been recorded in the Puget Sound, such as the 1949, 7.2-magnitude event, the 1965, 6.5-magnitude event, and the 2001, 6.8-magnitude event. These larger events have occurred under or near

Olympia, and the intensity of the event at the subject site will be less than that at the epicenter due to natural attenuation of seismic energy through the ground. Thus, liquefaction-induced settlement resulting from these earthquakes will likely be less than the predicted 0.5 to 2 inches. However, if MSPG cannot accept the risk of liquefaction-induced settlement of this magnitude, a deeper foundation system may need to be implemented. Recommendations for both shallow and deep foundation systems are presented in the "Foundations" section of this report.

6.4 Ground Motion

Structural design of the buildings should follow 2012 IBC standards using Site Class "D" as defined in Table 20.3-1 of *American Society of Civil Engineers* (ASCE) *7 – Minimum Design Loads for Buildings and Other Structures*.

7.0 LANDSLIDE HAZARDS AND MITIGATIONS

The topography of the subject site is relatively flat, becoming moderately sloping toward its east end. The northern side of the property is bounded by the Redmond Central Connector. The Redmond Central Connector is a bicycle trail that was constructed atop a former railroad embankment. Review of a topographic site plan prepared by Contour Engineering, LLC (Figure 2) indicates that the portion of the railroad embankment located adjacent to the site ranges in height from approximately 6 to 14 feet with an inclination of approximately 40 to 55 percent. Chapter 21.64.060 of the *Redmond Municipal Code* (RMC) indicates that "any area with a slope 40 percent or steeper with a vertical relief of 10 feet or more" classifies as a Landslide Hazard Area. The code does not provide an exemption for slopes created from previous legal grading. Consequently, that portion of the railroad embankment equal to or greater than 10 feet in height classifies as a Landslide Hazard Area. The code states that a 50-foot buffer must be maintained around Landslide Hazard Areas, unless it can be demonstrated through a technical study that a reduction in the buffer will adequately protect the proposed and surrounding development from the critical landslide hazard. The minimum buffer allowed per the code is 15 feet. The property line setback from the toe of the railroad embankment ranges from zero up to approximately 15 feet. The toe of the embankment is supported by a rockery along a portion of its length. The rockery is located along a portion of the north property boundary in the central portion of the site (adjacent to the 15814 Bear Creek Parkway building) and has a maximum height of approximately 5 feet.

Given the relatively low height of the embankment, its inclination, and its history supporting railroad traffic for many years, it is our opinion that a minimum buffer of 15 feet from the toe of the embankment will provide suitable mitigation of landslide hazards for the subject project.

8.0 EROSION HAZARDS AND MITIGATIONS

The natural sediments underlying the site consist predominantly of clean sand and gravel. These sediments are not particularly sensitive to erosion. However, portions of the existing fill and alluvial sediments do contain significant amounts of silt and will be sensitive to erosion and disturbance when wet. To mitigate the potential for off-site sediment transport, we recommend the following:

1. Construction activity should be scheduled or phased as much as possible to reduce the amount of earthwork activity that is performed during the winter months.

2. The winter performance of a site is dependent on a well-conceived plan for control of site erosion and storm water runoff. The project temporary erosion and sediment control (TESC) should include ground-cover measures, access roads, and staging areas. The contractor must implement and maintain the required measures. A site maintenance plan should be in place in the event storm water turbidity measurements are greater than Ecology standards.

3. TESC measures for a given area, to be graded or otherwise worked, should be installed prior to any activity within that area. The recommended sequence of construction within a given area would be to install sediment traps and/or ponds and establish perimeter flow control prior to starting mass grading.

4. During the wetter months of the year, or when large storm events are predicted during the summer months, each work area should be stabilized so that if precipitation occurs, the work area can receive the rainfall without excessive erosion or sediment transport. The required measures for an area to be “buttoned-up” will depend on the time of year and the duration the area will be left un-worked. During the winter months, areas that are to be left un-worked for more than 2 days should be mulched or covered with plastic. During the summer months, stabilization will usually consist of seal-rolling the subgrade. Such measures will aid in the contractor’s ability to get back into a work area after a storm event. The stabilization process also includes establishing temporary storm water conveyance channels through work areas to route runoff to the approved treatment facilities.

5. All disturbed areas should be revegetated as soon as possible. If it is outside of the growing season, the disturbed areas should be covered with mulch, as recommended in the erosion control plan. Straw mulch provides the most cost-effective cover measure and can be made wind-resistant with the application of a tackifier after it is placed.

6. Surface runoff and discharge should be controlled during and following development. Uncontrolled discharge may promote erosion and sediment transport.

7. Soils that are to be reused around the site should be stored in such a manner as to reduce erosion from the stockpile. Protective measures may include, but are not limited to, covering with plastic sheeting, the use of low stockpiles in flat areas, or the use of straw bales/silt fences around pile perimeters. During the period between October 1st and March 31st, these measures are required.

8. On-site erosion control inspections and turbidity monitoring (when required) should be performed in accordance with Ecology requirements. Weekly and monthly reporting to Ecology should be performed on a regularly scheduled basis. Temporary and permanent erosion control and drainage measures should be adjusted and maintained, as necessary, for the duration of project construction.

It is our opinion that with the proper implementation of the TESC plans and by field-adjusting appropriate mitigation elements (best management practices [BMPs]) throughout construction, as recommended by the erosion control inspector, the potential adverse impacts from erosion hazards on the project may be mitigated.

III. DESIGN RECOMMENDATIONS

9.0 INTRODUCTION

Our exploration indicates that, from a geotechnical standpoint, the subject site is suitable for the proposed development provided the risks discussed are accepted and the recommendations contained herein are properly followed. The site is underlain by moderately liquefiable alluvial sediments to depths of approximately 20 to 25 feet, and likely deeper in the eastern portion of the site where the natural sediments are overlain by fill. It is our opinion that the actual risk from liquefaction-induced settlement is low and that conventional slab-on-grade or spread footings may be used. If liquefaction hazards are to be totally mitigated, augercast piles or stone columns should be used for foundation support.

In the western portion of the site, west of boring EB-5, the granular alluvial sediments are overlain by up to approximately 10 feet of medium stiff to stiff silt. Portions of the silt contain interbeds of organic silt and peat. The organic silt and peat sediments present a high risk of consolidation and settlement if subjected to building loads and are not recommended for foundation or floor slab support. Accordingly, removal of these sediments below building areas, or use of a deep foundation/floor slab support system is recommended in this area.

10.0 SITE PREPARATION

Site preparation should include removal of all trees, brush, debris, and any other deleterious materials. If the existing structures and/or pavement are not part of future development plans, the structures should be demolished and any remaining foundation elements or buried utilities that are not to remain operational should be removed and backfilled with structural fill as discussed in the report. All disturbed soils resulting from demolition activities should be removed to expose underlying undisturbed native sediments and replaced with structural fill, as needed. All excavations below final grade made for demolition activities should be backfilled, as needed, with structural fill. Erosion and surface water control should be established around the clearing limits to satisfy local requirements. If the two existing monitoring wells on the site will not be preserved during construction, they should be decommissioned by a licensed well driller in accordance with *Washington Administrative Code* (WAC) Section 173-160. If any heating oil storage tanks or other similar structures are present on-site, they should be decommissioned and removed in accordance with applicable Ecology regulations.

10.1 Proof-Rolling and Subgrade Compaction

Following demolition, site stripping, and potential excavation, the stripped subgrade within the building pad and any new paving areas should be proof-rolled with heavy, rubber-tired

construction equipment, such as a fully loaded tandem-axle dump truck. Proof-rolling should be performed prior to structural fill placement. The proof-roll should be monitored by the geotechnical engineer so that any soft or yielding subgrade soils can be identified. Any soft/loose, yielding soils should be removed to a stable subgrade. The subgrade should then be scarified, adjusted in moisture content, and recompacted to the required density. Proof-rolling should only be attempted if soil moisture contents are at or near optimum moisture content. Proof-rolling of wet subgrades could result in further degradation. Low areas and excavations may then be raised to the planned finished grade with compacted structural fill.

In the western portion of the site, west of boring EB-5, the granular alluvial sediments are overlain by up to approximately 10 feet of medium stiff to stiff silt with interbeds of peat and organic silt. In this area, we recommend that any organic silt or peat exposed in pavement subgrade areas be removed. Where the pavement subgrade consists of medium stiff to stiff silt, we recommend that the silt be overlain by an engineering stabilization fabric (Mirafi 500X, or approved equivalent). A minimum of 12 inches of structural fill consisting of clean, free-draining sand and gravel should then be placed over the stabilization fabric to create a firm subgrade for the overlying pavement section.

10.2 Site Disturbance

The granular alluvial sediments generally contain less than 10 percent fine-grained material, which makes them slightly moisture-sensitive and subject to disturbance when wet. However, in the western portion of the site (west of boring EB-5), the granular alluvial sediments are overlain by silt which is highly moisture sensitive. 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. If crushed rock is considered for the access and staging areas, it should be underlain by stabilization fabric (such as Mirafi 500X or approved equivalent) to reduce the potential of fine-grained materials pumping up through the rock and turning the area to mud. The fabric will also aid in supporting construction equipment, thus reducing the amount of crushed rock required. We recommend that at least 10 inches of rock be placed over the fabric; however, due to the variable nature of the near-surface soils and differences in wheel loads, this thickness may have to be adjusted by the contractor in the field.

10.3 Temporary and Permanent 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, we anticipate that temporary, unsupported cut slopes in the existing alluvium can be made at a maximum slope of 1.5H:1V (Horizontal:Vertical) or flatter. As is typical with earthwork operations, some sloughing and raveling may occur, and cut slopes may have to be adjusted in the field. If ground water seepage is encountered in cut slopes, or if surface water is not routed away from

temporary cut slope faces, flatter slopes will be required. In addition, WISHA/OSHA regulations should be followed at all times. Permanent cut and structural fill slopes that are not intended to be exposed to surface water should be designed at inclinations of 2H:1V or flatter. All permanent cut or fill slopes should be compacted to at least 95 percent of the modified Proctor maximum dry density, as determined by ASTM:D 1557, and the slopes should be protected from erosion by sheet plastic until a vegetative cover can be established during favorable weather.

11.0 STRUCTURAL FILL

All references to structural fill in this report refer to subgrade preparation, fill type and 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.

After stripping, planned excavation, and any required overexcavation have been performed to the satisfaction of the geotechnical engineer, the upper 12 inches of exposed ground in areas to receive fill should be recompacted to 95 percent of the modified Proctor maximum density using ASTM:D 1557 as the standard. If the subgrade contains silty soils and 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 tested and 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, placed in maximum 8-inch loose lifts, with each lift being compacted to 95 percent of the modified Proctor maximum density using ASTM:D 1557 as the standard. In the case of roadway and utility trench filling, structural fill should be placed and compacted in accordance with current City of Redmond codes and standards. The top of the compacted fill should extend horizontally outward a minimum distance of 3 feet beyond the locations of the roadway edges before sloping down at an angle of 2H:1V.

The contractor should note that any proposed fill soils must be evaluated by AESI prior to their use in fills. This would require that we have a sample of the material 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.

Use of moisture-sensitive soil in structural fills should be limited to favorable dry weather conditions, and is only permitted if specifically allowed by project plans and specifications. The native soils present on-site contained variable amounts of silt. Samples of the granular alluvium and recessional outwash sediments tested for our study contained less than 10 percent fine-grained material, though isolated beds of material within the geologic unit may be more fine-grained. We therefore classify on-site soils as slightly moisture-sensitive. The silt and peat sediments encountered in the western portion of the site are not suitable for reuse in structural fill applications. 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 with at least 25 percent retained on the No. 4 sieve. This type of material should be used for structural fill building pad material if a mat foundation is used for building support.

A representative from our firm should inspect the stripped subgrade and be present during placement of structural fill to observe the work and perform a representative number of in-place density tests. In this way, the adequacy of the earthwork may be evaluated as filling progresses, and any problem areas may be corrected at that time. It is important to understand that taking random compaction tests on a part-time basis will not assure uniformity or acceptable performance of a fill. As such, we are available to aid the owner in developing a suitable monitoring and testing program.

12.0 FOUNDATIONS

In our opinion, the magnitude of the predicted liquefaction-induced settlement for the property is within tolerable limits given that our analysis is based on a very large, very rare seismic event. Actual seismic events experienced at the subject site will likely be much lower and result in less settlement. Although the liquefaction-induced settlement is anticipated to be minor, the effects associated with the magnitude of predicted settlement and differential settlement may result in cosmetic damage to the structure that would warrant repairs following a large earthquake. It is our opinion that conventional slab or spread footings could be used for foundation support of the proposed project provided they are founded upon suitable foundation bearing strata as described below.

As an alternative, if some differential settlements can be tolerated, a mat foundation could be used to partially mitigate the effects of liquefaction, as outlined below.

If MSPG cannot accept the risk for potential liquefaction-induced settlement on the order of 0.5 to 2 inches, a deep system is recommended. For full mitigation of liquefaction-induced settlement a deep foundation system is recommended to transfer building loads to the dense, liquefaction resistant, recessional outwash sediments encountered at depth. Based on our

explorations, dense, recessional outwash sediments are anticipated to underlie most areas of the site at depths ranging from approximately 20 to 25 feet. One exception is the eastern portion of the site (area of boring EB-1) where the natural sediments are overlain by fill and the depth to the dense outwash sediments is anticipated to be approximately 10 feet deeper.

In the western portion of the site, west of boring EB-5, the granular alluvial sediments are overlain by up to approximately 10 feet of medium stiff to stiff silt. Portions of the silt contain interbeds of organic silt and peat. The organic silt and peat sediments present a high risk of consolidation and settlement if subjected to building loads. For this reason, the silt sediments encountered west of boring EB-5 are not recommended for foundation or floor slab support. Consequently, the use of spread footing foundations and slab-on-grade floors west of boring EB-5 is contingent upon removal of all of the silt and peat sediments below building areas.

Recommendations for spread footing foundations and deep foundation systems consisting of either stone columns or augercast piles are provided below.

12.1 Conventional Spread Footings

Spread footings may be used for building support when founded either directly on the alluvial sand and gravel sediments, or on structural fill placed over these materials. Alluvial sand and gravel sediments suitable for foundation support were encountered in exploration borings EB-2, EB-3, and EB-5 at depths of approximately 1 to 2.5 feet. Due to the presence of overlying fill or unsuitable silt and peat sediments the depth to the alluvial sand and gravel in the remaining borings ranged from approximately 6 to 10.5 feet.

For footings founded either directly upon the natural alluvial sand and gravel, or on structural fill placed over these sediments, we recommend that an allowable bearing pressure of 3,500 pounds per square foot (psf) be used for design purposes, including both dead and live loads. We recommend that the footing subgrade be recompacted to a firm and unyielding condition prior to footing placement. An increase in the allowable bearing pressure of one-third may be used for short-term wind or seismic loading. If structural fill is placed below footing areas, the structural fill should extend horizontally beyond the footing edges a distance equal to or greater than the thickness of the fill.

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.

12.2 Stone Columns

Should MSPG wish to more completely mitigate liquefaction-induced settlement, or eliminate that need for overexcavation where the foundation bearing stratum is overlain by existing fill or silt/peat sediments, our recommended approach to foundation design is to install stone columns. Stone columns consist of columns of compacted crushed rock below the building pad. Installation of stone columns results in significant densification of the surrounding soils, as well as a network of compacted stone columns that transmit loads directly to more competent soils at depth. There is little consistency between different contractors who install stone columns with respect to their installation equipment and methods. The diameters, depth capability, compactive energy, and other critical factors of each contractor's equipment must be considered when designing a stone column foundation system, and therefore such systems are typically designed by the contractor who installs them. Once stone columns are installed, the building is constructed with a conventional shallow foundation system above a subgrade that has been improved through installation of stone columns.

The stone columns should be installed after the site is graded and the building pad fill is placed and compacted. The purpose of stone columns is to both improve existing loose soils and to transmit loads to more competent bearing materials at depth. Stone columns are formed by advancing a hollow mandrel to a pre-determined depth. Crushed rock is then compacted through the hollow mandrel in thin lifts. The result is a column of compacted aggregate and compaction of soils surrounding the stone columns. Stone columns are proprietary systems and are designed by the contractor who installs them. The contractor will determine the depth and diameter of the stone column holes and the appropriate spacing. Conventional shallow foundations are then constructed above the subgrade after stone columns have been installed. The stone columns contractor should review exploration logs contained in this report carefully. Our explorations encountered existing fill and alluvium. Although not encountered in our explorations, the existing fill may contain organic materials and scattered demolition waste. The existing fill and the existing native sediments may also contain coarse gravel and cobbles as noted on the exploration logs. Where drilling obstacles are encountered, the contractor should be prepared to relocate stone columns, or remove obstacles, as needed. The contractor should expect ground water as noted on our exploration logs in Appendix A. The contractor should not assume that the site is suitable for use of uncased open holes. In our opinion, using a system of stone columns, it would be possible to achieve an allowable foundation soil bearing pressure of 5,000 psf.

12.3 Augercast Pile Foundations

If the owner wishes to fully mitigate liquefaction hazards and associated foundation settlement, we recommend supporting the new structure on augercast piles. Construction of piles should be accomplished by a contractor experienced in their installation. Soils of alluvial origin may have gravel lenses, coarse gravels, or large cobbles present in them. It may be necessary to

have a backhoe present during pile installation to dig out obstacles and backfill the excavation prior to drilling piling. If obstacles are encountered at depths where removal with a backhoe is not feasible, it might be necessary to modify the pile layout to replace piles that cannot be completed according to the original design. Observation of pile installation by AESI is recommended to verify that the subsurface conditions observed at pile locations are consistent with the observations in our subsurface explorations, and consistent with assumptions made during preparation of the recommendations in this report. The City of Redmond will likely require geotechnical observation of augercast pile installation.

Augercast piles are formed by drilling to the required depth with a continuous flight, hollow-stem auger. Fluid grout is then pumped down the hollow-stem, under pressure, as the auger is withdrawn. Appropriately designed, reinforced steel cages are then lowered into the unset grout. A single reinforcing bar is often installed for the full length of the pile for transfer of uplift loads. Since the ground is placed under pressure, actual grout volumes used are typically 15 to 50 percent greater than the theoretical volume of the pile. The pile contractor must be required to provide a pressure gauge and a calibrated pump stroke counter so that the actual grout volume for each pile can be measured.

The proposed augercast piles would gain support primarily from end bearing, with some contribution from skin friction. Recommended allowable vertical and uplift capacities for 18-inch-diameter augercast piles are provided below in Table 2.

Table 2
Augercast Pile Recommendations

Pile Diameter (inches)	Estimated Total Pile Length (feet)	Recommended Minimum Embedment into Bearing (feet)	Vertical Capacity (kips)	Uplift Capacity (kips)
18	35 - 45	10	60	30

Note: The bearing stratum consists of the dense, recessional outwash sediments.

Allowable design loads may be increased by one-third for short-term wind or seismic loading. Anticipated settlement of pile-supported structures will generally be on the order of ½ inch or less.

A lateral capacity of 20 kips per pile for fixed-headed conditions (incorporated into the pile caps and grade beam system), and ½-inch of deflection at the ground surface can be assumed for 18-inch-diameter piles. Greater lateral capacities are possible for greater allowable deflections. The depth of fixity can be assumed at 20 feet.

Augercast piles should penetrate a minimum of 10 feet into the bearing soils to develop the capacities shown above, in Table 2. Based on the explorations completed for this project, the

depth to the bearing stratum, consisting of the dense recessional outwash sediments, is anticipated to be encountered in most areas of the site within a depth of approximately 25 feet. Due to the presence of existing fill, the depth to the bearing stratum is anticipated to be approximately 10 feet deeper in the far eastern portion of the site (area of boring EB-1). For this reason, the actual total length of each pile will vary based on the conditions encountered during drilling. Since completion of the piles takes place below ground, the judgment and experience of the geotechnical engineer, or their field representative must be used as a basis for determining the required penetration and acceptability of each pile. Consequently, the use of the presented pile capacities in the design requires that installation of all piles be observed by the geotechnical engineer or their representative who can interpret and collect the installation data and observe the contractor's operations. AESI, acting as the owner's field representative, would observe the lengths of the piles and keep records of pertinent installation data. A final summary report would then be distributed following completion of pile installation.

Piles with lateral spacing less than 6 pile diameters from another pile along the direction of force should be considered to be in the zone of influence, and the lateral capacity and the reduction factors presented below should be used.

Table 3
Lateral Reduction Factors

Pile Spacing in Direction of Loading	Reduction Factor
6 diameters	1.0
5 diameters	0.8
4 diameters	0.6
3 diameters	0.4

Based on the loose to medium dense conditions of the soils through which the augercast piles are to be drilled, care should be taken in construction planning to allow grout time to set prior to drilling adjacent piles. Typically, 24 hours or 1 day of set time is recommended for piles closer than 3 diameters or 10 feet, whichever is greater.

12.4 Drainage Considerations

Foundations should be provided with foundation drains. Drains should consist of rigid, perforated, polyvinyl chloride (PVC) pipe surrounded by washed pea gravel. The drains should be constructed with sufficient gradient to allow gravity discharge away from the proposed buildings. 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

walls should be sloped downward away from the proposed structures to achieve surface drainage.

12.5 Lateral Earth Pressures and Coefficients of Friction

Lateral loads can be resisted by friction between the mat foundation or grade beams and the alluvium or supporting structural fill soils, and by passive earth pressure acting on the buried portions of the mat foundations/grade beams. We recommend the following allowable design parameters:

- Passive equivalent fluid = 200 pounds per cubic foot (pcf)
- Coefficient of friction = 0.30

If an augercast pile foundation is used, base friction on the grade beams should be ignored in the structural design.

13.0 FLOOR SUPPORT

Slab-on-grade floors may be constructed on medium dense native soils or structural fill placed over the alluvial sand and gravel sediments. We recommend that the alluvial sediments be recompacted to a firm and unyielding condition prior to placement of the structural fill. All fill placed beneath the slab must be compacted to at least 95 percent of ASTM:D 1557. This layer of structural fill will reduce but not eliminate the risk of liquefaction-induced and post-construction settlement. Elimination of this risk can be accomplished by supporting the slab fully on augercast piles or a subgrade improved using stone columns.

Floor slabs can be supported directly on a subgrade improved by stone columns. If augercast piles are used, the floor slab should be supported on the augercast piles, or isolated from the foundation elements and supported on a minimum thickness of 2 feet of granular structural fill compacted to 95 percent of ASTM:D 1557 to provide uniform support of the slab. On-grade support of floor slabs in areas underlain by alluvial silt and peat (i.e., the portion of the site west of boring EB-5) is not recommended.

Regardless of the type of floor slab support used, floor slabs should be cast atop a minimum of 4 inches of clean, washed, crushed rock or pea gravel to act as a capillary break. Floor slabs should also be protected from dampness by a plastic moisture vapor retarder at least 10 mils thick. The moisture vapor retarder should be placed between the capillary break material and the concrete slab.

14.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

We recommend that AESI perform a geotechnical review of the plans prior to final design completion. In this way, we can confirm that our earthwork and foundation recommendations have been properly interpreted and implemented in the design.

We are also available to provide geotechnical engineering services during construction. The integrity of the foundation system 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 this current scope of work. If these services are desired, please let us know, and we will prepare a cost proposal.

15.0 CLOSING

We have enjoyed working with you on this study and are confident that 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.

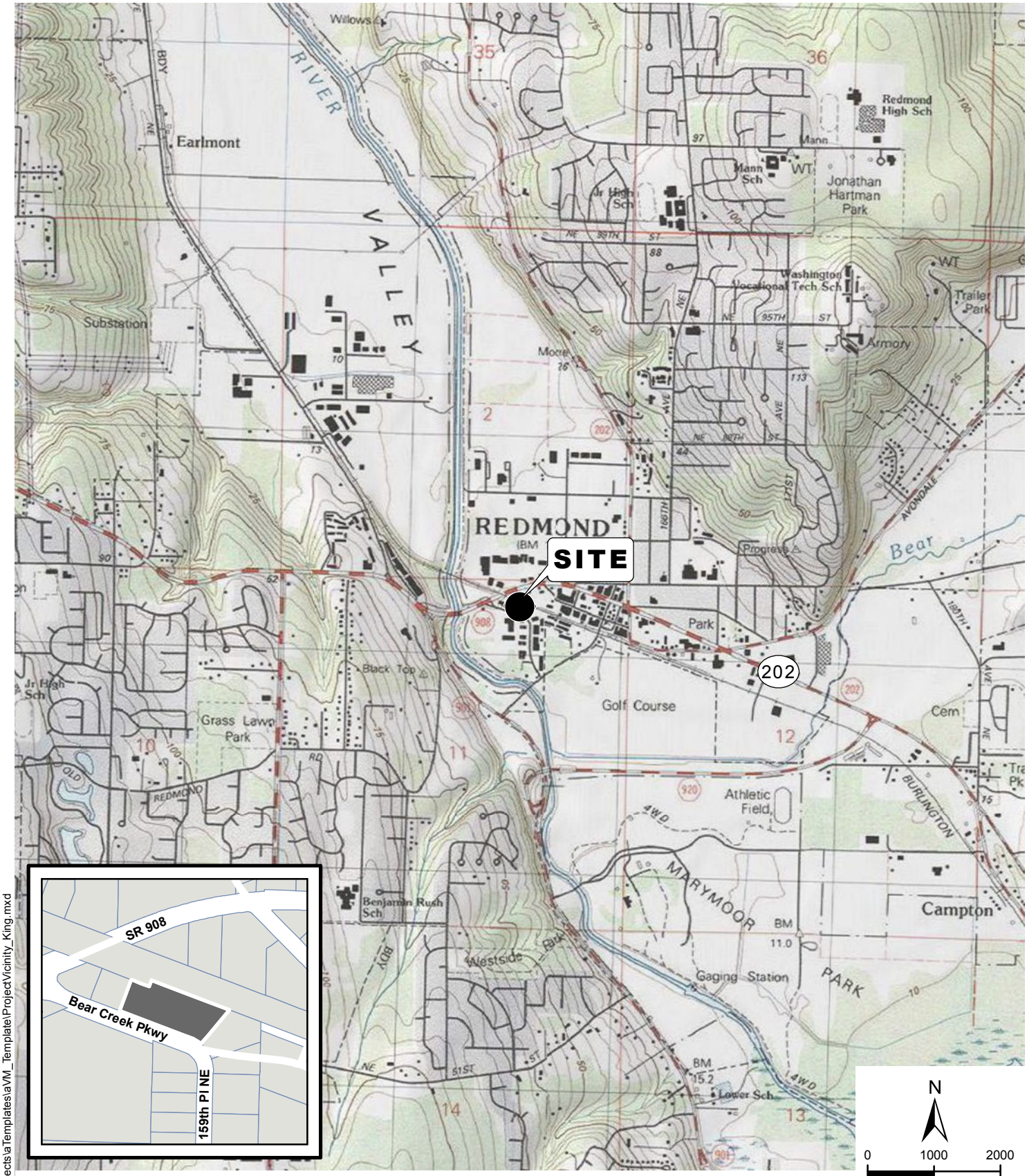
Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

KURT D. MERRIMAN
STATE OF WASHINGTON
REGISTERED
PROFESSIONAL ENGINEER

Timothy J. Peter, L.G., L.E.G., L.Hg.
Senior Project Geologist

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

Attachments: Figure 1: Vicinity Map
Figure 2: Site and Exploration Plan
Appendix A: Exploration Logs
Appendix B: Laboratory Testing Results



REFERENCE: USGS, KING CO

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

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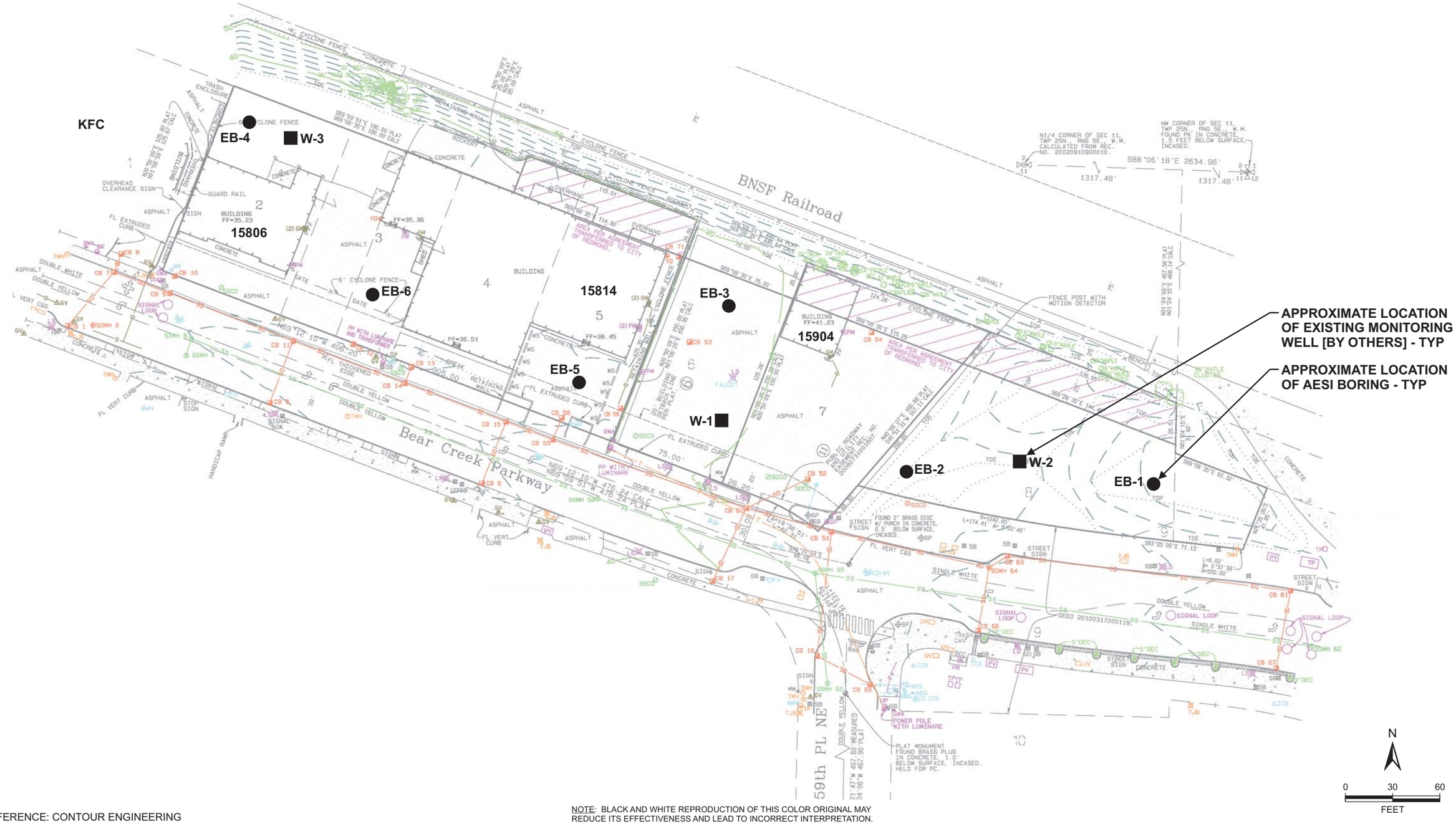
VICINITY MAP

BEAR CREEK PARKWAY DEVELOPMENT
REDMOND, WASHINGTON

FIGURE 1

DATE 6/15

PROJ. NO. KE140705A



REFERENCE: CONTOUR ENGINEERING

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

140705 Bear Creek Project \ 140705 F2 Site and Exploration Plan.cdr

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SITE AND EXPLORATION PLAN

BEAR CREEK PARKWAY PROPERTY

REDMOND, WASHINGTON

FIGURE 2

DATE 5/15

PROJ. NO. KE140705A

APPENDIX A

Exploration Logs

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Exploration Log

Project Number	Exploration Number	Sheet
KE140705A	EB-1	1 of 1

Project Name	Bear Creek Parkway Property	Ground Surface Elevation (ft)	48
Location	Redmond, WA	Datum	NAVD88
Driller/Equipment	EDI / B61 HD	Date Start/Finish	5/20/15,5/20/15
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	8 inches

Depth (ft)	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
				Fill					
		S-1		Overstated, rock in shoe. Slightly moist, light brown, fine to coarse SAND, some silt, some fine to coarse gravel; nonstratified (SW/SM).			7 20 24	▲44	
5		S-2		Moist, brown, gravelly, very silty fine to coarse SAND, some fine to coarse gravel, trace organics; nonstratified (SM).			22 14 7	▲21	
		S-3		Trace recovery; looks similar to S-2.			11 21 24	▲45	
10		S-4		Overstated.			50/6"	▲50/6"	
				Holocene Alluvium Moist, brownish orange, fine to coarse SAND, trace silt, some fine to coarse rounded gravel (SW/SM).					
15		S-5		Moist, yellowish to brownish orange, fine to medium SAND, trace silt, trace gravel; iron-oxide; stratified (SP).			7 7 10	▲17	
20		S-6		Overstated. Moist, tan brown, gravelly fine to coarse SAND, trace silt; iron oxide staining; stratified (SW).			9 14 17	▲31	
				Bottom of exploration boring at 21.5 feet No ground water encountered.					

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: FSM
Approved by: CJK

AESIBOR 140705.GPJ June 2, 2015

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Exploration Log

Project Number	Exploration Number	Sheet
KE140705A	EB-2	1 of 2

Project Name	Bear Creek Parkway Property	Ground Surface Elevation (ft)	41
Location	Redmond, WA	Datum	NAVD88
Driller/Equipment	EDI / B61 HD	Date Start/Finish	5/20/15,5/20/15
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	8 inches

Depth (ft)	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
				Fill Cuttings are moist, brown, fine to coarse gravel (crushed rock) and quarry spalls from 0 to 2.5 feet.					
				Holocene Alluvium					
		S-1		Moist, brownish orange, fine to coarse SAND, trace ranging to some silt, some fine to coarse rounded gravel; crudely bedded; iron-oxide staining (SW).			8 7 8	▲15	
5		S-2		Moist, light brownish gray, fine to coarse SAND, trace silt, trace ranging to some fine to coarse rounded gravel; stratified (SW).			6 7 10	▲17	
		S-3		Trace recovery; similar to S-2, slight iron-oxide, light orange brown color (SW).			7 8 11	▲19	
10		S-4		Moist, light brownish orange, fine to coarse SAND, some silt, some fine to coarse rounded gravel; stratified; iron-oxide staining (SW/SM). Drill action suggests coarse gravelly layers from 10 to 15 feet.			5 6 7	▲13	
15		S-5		Wet, light olive, fine to coarse rounded GRAVEL, some coarse sand, trace silt (GW). Ground water at 16.8 feet with bottom of boring at 25 feet.		▼	6 14 12	▲26	
20				**Vashon Recessional Outwash**					
		S-6		Wet, light olive brown, fine to coarse rounded GRAVEL, some fine to coarse sand, trace silt (GW). Smooth drill action suggesting no significant coarse rock from 21 1/2 to 25 feet.			8 22 22	▲44	

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: FSM
Approved by: CJK

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Exploration Log

Project Number	Exploration Number	Sheet
KE140705A	EB-2	2 of 2

Project Name	Bear Creek Parkway Property	Ground Surface Elevation (ft)	41
Location	Redmond, WA	Datum	NAVD88
Driller/Equipment	EDI / B61 HD	Date Start/Finish	5/20/15,5/20/15
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	8 inches

Depth (ft)	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
		S-7		Approximate 1 foot heave, driller added water to flush sampler to prescribed sample depth. Overstated? Wet, olive, fine to medium SAND, trace silt; nonstratified; looks disturbed (possible heave) (SP).			9 14 32	▲46	
30		S-8		Approximate 1 foot heave, flush with water as above. Wet, light brownish olive, fine SAND, trace ranging to some silt; massive (SP).			7 12 22	▲34	
35		S-9		Approximate 2 foot heave, flush as above. Wet, light brownish olive, fine SAND, trace silt, and fine to medium SAND, trace silt, trace to some fine gravel and coarse sand; interbedded (SP).			10 12 24	▲36	
40		S-10		Approximate 1.5 foot heave, flush as above. Wet, brownish gray, medium to coarse SAND, some fine to coarse subrounded to rounded gravel, trace silt; crudely bedded (SP/SW).			17 30 36	▲66	
				Bottom of exploration boring at 41.5 feet					
45									

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: FSM
Approved by: CJK

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Exploration Log

Project Number	Exploration Number	Sheet
KE140705A	EB-3	1 of 1

Project Name	Bear Creek Parkway Property	Ground Surface Elevation (ft)	40
Location	Redmond, WA	Datum	NAVD88
Driller/Equipment	EDI / HSA / SPT / B61	Date Start/Finish	5/20/15,5/20/15
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	8 inches

Depth (ft)	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
				Asphalt - 2 inches					
				Holocene Alluvium					
		S-1		Moist, brownish gray, very gravelly, well-graded SAND, trace gravel (SW).			3 4 5	▲9	
5		S-2		Moist, grayish brown, very sandy GRAVEL, trace silt (GW).			7 7 6	▲13	
		S-3		Moist, grayish brown, very gravelly SAND, trace to some silt (SW/SM).			5 5 5	▲10	
10		S-4					3 6 10	▲16	
15		S-5		Moist, grayish brown, very sandy GRAVEL, trace silt (GW).		▼	5 9 12	▲21	
20		S-6		Wet, brown, silty fine SAND (SM).			5 7 12	▲19	
				Bottom of exploration boring at 21.5 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: TJP

Approved by: CJK

AESIBOR 140705.GPJ June 2, 2015

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Exploration Log

Project Number	Exploration Number	Sheet
KE140705A	EB-4	1 of 1

Project Name	Bear Creek Parkway Property	Ground Surface Elevation (ft)	38
Location	Redmond, WA	Datum	NAVD88
Driller/Equipment	EDI / HSA / SPT / B61	Date Start/Finish	5/20/15,5/20/15
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	8 inches

Depth (ft)	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
				Crushed Rock Surfacing					
				Holocene Alluvium					
		S-1		Very moist, grayish tan, fine sandy SILT (SM).			3 4 6	▲10	
5				Very moist, dark brown, organic SILT (OL).					
		S-2		Moist, grayish tan, fine SAND, some silt, some gravel (SP).			2 3 4	▲7	
				Very moist, dark brown, PEAT (PT).					
		S-3		Moist, gray, very gravelly, well-graded SAND, trace silt (SW).			5 6 10	▲16	
10		S-4		Becomes wet, grayish tan. (Blow count likely overstated due to gravel content)		▼	5 9 17	▲26	
15				Some gravel.					
		S-5		Wet, tan-gray, sandy GRAVEL, trace silt (GW).			5 9 9	▲18	
				Vashon Recessional Outwash					
20		S-6		Wet, tan-gray, well-graded SAND, trace to some gravel (SW).			5 20 28	▲48	
				Sand becomes fine-grained, with some silt, some gravel (SP/SM).					
				Bottom of exploration boring at 21.5 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: TJP
Approved by: CJK`

AESIBOR 140705.GPJ June 2, 2015

associated earth sciences incorporated

Exploration Log

Project Number	Exploration Number	Sheet
KE140705A	EB-5	1 of 1

Project Name	Bear Creek Parkway Property	Ground Surface Elevation (ft)	39
Location	Redmond, WA	Datum	NAVD88
Driller/Equipment	EDI / HSA / SPT / B61	Date Start/Finish	5/21/15,5/21/15
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	8 inches

Depth (ft)	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
				Asphalt Pavement - 2 inches					
				Fill					
				Moist, dark brown, gravelly SAND, with abundant organics (SW).					
				Holocene Alluvium					
		S-1		Moist, brown, gravelly SAND, trace silt; poor recovery (SW). (Blow count likely overstated)			7 10 13	▲23	
5		S-2		Becomes very gravelly. (Blow count likely overstated)			6 10 10	▲20	
		S-3		Becomes grayish brown. (Blow count likely overstated)			7 13 11	▲24	
10		S-4		Decreased gravel content (gravelly); gravel finer grained, some slit (SW/SM).			4 4 6	▲10	
				Becomes wet.		▼			
15		S-5		Wet, gray, sandy GRAVEL, trace silt (GW). (Blow count likely overstated due to gravel content)			7 17 18	▲35	
20		S-6		(Blow count likely overstated due to gravel content)			12 8 18	▲26	
				Bottom of exploration boring at 21.5 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: TJP
Approved by: CJK

associated earth sciences incorporated

Exploration Log

Project Number	Exploration Number	Sheet
KE140705A	EB-6	1 of 2

Project Name	Bear Creek Parkway Property	Ground Surface Elevation (ft)	38
Location	Redmond, WA	Datum	NAVD88
Driller/Equipment	EDI / HSA / SPT / B61	Date Start/Finish	5/21/15,5/21/15
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	8 inches

Depth (ft)	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
				Asphalt Pavement - 1 inch					
				Holocene Alluvium					
				Organic silt in cuttings.					
		S-1		Very moist, brownish gray, SILT (ML).			2 3 4	▲7	
5				Becomes clayey, trace organics.					
		S-2					0 3 12	▲15	
				Moist, grayish brown, very gravelly SAND, trace silt (SW).					
		S-3		(Blow count likely overstated due to gravel content)			6 11 15	▲26	
10									
		S-4		Wet, gray, sandy GRAVEL, trace silt (GW). (Blow count likely overstated)		▼	12 15 22	▲37	
15									
		S-5					6 8 12	▲20	
				Vashon Recessional Outwash					
20									
		S-6		Wet, gray, very sandy GRAVEL, trace silt (GW).			8 15 30	▲45	
				(Approximately 2 inches of heave in auger after drilling to 25 feet; driller flushed out prior to sampling)					

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: TJP
Approved by: CJK

associated earth sciences incorporated

Exploration Log

Project Number	Exploration Number	Sheet
KE140705A	EB-6	2 of 2

Project Name	Bear Creek Parkway Property	Ground Surface Elevation (ft)	38
Location	Redmond, WA	Datum	NAVD88
Driller/Equipment	EDI / HSA / SPT / B61	Date Start/Finish	5/21/15,5/21/15
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	8 inches

Depth (ft)	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
		S-7		Wet, grayish brown, very gravelly SAND, trace silt (SW).			7 17 38	▲55	
				(Approximately 12 inches of heave in auger at 30 feet; driller flushed out prior to sampling)					
30		S-8		Wet, brownish gray, very sandy GRAVEL, trace silt (GW).			8 14 39	▲53	
				(Approximately 4 inches of heave in auger at 35 feet; driller flushed out heave prior to sampling)					
35		S-9		Wet, grayish brown, very gravelly SAND, trace silt (SW).			10 23 50/4"	▲73/10"	
				(Approximately 2 1/2 feet of heave in auger at 40 feet; driller flushed out prior to sampling)					
40		S-10		Trace gravel.			3 12 50	▲62	
				Bottom of exploration boring at 41.5 feet					
45									

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: TJP
Approved by: CJK

AESIBOR 140705.GPJ June 2, 2015

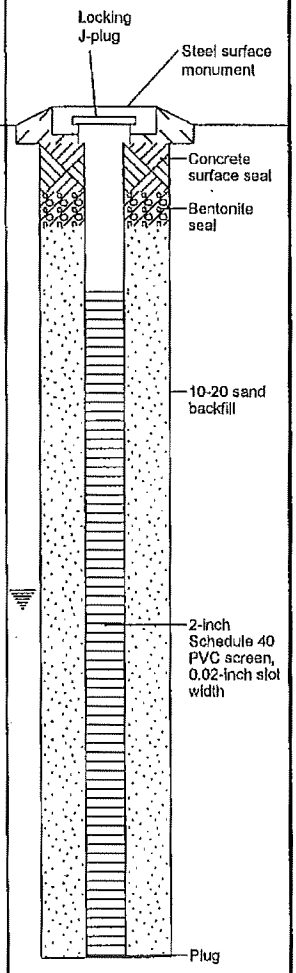
W-2

Date(s) Drilled	06/21/07	Logged By	AJ	Checked By	TNO
Drilling Contractor	Cascade Drilling	Drilling Method	Hollow-stem Auger	Sampling Methods	Split Barrel
Auger Data	2.4-inch I.D.	Hammer Data	140 lb hammer/30 in drop automatic	Drilling Equipment	CME 55 Track mounted
Total Exploration Depth (ft)	26.5	Ground Surface Elevation (ft)		Groundwater Level (ft. bgs)	14.5
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES Interval Recovered (in)	Blows/foot	Sub-Sample Sample Number	Testing	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Sheen	Headspace Vapor PID (ppm)
0						AC	2 inches asphalt concrete		
						SP	Brown fine to coarse sand with gravel and cobbles (medium dense, moist) (fill)		
	8	49	1					NS	<1
							Brown fine to coarse sand with gravel (medium dense, moist)		
5	8	50/6"	2				Becomes dense at 5 feet	NS	<1
	12	39	3					NS	<1
10	12	50/6"	4				With decreasing gravel at 10 feet	NS	<1
	3	50/3"	5	CA				NS	<1
15	8	50/6"	6	CA		GP	Brown fine gravel with sand (dense, wet)	NS	<1
	10	40/6"	7			SP	Brown fine to coarse sand with gravel (dense, wet)	NS	<1
20	10	50/5"	8					NS	<1
	10	50/4"	9					NS	<1
25	10	50/6"	10				Medium sand at 25 feet	NS	<1
30									
35									

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

WELL CONSTRUCTION



LOG OF MONITORING WELL MW-1

GEOENGINEERS

Project: Arend Property - Budget Rental
Project Location: Redmond, Washington
Project Number: 0500-133-02

Figure E-2
Sheet 1 of 1

V6_ENVWELL P:\0\0500133\02\FINALS\050013302.GPJ GEIV6_1.GDT 8/10/07

Western Wall (W-3)
(Near EB-4)

340413

RESOURCE PROTECTION/GEOTECH SOIL BORING WELL REPORT

Washington State Department of Ecology

Original and 1st copy - Ecology; 2nd Copy -- owner; 3rd copy - driller

PROPOSED USE:
☒ Construction
☐ Decommission *ORIGINAL INSTALLATION Notice of Intent Number* _____
Consulting Firm Terra Associates

Current Notice of Intent No. REQ 3052
Type of Well ☒ Resource Protection ☐ Geotech Soil Boring
Unique Ecology Well ID Tag No. BAA 940

DRILLING METHOD
☒ Hollow Stem Auger ☐ Air Rotary
☐ Mud Rotary ☐ Dual Rotary
☐ Core ☐ Other _____
Borehole Diameter 8

WELL LOCATION
Project Name _____
Owner NDC Building LLC
Well Address 7549 159th Place NE
City Redmond County King
Location NE 1/4 NE 1/4 Sec 11 Twn 25 R 5 (E) or W

MONUMENT
☐ Above Ground Riser ☐ 6" x 5' ☐ 8" x 5' Stick up height ____ ft
☒ Flush Mount ☒ 8" ☐ 12" ☐ Other _____
Amount of Concrete used _____

Tax Parcel No. _____
Construction/Decommission Start Date 2/11/09
Construction/Decommission Completed Date 2/11/09
Static Level _____

CASING INSTALLED
☐ PVC ☒ Sch 40 ☐ Sch 80 ☐ Inclinometer ☐ Other _____
☒ Threaded 2" Diameter from 0 ft to 10 ft
☐ Glued ____" Diameter from ____ ft to ____ ft
☐ Welded ____" Diameter from ____ ft to ____ ft

SCREEN
☐ PVC ☒ Sch 40 ☐ Sch 80 ☐ Other _____
Diameter 2 Slot Size .010 from 10 ft to 20 ft
☐ Pre Pack Type ☐ PVC ☐ Sch 40 ☐ Sch 80 ☐ Other _____
Diameter of inner screen ____" x Diameter of outer screen ____"
Slot Size ____ Installed from ____ ft to ____ ft
☐ Stainless Steel ____" Diameter from ____ ft to ____ ft
☐ Other _____ ____" Diameter from ____ ft to ____ ft

SEAL
Type of material used ☒ Bentonite Chips Amount 4
☐ Bentonite Grout Amount _____
☐ Portland Cement Amount _____
☐ Other _____ Amount _____
Placed from 2 ft to 9 ft

SAND/GRAVEL PACK
Type of material used ☒ Silica Sand Size 10/20 ☐ Pea Gravel
☐ Other _____
Placed from 9 ft to 20 ft
Amount of material used 7 SACKS

CONSTRUCTION OR DECOMMISSION PROCEDURE

Material or Formation	From	To
Gravel	0	1
Fill	1	2
Med Brn Sand	2	5
Peat	5	9
Sand + Gravel	9	20

RECEIVED
MAY 08 2009
Dept of Ecology
WR-NWRO

WELL CONSTRUCTION CERTIFICATION: I constructed and/or accept responsibility for construction of this well, and its compliance with all Washington well construction standards. Materials used and the information reported above are true to my best knowledge and belief.

☒ Driller ☐ Trainee Name (print) Cory M James
Driller / Trainee Signature [signature]
Driller or Trainee License No. 2828

Drilling Company Gregory Drilling, Inc.
If Trainee, licensed driller's Signature and License No. _____

The Department of Ecology does NOT Warranty the Data and/or the Information on this Well Report.

APPENDIX B

Laboratory Testing Results

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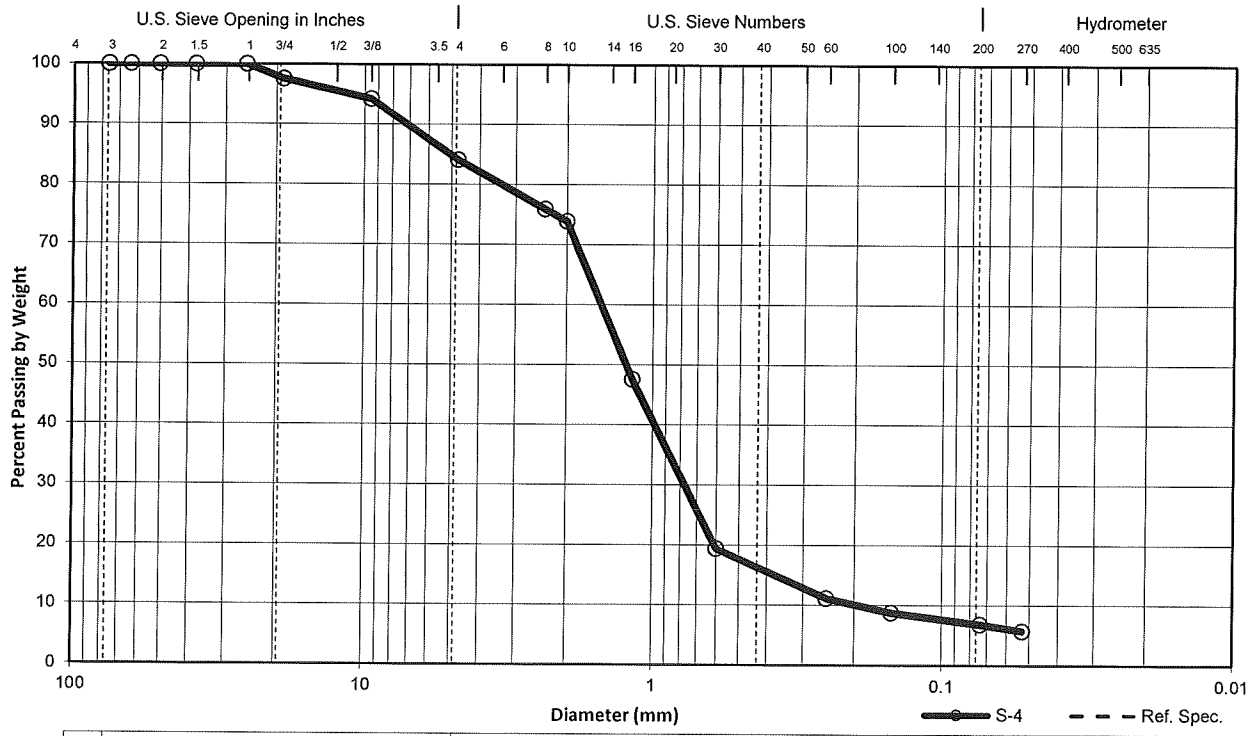
GRAIN SIZE ANALYSIS - MECHANICAL ASTM D422

Project Name	Project Number	Date Sampled	Date Tested	Tested By
Bear Creek Parkway	**KE140705A**	**5/21/2015**	**6/1/2015**	**MKM**
Sample Source	Sample No.	Depth (ft)	Soil Description	
EB-2	**S-4**	**10**	**gravelly SAND, some silt (SW-SM)**	
Total Sample Dry Wt. (g)	Moisture Content (%)	D_{10} (mm)	Reference Specification	
516.6	**5**	**0.195**		



Cobb.	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Sieve. No.	Diam. (mm)	Cum. Wt. Ret. (g)	% Ret. by Wt.	% Passing by Wt.	% Specs. Pass. by Wt.	
					Min	Max
3	76.1	0.0	0.0	100.0		
2.5	64	0.0	0.0	100.0		
2	50.8	0.0	0.0	100.0		
1.5	38.1	0.0	0.0	100.0		
1	25.4	0.0	0.0	100.0		
3/4	19	12.6	2.4	97.6		
3/8	9.51	30.0	5.8	94.2		
#4	4.76	82.0	15.9	84.1		
#8	2.38	124.6	24.1	75.9		
#10	2	135.0	26.1	73.9		
#20	1.18	271.0	52.5	47.5		
#40	0.6	416.5	80.6	19.4		
#60	0.25	459.1	88.9	11.1		
#100	0.149	471.5	91.3	8.7		
#200	0.074	481.4	93.2	6.8		
#270	0.053	487.0	94.3	5.7		

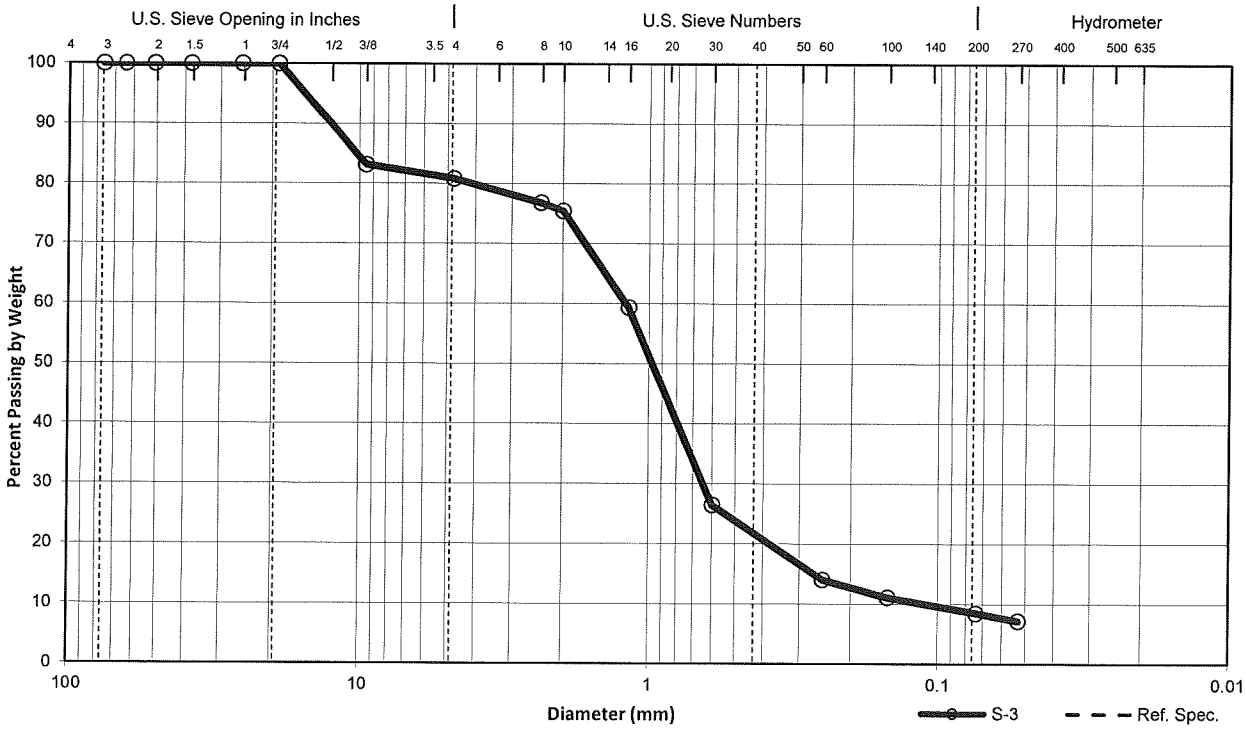
associated earth sciences incorporated

GRAIN SIZE ANALYSIS - MECHANICAL ASTM D422

Project Name	Project Number	Date Sampled	Date Tested	Tested By
Bear Creek Parkway	**KE140705A**	**5/21/2015**	**6/1/2015**	**MKM**

Sample Source	Sample No.	Depth (ft)	Soil Description
EB-3	**S-3**	**7.5**	**gravelly SAND, some silt (SP-SM)**

Total Sample Dry Wt. (g)	Moisture Content (%)	D_{10} (mm)	Reference Specification
291.6	**5**	**0.111**	



Cobb.	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Sieve. No.	Diam. (mm)	Cum. Wt. Ret. (g)	% Ret. by Wt.	% Passing by Wt.	% Specs. Pass. by Wt.	
					Min	Max
3	76.1	0.0	0.0	100.0		
2.5	64	0.0	0.0	100.0		
2	50.8	0.0	0.0	100.0		
1.5	38.1	0.0	0.0	100.0		
1	25.4	0.0	0.0	100.0		
3/4	19	0.0	0.0	100.0		
3/8	9.51	49.2	16.9	83.1		
#4	4.76	55.9	19.2	80.8		
#8	2.38	67.7	23.2	76.8		
#10	2	71.6	24.5	75.5		
#20	1.18	118.5	40.6	59.4		
#40	0.6	214.5	73.5	26.5		
#60	0.25	250.7	86.0	14.0		
#100	0.149	259.4	88.9	11.1		
#200	0.074	267.0	91.5	8.5		
#270	0.053	270.7	92.8	7.2		

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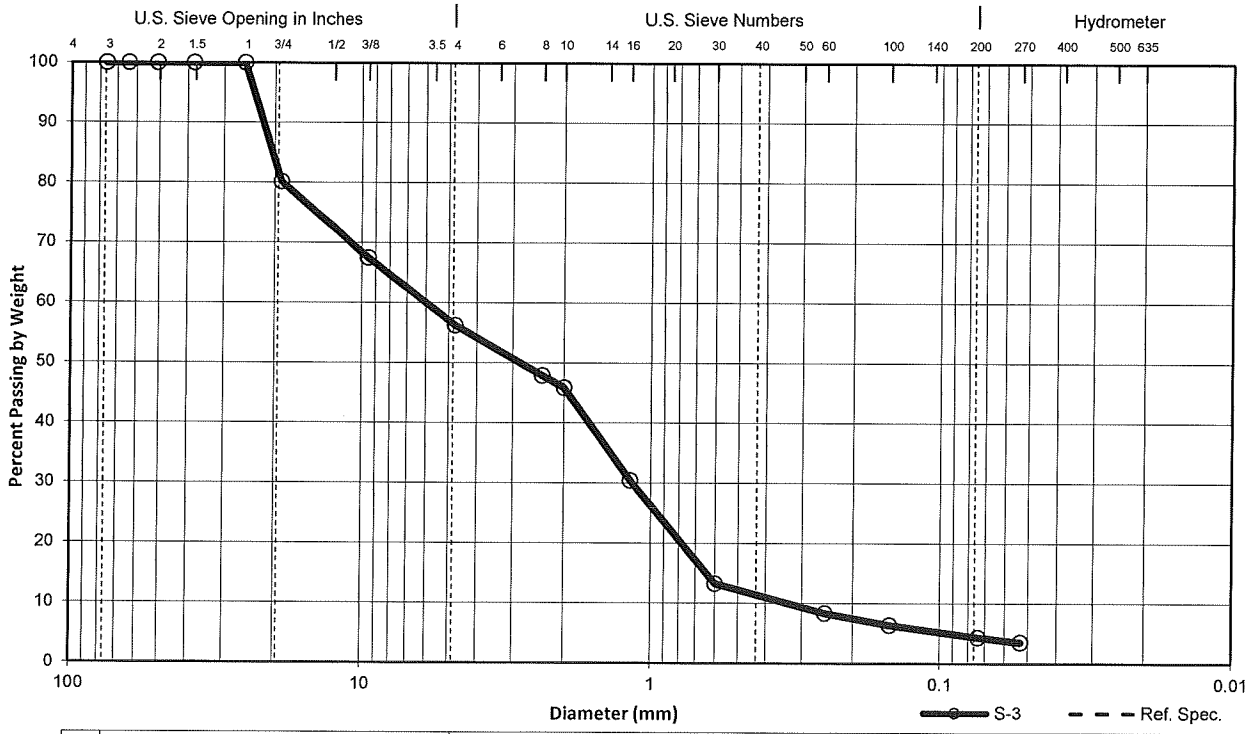
GRAIN SIZE ANALYSIS - MECHANICAL ASTM D422

Project Name	Project Number	Date Sampled	Date Tested	Tested By
Bear Creek Parkway	**KE140705A**	**5/21/2015**	**6/1/2015**	**MKM**

Sample Source	Sample No.	Depth (ft)	Soil Description
EB-5	**S-3**	**7.5**	**very gravelly SAND, trace silt (SP)**
Total Sample Dry Wt. (g)	Moisture Content (%)	D_{10} (mm)	Reference Specification
294.0	**4**	**0.336**	



Cobb.	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Sieve. No.	Diam. (mm)	Cum. Wt. Ret. (g)	% Ret. by Wt.	% Passing by Wt.	% Specs. Pass. by Wt.	
					Min	Max
3	76.1	0.0	0.0	100.0		
2.5	64	0.0	0.0	100.0		
2	50.8	0.0	0.0	100.0		
1.5	38.1	0.0	0.0	100.0		
1	25.4	0.0	0.0	100.0		
3/4	19	58.4	19.9	80.1		
3/8	9.51	95.7	32.5	67.5		
#4	4.76	128.8	43.8	56.2		
#8	2.38	153.3	52.1	47.9		
#10	2	159.3	54.2	45.8		
#20	1.18	204.6	69.6	30.4		
#40	0.6	255.1	86.8	13.2		
#60	0.25	269.5	91.7	8.3		
#100	0.149	275.3	93.6	6.4		
#200	0.074	281.2	95.6	4.4		
#270	0.053	283.6	96.5	3.5		

Kirkland Office | 911 Fifth Avenue | Kirkland, WA 98033 P | 425.827.7701 F| 425.827.5424
Everett Office | 2911 ½ Hewitt Avenue, Suite 2 | Everett, WA 98201 P | 425.259.0522 F | 425.252.3408
Tacoma Office | 1552 Commerce Street, Suite 102 | Tacoma, WA 98402 P | 253.722.2992 F | 253.722.2993
www.aesgeo.com

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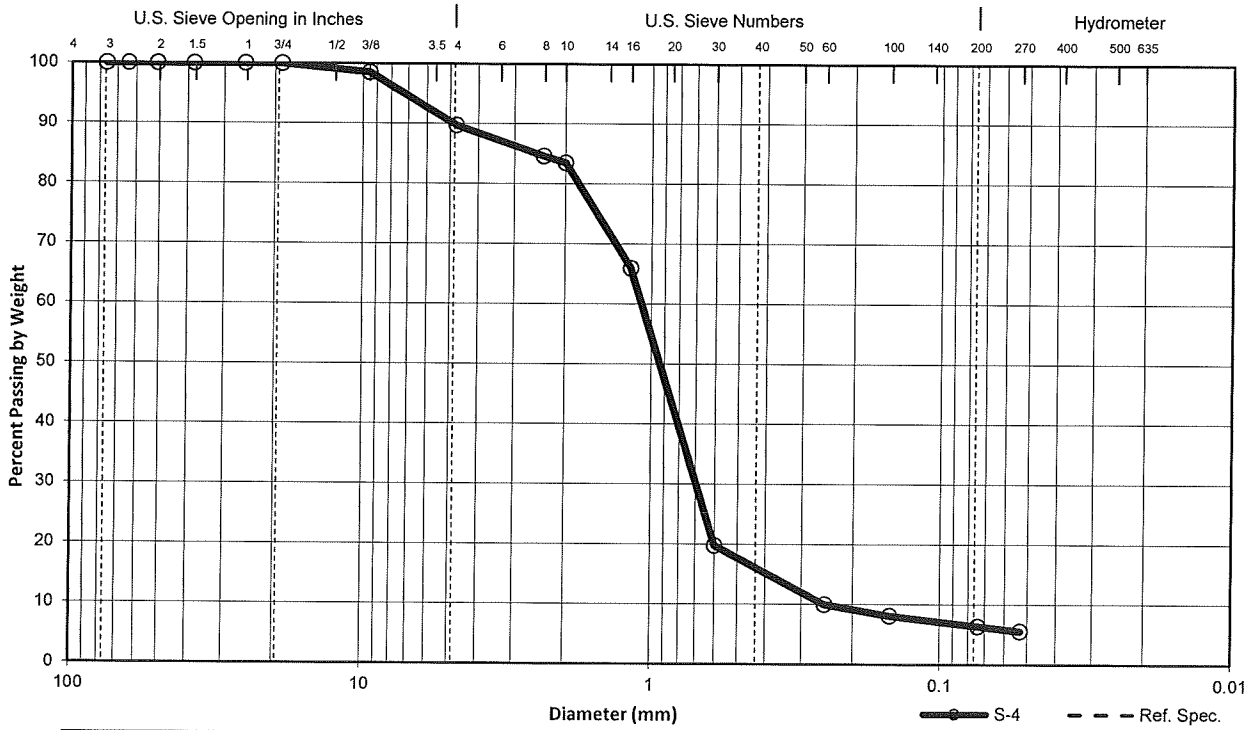
GRAIN SIZE ANALYSIS - MECHANICAL ASTM D422

Project Name	Project Number	Date Sampled	Date Tested	Tested By
Bear Creek Parkway	**KE140705A**	**5/21/2015**	**6/1/2015**	**MKM**
Sample Source	Sample No.	Depth (ft)	Soil Description	
EB-5	**S-4**	**10**	**gravelly SAND, some silt (SW-SM)**	
Total Sample Dry Wt. (g)	Moisture Content (%)	D_{10} (mm)	Reference Specification	
506.7	**9**	**0.250**		



Sieve. No.	Diam. (mm)	Cum. Wt. Ret. (g)	% Ret. by Wt.	% Passing by Wt.	% Specs. Pass. by Wt.	
					Min	Max
3	76.1	0.0	0.0	100.0		
2.5	64	0.0	0.0	100.0		
2	50.8	0.0	0.0	100.0		
1.5	38.1	0.0	0.0	100.0		
1	25.4	0.0	0.0	100.0		
3/4	19	0.0	0.0	100.0		
3/8	9.51	7.5	1.5	98.5		
#4	4.76	52.2	10.3	89.7		
#8	2.38	78.1	15.4	84.6		
#10	2	83.6	16.5	83.5		
#20	1.18	172.7	34.1	65.9		
#40	0.6	406.7	80.3	19.7		
#60	0.25	456.2	90.0	10.0		
#100	0.149	465.7	91.9	8.1		
#200	0.074	474.3	93.6	6.4		
#270	0.053	478.6	94.5	5.5		

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Percent Passing #200
ASTM D 1140

Date Sampled	Project	Project No.		Soil Description
5/21/2015	Bear Creek Parkway	KE140705A		
Tested By / Date	**Location**	**EB/EP No.**	**Depth**	
MKM / 6/1/15				

Sample I.D.	EB-6, S-6	EB-2, S-5	EB-2, S-8
Wet Weight	932.3	726.3	793.0
Dry Weight	887.9	678.4	722.5
Water Weight	44.4	47.9	70.5
Pan	518.2	296.1	452.8
Actual Dry Weight	369.7	382.3	269.7
Percent of Water Weight	12.0	12.5	26.1
After Wash Weight	368.6	376.9	248.9
Percent Passing #200	0.3	1.4	7.7

Sample I.D.	EB-6, S-5
Wet Weight	833.4
Dry Weight	806.4
Water Weight	27.0
Pan	448.5
Actual Dry Weight	357.9
Percent of Water Weight	7.5
After Wash Weight	352.0
Percent Passing #200	1.6