November 14, 2014
Project No. KE140449A

IS Property Investments, LLC
419 Occidental Avenue South, Suite 300
Seattle, Washington 98104

Attention: Mr. Pete Lymberis

Subject: Subsurface Exploration, Geologic Hazard, and
Preliminary Geotechnical Engineering Report
Rainsong Development Phase I and II
Woodinville-Redmond Road NE and NE 90th Street
Redmond, Washington

Dear Mr. Lymberis:

We are pleased to present the enclosed copies of the referenced report. This report presents updates to our subsurface exploration, geologic hazard, and geotechnical engineering report for the Phase I section of the proposed project, dated November 30, 1998, and offers recommendations for the preliminary design and development of the proposed project. This report also summarizes our work and offers recommendations for the Phase II section of the proposed project. Our recommendations are preliminary in that definite building locations and/or construction details have not been finalized at the time of this report. Detailed grading, drainage, and site development plans will be needed before a final geotechnical report can be prepared.

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

Matthew A. Miller, P.E.
Principal Engineer

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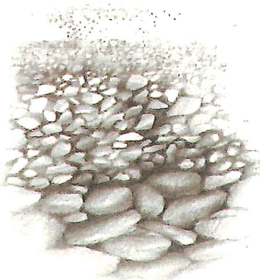

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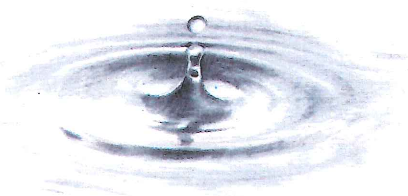

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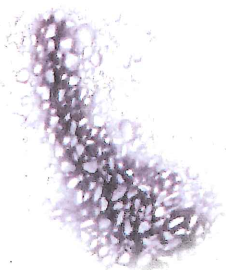

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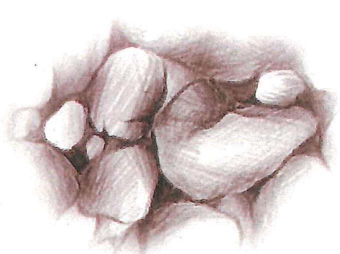

Geologic Assessments

Associated Earth Sciences, Inc.

Serving the Pacific Northwest Since 1981

Subsurface Exploration, Geologic Hazard, and
Preliminary Geotechnical Engineering Report

RAINSONG DEVELOPMENT PHASE I AND II

Redmond, Washington

Prepared for:

IS Property Investments, LLC

Project No. KE140449A
November 14, 2014

SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, AND PRELIMINARY GEOTECHNICAL ENGINEERING REPORT

RAINSONG DEVELOPMENT PHASE I AND II

Redmond, Washington

Prepared for:
IS Property Investments, LLC
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Seattle, Washington 98104

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November 14, 2014
Project No. KE140449A

I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of our subsurface exploration, geologic hazard, and preliminary geotechnical engineering study for Phase I and Phase II of the proposed Rainsong development project. Associated Earth Sciences, Inc. (AESI) previously completed a "Subsurface Exploration, Geologic Hazard, and Preliminary Geotechnical Engineering Report" dated November 30, 1998, a "Recommended Critical Area Buffer" letter dated January 7, 1999, a "Rockery Plan Review" dated June 11, 2004, and a "Geotechnical Report Update" dated August 11, 2006 for the Phase I portion of the site. This report updates our previous recommendations and presents additional data and recommendations for both Phase I and Phase II of site development. Our recommendations are preliminary in that a definite building layout and other significant construction details have not been finalized at the time of this report. The project location is shown on the attached "Vicinity Map," Figure 1. The "Site and Exploration Plan," showing approximate exploration locations completed for both our 1998 and current study, is attached as Figure 2. When project plans are near completion, the conclusions and recommendations contained in this report should be reviewed and modified, or verified, as necessary based on the final plans.

1.1 Purpose and Scope

The purpose of this report is to provide subsurface data to be utilized in the preliminary design and development of the Rainsong development project. Our original study included a review of available geologic literature, observing excavation of test pits, and performing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow ground water. Geologic hazard evaluations and engineering studies were also conducted to determine suitable geologic hazard mitigation techniques, the type of suitable foundation, allowable foundation soil bearing pressures, anticipated settlements, basement/retaining wall lateral pressures, floor support recommendations, and drainage considerations. This phase of study provides updated engineering recommendations for current codes that might have changed since the date of our original report, and an additional phase of field explorations consisting of four geotechnical exploration borings across the site. This report summarizes our previous and current fieldwork and offers development recommendations based on our present understanding of the project and updated engineering hazard analysis.

1.2 Authorization

Authorization to proceed with this study was granted by Mr. Pete Lymberis by means of an email notice to proceed. Our study was accomplished in general accordance with our proposal dated August 12, 2014. This report has been prepared for the exclusive use of IS Property Investments, LLC, 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 and engineering geology practices in effect in this area at the time our report was prepared. No other warranty, expressed or implied, is made. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner.

2.0 PROJECT AND SITE DESCRIPTION

This report was completed with an understanding of the project based on discussions with Mr. Pete Lymberis, our knowledge of the site based on our previous work for the project, and on review of proposed site plans and topography, prepared by Freiheit & Ho Architects, dated November 5, 2014. Present plans call for the construction of 44 attached single-family residential units in eight buildings. Development will consist of four buildings on the lower half of the property, accessed from Woodinville-Redmond Road, and another four buildings situated on the eastern, higher portion of the site, accessed from NE 91st Street. It is our understanding that the buildings will consist of two-story structures over daylight basements, with four to eight units per building. We anticipate that the structures will use conventional wood-frame construction, with slab-on-grade basement floors. Maximum cuts and walls of up to about 10 feet in height are anticipated in order to construct the proposed structures. To our knowledge, infiltration of storm water is not being considered for the site.

The property is located on the northeast side of Woodinville-Redmond Road, opposite its intersection with NE 90th Street in Redmond, Washington. The irregularly shaped parcel consists of three parcels (King County Parcel Nos. 0225059-005, -209, -201) and it is anticipated that an additional swath of property, about 50 feet wide, from the adjacent church (King County Parcel No: 7200000350) will be incorporated into the Rainsong property. The total property area encompasses an area of approximately 3.1 acres, and is located on a southwest-facing slope. Slope gradients in the western, lower portion of the site range from approximately 10 to 30 percent. Farther up the slope to the east, the gradients steepen to approximately 30 to 40 percent. The total elevation change across the property is approximately 105 feet. Two existing structures are present in the eastern portion of the site. These include a two-story duplex, and a two-story, six-unit apartment building. Access to the buildings is provided by an asphalt-paved driveway, which enters the property off of Woodinville-Redmond Road, and winds up the slope to an asphalt-paved parking lot at the upper, east end of the site. During our reconnaissance of the site in 1998, we also observed remnants of a building foundation near the lower, west end of the site, just north of exploration pit EP-2 (Figure 1). Vegetation on the site consists of scattered groups of deciduous and coniferous trees, with moderate to thick natural undergrowth and blackberry thickets.

3.0 SUBSURFACE EXPLORATION

Our field study included observation of eight exploration pits in 1998, the current advancement of four exploration borings, and performing geologic hazard reconnaissance to gain information about the site. 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. If changes occurred between sample intervals in our borings, they were interpreted. Our explorations were approximately located in the field by measuring from known site features shown on the "Site and Exploration Plan," Figure 2. We have also completed two geologic cross-sections showing our interpretation of the site's subsurface geology across cross-section lines A-A' (Figure 3) and B-B' (Figure 4).

The conclusions and recommendations presented in this report are based on the eight exploration pits and four exploration borings completed for this study. The number, location, and depth of the explorations were completed within site and budget 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

Exploration pits were excavated in 1998 with a tractor-mounted backhoe. The pits permitted direct, visual observation of subsurface conditions. Materials encountered in the exploration pits were studied and classified in the field by a geologist from our firm. All exploration pits were backfilled immediately after examination and logging. Selected samples were then transported to our laboratory for further visual classification and testing, as necessary.

3.2 Exploration Borings

The exploration borings were completed October 30, 2014 by advancing a 4-inch inside-diameter, hollow-stem auger with a trailer-mounted drill rig. During the drilling process, samples were obtained at generally 5-foot-depth intervals. The borings were continuously observed and logged by a geotechnical engineer from our firm. The exploration logs presented in the Appendix 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 50 blows for the 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 watertight containers. The samples were then transported to our laboratory for further visual classification and laboratory testing, as necessary.

4.0 SUBSURFACE CONDITIONS

Subsurface conditions at the project site were inferred from the field explorations accomplished for this study, and visual reconnaissance of the site. As shown on the field logs, the explorations generally encountered natural deposits, consisting of loose to medium dense sand and gravel, with variable amounts of silt, overlying dense to very dense/hard sediments of silty fine sand and silt. Minor amounts of fill, debris flow, and ice contact deposits were also encountered at shallow depths (less than 10 feet) in the lower portion of the site. The following section presents more detailed subsurface information, organized from the upper (youngest) to the lower (oldest) sediment types.

4.1 Stratigraphy

Fill Soil

Fill soils (those not naturally placed) were encountered at the locations of exploration pits EP-1 and EP-2, and borings EB-3 and EB-4. The fill encountered in the exploration pits and EB-3 consisted of loose to medium dense, moist to wet, brown, silty sand with varying amounts of gravel, occasional roots, and scattered wood debris. Fill thicknesses encountered at the locations of pits EP-1 and EP-2 ranged from 2 to 3½ feet, respectively. Fill in boring EB-3 was about 8 to 9 feet thick, and is likely the result of grading the road that currently ascends the site from west to east. The existing fill is not suitable for foundation support.

Topsoil

A surficial topsoil layer, approximately ½ foot in thickness, was encountered at the locations of exploration pits EP-3 through EP-8, and in boring EB-2. The topsoil contains substantial quantities of organic matter and is unsuitable for foundation or pavement support.

Debris Flow Deposits

The sediment encountered below the surficial fill layer at the locations of exploration pits EP-1 and EP-2 consisted of debris flow deposits. At the location of pit EP-1, the debris flow deposits were approximately 1½ feet in thickness, and consisted of loose to medium dense, silty sand with gravel, some cobbles, scattered boulders, and lumps of silt. At the location of pit EP-2, the debris flow deposits consisted of approximately 4½ feet of medium dense, moist, silty sand with some gravel and scattered cobbles, underlain by a 1½-foot-thick bed of stiff, wet, mottled, blue-gray silty clay. The clay layer exhibited a sheared appearance, and contained moderate amounts of organic debris.

Recessional Outwash

This unit was encountered below the surficial topsoil layer at the locations of exploration pits EP-4 through EP-8. These sediments generally consisted of loose to medium dense, tan to gray sand, with variable amounts of silt and gravel. The upper 1½ to 2½ feet of this unit was weathered to a reddish brown color. The recessional outwash sediments were deposited by meltwater streams emanating from the retreating glacial ice during the latter part of the Vashon Stade of the Fraser Glaciation, approximately 12,500 years ago. This unit generally extended to a depth of 3 to 4½ feet, but at the location of exploration pit EP-7, it extended to a depth of approximately 7½ feet. Recessional outwash was not encountered in the exploration borings.

Ice Contact Sediments

Sediments encountered underlying fill or recessional outwash in pits EP-1 through EP-3 and in borings EB-3 and EB-4 generally consisted of dense to hard, unsorted, silty gravel to sandy silt with variable gravel, silt, and cobble content. Gravels/cobbles were typically coated in a thin veneer of silt/clay. These sediments were interpreted to be representative of ice contact deposits. Ice contact deposits consist of sediments deposited in contact with glacial ice by meltwater on, under, within, or marginal to the glacier. At the locations of exploration pits EP-1 and EP-2, the ice contact sediments extended to the full depths explored of approximately 6 feet and 10½ feet, respectively. At the location of exploration pit EP-3, this unit extended to a depth of approximately 7.5 feet. In borings EB-3 and EB-4, this unit extended to depths of 17 and 7.5 feet, respectively. It is anticipated that a relatively thin layer (less than 10 feet thick) of ice contact deposits is draped over most of the lower half of the site, as shown in Figures 3 and 4.

The ice contact deposits encountered on the site are suitable for support of foundations, floor slabs, and paving, with proper preparation. Ice contact deposits are silty and moisture-sensitive. In the presence of moisture contents above the optimum moisture content for compaction purposes, these deposits can be easily disturbed by vehicles and earthwork equipment. Careful management of moisture-sensitive soils, as recommended in this report, will be needed to reduce the potential for disturbance of wet soils and costs associated with repairing disturbed soils.

Lodgement Till

Although lodgement till sediments were interpreted to have been encountered in exploration pits EP-3 and EP-7 from our 1998 field study, additional analysis during our current phase of fieldwork suggests that Vashon-age lodgement till (approximately 12,500 to 15,000 years old) is not present on this site. Sediments in EP-3 and EP-7 that were previously classified as lodgement till are more likely silty sand outwash deposits or pre-Fraser deposits, as discussed below.

Advance Outwash

Sediments encountered below the recessional outwash at the locations of exploration pits EP-4, EP-5, EP-6, and EP-8, and below the lodgement till at the location of exploration pit EP-7, generally consisted of dense to very dense, stratified sand and gravel deposits, with variable amounts of silt. In our 1998 report, these sediments were interpreted to be representative of advance outwash deposits. The advance outwash was deposited by meltwater streams from the advancing glacial ice during the Vashon Stade of the Fraser Glaciation approximately 12,500 to 15,000 years ago. Because the outwash was deposited during the advance of the glacial ice, it was overridden by the glacier subsequent to its deposition, and therefore, is consolidated in its unweathered state.

Our additional explorations from this phase of fieldwork did not encounter Vashon advance outwash sediments. Although it is possible that our explorations “missed” the unit previously classified as Vashon outwash deposits, an alternate interpretation suggests this unit may be part of the ice contact deposits identified in this phase of exploration. Whether these sediments are ice contact or advance outwash, the near-surface sediments (within 10 to 15 feet) should be considered moisture-sensitive due to the high silt content, and should be carefully managed to reduce the potential for disturbance of wet soils and costs associated with repairing disturbed soils. Dense/hard advance outwash and ice contact deposits are considered suitable for support of foundations, floor slabs, and paving, with proper preparation.

Pre-Fraser Outwash Deposits

Sediments encountered below the pavement at the location of boring EB-1 consisted of dense to very dense, stratified gravelly sand deposits with variable amounts of silt. These sediments were interpreted to be representative of pre-Fraser outwash deposits, which were deposited by meltwater streams emanating from glacial ice during pre-Fraser times (>20,000 years before present). The outwash was overridden by the glacier subsequent to its deposition, and therefore, is consolidated in its unweathered state. It is considered suitable for support of foundations, floor slabs, and paving, with proper preparation. It may be moisture-sensitive and should be carefully managed to reduce the potential for disturbance of wet soils and costs associated with repairing disturbed soils.

Pre-Fraser Non-Glacial Deposits (Younger and Older)

Sediments encountered below the pre-Fraser outwash in boring EB-1, and just below the topsoil in boring EB-2, consisted of dense to very dense, brown to bluish gray, very silty fine sand, with varying gravel contents. Occasional small fragments of organic material were present. These sediments were interpreted to be representative of pre-Fraser non-glacial deposits, and were deposited during a non-glacial period of time in the Puget Sound region. The sediments were subsequently overridden by at least the Vashon ice sheet and consolidated. The very moist conditions of the retrieved samples and drilled cuttings indicate that there might be a thin zone of “perched” water near the base of this formation, above the lower-permeability glaciomarine drift deposits (described below). Geologic Cross-Section A-A’ (Figure 3) shows two different pre-Fraser non-glacial units were encountered in EB-2. The non-glacial units were separated by a glaciomarine deposit (described below) and are interpreted to represent different geologic units of undetermined age.

Pre-Fraser non-glacial deposits are considered suitable for support of foundations, floor slabs, and paving, with proper preparation. They are moisture-sensitive and should be carefully managed.

Pre-Fraser Glaciomarine Deposits (Younger and Older)

Sediments encountered below the ice contact deposits in borings EB-3 and EB-4, and below pre-Fraser non-glacial deposits in EB-2, generally consisted of very stiff to hard, laminated gray silt, which exhibited a somewhat fractured texture, was slightly moist to very moist, and reacted chemically with hydrochloric acid. These sediments were interpreted to be representative of pre-Fraser glaciomarine drift (GMD) deposits, and were deposited in a marine environment. GMD can sometimes react when exposed to hydrochloric acid due to the presence of disseminated calcium carbonate. The sediments were subsequently overridden by the Vashon ice sheet and consolidated. Geologic Cross-Section A-A' (Figure 3) shows two different pre-Fraser glaciomarine units subdivided into younger and older. The younger pre-Fraser glaciomarine drift was only encountered in EB-2 were it was less than 15 feet thick. The base of the younger pre-Fraser glaciomarine drift was marked by an organic-rich non-glacial unit encountered in EB-2 at a depth of approximately 32 feet. The older glaciomarine drift was encountered in EB-3 and EB-4 at depths of about 18 and 8 feet below ground surface, respectively.

Pre-Fraser GMD is considered suitable for support of foundations, floor slabs, and paving, with proper preparation. It is moisture-sensitive and should be carefully managed.

4.2 Hydrology

Ground water was encountered in exploration borings EB-2, EB-3, and EB-4, at depths of approximately 23.5 feet, 17.5 feet, and 16 feet, respectively. The ground water observed in our borings is interpreted to represent thin zones of "perched" ground water within the ice contact or pre-Fraser non-glacial deposits, above low-permeability GMD silts. Perched water occurs when surface water infiltrates down through relatively permeable soils and becomes trapped or "perched" atop a comparatively impermeable barrier such as the GMD. This water may travel as interflow and typically will follow the ground surface topography. The duration and quantity of interflow seepage will largely depend on the soil grain size distribution, topography, and seasonal precipitation. Ground water levels that were observed during the short period of time that the explorations were open may not represent equilibrium levels, which could be shallower. Ground water conditions should be expected to vary in response to changes in seasonal precipitation, on- and off-site land usage, and other factors.

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.

5.0 SLOPE HAZARDS AND MITIGATIONS

Slope gradients at the site are moderate (approximately 10 to 40 percent). The sediments underlying the slope generally consist of glacially consolidated sand and silt, with relatively thin, surficial deposits of loose to medium dense, weathered glacial sediments and recessional outwash. Ground water was encountered within the pre-Fraser non-glacial sediments and appears to be "perched" above the lower-permeability pre-Fraser glacial and glaciomarine deposits.

We understand that the project is regulated under the City of Redmond Zoning Code (RZC). Section 21.064.060 of the RZC defines landslide hazard areas as any area with a slope 40 percent or steeper with a vertical relief of 10 feet or more. The RZC prohibits most development within a landslide hazard area buffer, which is defined as 50 feet from the top or toe of the slope. However, the buffer may be reduced to a minimum of 15 feet upon approval of a geotechnical engineer.

The sediments underlying the slope generally consist of glacially consolidated sand, silt, and gravel, with relatively thin, surficial deposits of loose to medium dense, weathered glacial sediments and recessional outwash. Adverse ground water conditions were not observed in the explorations accomplished for our study. Based on the subsurface conditions encountered, it is our opinion that a minimum buffer of 15 feet from areas in excess of 40 percent grade that exceeds 10 feet in vertical height is sufficient to adequately protect the proposed and surrounding developments from the critical landslide hazard. AESI should be provided a copy of the grading plan and updated topographic survey for review when it becomes available. We also recommend that the surveyor's scope of work include identifying areas that meet RZC geometric criteria for landslide hazard areas.

Debris flow deposits were encountered near the toe of the slope at the locations of exploration pits EP-1 and EP-2. The presence of these sediments indicates that some earth movement has occurred at the site in the past. Where encountered in our explorations, the debris flow deposits were of limited thickness. Consequently, the debris flow sediments are indicative of shallow, surficial earth movement, and not a deep-seated landslide. In addition, it is our opinion that the debris flows likely occurred during pre-historic, not recent times, and the conditions that caused the debris flow no longer exist. No other evidence of past earth movement at the site was observed. Development of the site will control storm water such that the site stability will be further improved. Therefore, it is our opinion that the risk of damage to the proposed structures by landsliding is low, provided that the recommendations presented in this report are properly followed. Risk increases during or following sustained, heavy precipitation events and/or significant seismic events.

In order to mitigate potential hazards associated with the debris flow deposits in the lower portion of the slope, we recommend that buildings constructed in this area be founded upon the underlying, undisturbed, dense/hard glacial sediments. Specific recommendations for building support are provided in the "Foundations" section of this report.

6.0 SEISMIC HAZARDS AND MITIGATIONS

Earthquakes occur in the Puget Lowland with great regularity. The vast majority of these 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 1965, 6.5-magnitude event; and the 2001, 6.8-magnitude event. The 1949 earthquake appears to have been the largest in this area during recorded history. Evaluation of return rates indicates that an earthquake of a magnitude between 6.0 and 7.0 is likely within a given 25- to 40-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 project site is located approximately 6 miles north of the Seattle Fault Zone. Recent studies by the United States Geological Survey (USGS; e.g., Johnson et al., 1994, *Origin and Evolution of the Seattle Fault and Seattle Basin, Washington*, Geology, v. 22, p.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, p. 1042-1053) have provided evidence of surficial ground rupture along a northern splay of the Seattle Fault. The recognition of this fault is relatively new, and data pertaining to it are limited, with the studies still ongoing. 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.

The recurrence interval for movement along this fault system is still unknown, although it is hypothesized to be in excess of several thousand years. Due to the suspected long recurrence interval and distance from the fault zone, the potential for surficial ground rupture at the site is considered to be low during the expected life of the structures and no mitigation efforts beyond complying with the 2012 *International Building Code* (IBC) are recommended.

6.2 Seismically Induced Landslides

The on-site, natural sediments found during the explorations are glacially consolidated materials and are not sensitive to landsliding given the topographic conditions at the site. No current evidence of landslide activity causing distress to surrounding structures was observed. Given the subsurface and topographic conditions within and adjacent to the proposed development area, it is our opinion that the risk of damage to the proposed project by landsliding is low. This opinion is dependent upon site grading and construction practices being completed in accordance with the geotechnical recommendations presented in this report.

6.3 Liquefaction

Liquefaction is a condition where loose, saturated, typically fine-grained soils lose shear strength when subjected to high-intensity cyclic loads, such as occur during earthquakes. The resulting reduction in strength can cause differential foundation settlements and slope failures. Loose, saturated, fine-grained soils that cannot dissipate the buildup of pore water pressure are the predominant type of sediments subject to liquefaction.

The observed site soils were dense/hard, and where saturated consisted of hard silts. These soils are not expected to be prone to liquefaction. A detailed liquefaction hazard analysis was not performed as part of this study, and none is warranted, in our opinion.

6.4 Seismic Site Class (2012 IBC)

In our opinion, the subsurface conditions at the site are consistent with seismic Site Class "D" in accordance with the 2012 IBC, and the publication ASCE 7 referenced therein, the most recent version of which is ASCE 7-10.

7.0 EROSION HAZARDS AND MITIGATIONS

As of October 1, 2008, the Washington State Department of Ecology (Ecology) Construction Storm Water General Permit (also known as the National Pollutant Discharge Elimination System [NPDES] permit) requires weekly Temporary Erosion and Sedimentation Control (TESC) inspections and turbidity monitoring for all sites 1 or more acres in size that discharge storm water to surface waters of the state. Because we anticipate that the proposed project will require disturbance of more than 1 acre, we anticipate that these inspection and reporting requirements will be triggered. The following recommendations are related to general erosion potential and mitigation.

The erosion potential of the site soils is moderate, but may be high if steep slopes remain unvegetated during construction. The most effective erosion control measure is the maintenance of adequate ground cover. Maintaining cover measures atop disturbed ground provides the greatest reduction to the potential generation of turbid runoff and sediment transport. During the local wet season (October 1 through March 31), exposed soil should not remain uncovered for more than 2 days unless it is actively being worked. Ground-cover measures can include erosion control matting, plastic sheeting, straw mulch, crushed rock or recycled concrete, or mature hydroseed.

To mitigate the erosion hazards and potential for off-site sediment transport, we recommend the following:

1. The winter performance of a site is dependent on a well-conceived plan for control of site erosion and storm water runoff. It is easier to keep the soil on the ground than to remove it from storm water. The owner and the design team should include adequate ground-cover measures, access roads, and staging areas in the project bid to give the selected contractor a workable site. The selected contractor needs to be prepared to implement and maintain the required measures to reduce the amount of exposed ground. A site maintenance plan should be in place in the event storm water turbidity measurements are greater than the Ecology standards.

2. All 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.

3. 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 showers occur, 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.

4. 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.

5. Surface runoff and discharge should be controlled during and following development. Uncontrolled discharge may promote erosion and sediment transport. Under no circumstances should concentrated discharges be allowed to flow over significant slopes.

6. 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 1 and March 31, these measures are required.

7. On-site erosion control inspections and turbidity monitoring should be performed in accordance with Ecology requirements. Weekly and monthly reporting to Ecology should be performed on a regularly scheduled basis. TESC monitoring should be part of the weekly construction team meetings. Temporary and permanent erosion control and drainage measures should be adjusted and maintained, as necessary, at the time of construction.

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

III. PRELIMINARY DESIGN RECOMMENDATIONS

8.0 INTRODUCTION

Our exploration indicates, from a geotechnical standpoint, the site is suitable for the proposed development, provided the recommendations contained herein are properly followed. The bearing stratum, consisting of medium dense to very dense glacial and pre-Fraser consolidated sediments, was generally encountered at depths of approximately 2 to 3 feet below the existing grades. However, in the lower portion of the site, at the locations of exploration pits EP-1 and EP-2, and at boring EB-3, where the bearing sediments were overlain by existing fill and debris flow deposits, the depth to bearing soils ranged from approximately 4 feet to 9½ feet. Existing fill soils are also likely present beneath the existing driveway, parking area, duplex, and apartment building. Consequently, the depth to bearing sediments in these areas should be expected to be somewhat deeper than the 2 to 3 feet typical for the undisturbed portions of the site. Conventional spread footing foundations may be suitable for building support in all areas of the site. However, in the lowermost portion of the site, in the area of exploration pits EP-1 and EP-2, where the depth to bearing soils is greater, it may be more economical to overexcavate and use rock trenches if the fill is not completely removed for the placement of the daylight basements. Other measures such as some type of deep foundation including drilled-in-place, concrete piers, or pipe piles may also be considered depending upon the final grading of the building pads. The final foundation alternative may not be evident until the grading plan is completed. Detailed recommendations are presented in the “Foundations” section below.

Our recommendations also assume that landslide hazard area buffers (as defined by the RZC), if present, are located such that they will not interfere with any proposed development, or that any proposed development will result in less-steep slopes and therefore more stable slope conditions. Due to the preliminary stage of the project, this report contains general guidelines for design of walls and foundations. Once a final plan has been decided upon, we recommend we be allowed to review completed plans to confirm that our geotechnical recommendations have been adequately interpreted and incorporated into the design.

9.0 SITE PREPARATION

Following demolition of the existing structures on the site, any remaining foundation elements that are under the proposed driveway or building areas should be removed. Any buried utilities should be removed or relocated if they are under the building areas. The resulting depressions should be backfilled with structural fill, as discussed under the “Structural Fill” section of this report.

Site preparation of the planned building and driveway areas should include removal of all trees, brush, debris, and any other deleterious material. Additionally, the upper, organic topsoil should be removed, and the remaining roots grubbed.

If drilled piers or pipe piles are utilized for foundation and floor support under buildings in the lower portion of the site, no additional site preparation will be required below these areas after excavation to the desired building subgrade. For those areas where conventional spread footings are to be utilized for foundation support, any areas where loose, surficial soils exist due to grubbing operations should be considered as fill to the depth of disturbance, and treated as subsequently recommended for structural fill placement. Areas of existing fill soils should be stripped down to the underlying natural, recessional outwash, ice contact, or pre-Fraser sediments. The reddish brown, weathered glacial sediments present within approximately 2 to 3 feet of the ground surface across the majority of the site generally contained moderate to substantial quantities of roots, and should be removed below building footprints.

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 loose to medium dense, natural sediments can be made at a maximum slope of 1.5H:1V (Horizontal:Vertical). Unsupported cut slopes in the very dense, glacial sediments can be made at a maximum slope of approximately 1H:1V. 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.

Permanent, unsupported cut slopes should not exceed a maximum gradient of 2H:1V.

Portions of the on-site soils contain a high percentage of 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 in these areas are not softened. If disturbance occurs, the softened soils should be removed and the area brought to grade with structural fill. Consideration should be given to protecting access and staging areas with an appropriate section of crushed rock during wet weather construction.

If crushed rock is considered for the access and staging areas, it should be underlain by engineering stabilization fabric 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.

10.0 STRUCTURAL FILL

We anticipate that placement of structural fill will be necessary to establish desired grades in some areas. For foundations supported by recessional outwash sediments, the upper 1 foot of these materials should be removed, the upper 12 inches of exposed subgrade recompacted to a firm and unyielding condition, and the 1 foot of excavated soil replaced as 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.

After overexcavation/stripping has been performed to the satisfaction of the geotechnical engineer/engineering geologist, the upper 12 inches of 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 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, the backfill should be placed and compacted in accordance with current local or county codes and standards. The top of the compacted fill should extend horizontally outward a minimum distance of 3 feet beyond the location of the perimeter footings or roadway edges before sloping down at a maximum angle of 2H:1V. Structural fill placed in foundation excavations must extend a minimum distance of 2 feet beyond the edges of the footings.

If fill is to be placed on slopes steeper than 5H:1V, the base of the fill should be tied to firm, stable subsoil by appropriate keying and benching, which would be established in the field to suit the particular soil conditions at the time of grading. The keyway will act as a shear key to embed the toe of the new fill into the hillside. Generally, the keyway for hillside fills should be at least 8 feet wide and cut into suitable native soils. Level benches would then be cut horizontally across the hill, following the contours of the slope. No specific width is required for the benches, although they are usually a few feet wider than the dozer being used to cut them. All fills proposed over a slope should be reviewed by our office prior to construction. All fill placement on this site carries potentially significant slope stability implications, and the project should be laid out in such a way as to minimize placement of new fill above existing and proposed slopes.

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 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 soils in structural fills should be limited to favorable dry weather conditions. The on-site soils are suitable for use as structural fill, but generally contained significant amounts of silt and are considered to be moisture-sensitive. During grading, we suggest that cleaner material be segregated and stockpiled separately from the more silty soils. In addition, 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. At least 30 percent of the free-draining fill should consist of material retained on the No. 4 sieve, with no fraction exceeding a diameter of 6 inches.

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.

11.0 FOUNDATIONS

Spread footings may be utilized for building support when founded either directly on the medium dense to very dense glacial sediments, recompacted outwash sediments, or on structural fill placed as described under the “Site Preparation” and “Structural Fill” sections of this report. For areas underlain by recessional outwash sediments, the upper foot should be compacted to a firm and unyielding condition, and then the initial foot replaced and compacted as structural fill. Structural fill placed below footings must also extend a minimum of 2 feet beyond the edges of the footings. Without a grading plan, we are unsure which buildings will be founded upon native soils at grade, or deeper sediments due to substantial cuts.

For footings founded either directly upon the medium dense to very dense glacial sediments, recompacted recessional outwash sediments, or structural fill as described above, we recommend that an allowable bearing pressure of 2,500 pounds per square foot (psf) be utilized for design purposes, including both dead and live loads. An increase of one-third may be used for short-term wind or seismic loading. Perimeter footings for the proposed buildings should be buried a minimum of 18 inches into the surrounding soil for frost protection. No minimum burial depth is required for interior footings; however, all footings must penetrate to the prescribed stratum and no footings should be founded in or above loose, organic, or existing fill soils. To limit settlements, all footings should have a minimum width of 18 inches.

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 which has not been compacted to at least 95 percent of ASTM:D 1557. In addition, a 1.5H:1V line extending down from any footing must not daylight because sloughing or raveling may eventually undermine the footing. Thus, footings should not be placed near the edge of steps or cuts in the bearing soils.

Anticipated settlement of footings founded as described above should be on the order of 1 inch. However, disturbed soil not removed from footing excavations 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 design bearing capacity of the soils has been attained and that construction conforms with the recommendations contained in this report. Such inspections may be required by the governing municipality. Perimeter footing drains should be provided as discussed under the “Drainage Considerations” section of this report.

As previously stated, exploration pits EP-1 and EP-2, located in the lower (western) portion of the site, encountered several feet of existing fill soils and debris flow deposits not suitable for foundations support. At the locations of pits EP-1 and EP-2, the foundation bearing sediments, consisting of dense, ice contact sediments, were encountered at depths of approximately 4 feet and 9½ feet, respectively. Based upon the explorations completed for our study, it appears that the fill and debris flow sediments may extend below the footprint area proposed for the two westernmost buildings. Because of the depth to the bearing stratum in this area, it may be more economical to utilize a deep foundation system consisting of drilled-in-place concrete piers or pipe piles rather than a conventional spread footing foundation, as described in Sections 11.2 “Cast-in-Place Concrete Piles/Piers and 11.3 “Pipe Piles.”

11.1 Rock Trenches

Because the fill may be considered too deep to economically extend the footings down to suitable bearing, rock trenches extended down to the dense native soils are an alternative for foundation support.

The trenches should have a minimum width of 4 feet (or as designated by the field engineer/engineering geologist) and be excavated down to the dense natural soils. Because of the potential for caving, the actual trench width may be greater than specified. It would be appropriate to backfill the trenches as the excavation proceeds to reduce caving. The use of a larger, track-mounted backhoe will greatly speed trench excavation over the use of a conventional, rubber-tired backhoe. In order to reduce disturbance of the bearing soils exposed in the trench, it is strongly recommended that the teeth of the backhoe bucket be covered with a digging plate.

To determine when suitable bearing has been achieved and to verify proper rock placement, the geotechnical engineer/engineering geologist must be present on a full-time basis during footing trench excavation and backfill. Any potential seepage entering the excavation on an overnight basis must be removed prior to commencing trench excavation the following day.

After the bearing stratum has been reached, the trench should be immediately backfilled. We recommend the use of “railroad ballast” or 2- to 4-inch-size crushed rock for backfill. The crushed rock must be tamped into place to achieve a tightly-packed mass; this may be done with either a “Hoepac”-type compactor mounted on the backhoe or more typically, with the bucket of the backhoe itself. Staging areas should be maintained so that the rock is not contaminated by mud prior to placement in the trench. Equipment access to trench locations should also be maintained.

Spread footings may then be used for building support when placed over properly constructed rock trenches. Footings which bear on approved rock trenches may be designed for an allowable bearing pressure of 2,500 psf including both dead and live loads. An increase of one-third may be used for short-term wind or seismic loading. However, all rock trenches must penetrate to the prescribed bearing stratum and no trenches should be founded in or above loose, organic, or existing fill soils. In addition, all footings must be centered over the trenches and have a minimum width of 14 inches for one-story structures, 16 inches for two-story structures, and 18 inches for three-story structures.

All footing areas should be inspected by AESI prior to placing concrete, to verify that the rock trenches are undisturbed and construction conforms with the recommendations contained in this report. Such inspections may be required by the governing municipality.

11.2 Cast-in-Place Concrete Piles/Piers

Cast-in-place concrete piles/piers could also be used for foundation support, and might reduce or eliminate the need for shoring walls. We recommend that the placement of all piles be accomplished by a contractor experienced in their installation. Soils encountered at the site may have gravel lenses or boulders present in them.

All piles should penetrate a minimum of 10 feet into the native soils. Piles with a minimum diameter of 12 inches will be capable of supporting allowable axial compressive capacities of 15 tons per pile if installed as noted above. Allowable design loads may be increased by one-third for short-term wind or seismic loading. Anticipated settlements of pile-supported structures will generally be on the order of ½ inch or less. Larger loads may be obtained with larger-diameter piling, if required.

Lateral Pile Capacity

Although the majority of lateral resistance to wind and seismic loading will be generated by the pile caps and grade beams, the piles will provide an additional capacity of 5 tons (applied at the pile top) per pile, assuming that the pile heads are fixed against rotation by a rigid pile cap. Piles within 10 feet of another pile along the direction of force should be considered to be in the zone of influence and the lateral capacity of only one of these piles should be used in design. If the lateral contribution of the piles is more critical to the practical design of the structure, we can provide a comprehensive lateral pile analysis. Such an analysis would present lateral pile capacities taking into account the interaction between piles.

Pile/Pier Inspections

The actual total length of each pile may be adjusted in the field based on required capacity and conditions encountered during drilling. Since completion of the pile 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, use of the presented pile capacities in the design requires that all piles be inspected by a qualified geotechnical engineer or engineering geologist from our firm who can interpret and collect the installation data and examine the contractor's operations. AESI, acting as the owner's field representative, would determine the required lengths of the piles and keep records of pertinent installation data. A final summary report would then be distributed, following completion of pile installation.

11.3 Pipe Piles

Pipe piles consisting of 3-, 4-, or 6-inch-diameter, driven steel pipe sections may be another alternative to foundation support. The pipe piles should be driven to refusal with equipment appropriate to the pipe diameter. Multiple pipe sections should be joined with compression fittings that fit inside the pipe or welding of the pipe sections. Table 1 summarizes typical wall thicknesses, driving equipment, refusal criteria, and allowable axial compressive loads for each pipe diameter. On-site load testing of at least two piles will need to be performed to at least 200 percent of the design load to verify that the pile capacities are achievable in the site soils.

The load test procedures should be observed by an AESI representative and the test results reviewed by an AESI geotechnical engineer.

Table 1
Pipe Pile Summary

Nominal Pile Diameter (inches)	Wall Thickness	Typical Installation Equipment	Refusal Criteria* (seconds/inch)	Allowable Axial Compressive Load** (kips)
3	Schedule 40	650-lb hydraulic hammer	20	10
4	Schedule 40	850-lb hydraulic hammer	15	16
6	Schedule 40	1,250-lb hydraulic hammer	15	20

* Based on listed installation equipment. Other equipment may alter refusal criteria.
** Allowable loads may be increased with acceptable load testing to twice the design load.

If uplift loads are expected to be placed on the piles at any time, the connections must be welded. Uplift capacity of pipe piles is typically low, and AESI should be contacted if the designer requires uplift capacity for the piles. Piles may be battered up to 15 degrees to develop lateral capacity. Battered piles inclined up to 15 degrees should be designed with an allowable axial compressive capacity equal to that used for vertical piles. Although vertical pipe piles can provide some lateral resistance, we recommend that these contributions be neglected in designing the new foundation system. Lateral resistance at the foundation level may be provided by passive soil resistance acting as grade beams, as described in the following section. The structural engineer should provide pile spacing, locations, splicing details, foundation connection details, and any other structural design recommendations that are needed. No minimum pile spacing requirements are necessary for pipe piles from a geotechnical standpoint. We anticipate that piles will have to be driven to 20 to 25 feet to reach the refusal criteria.

Since pipe piles are driven until specific refusal criteria are achieved, rather than to a specific depth, accurate estimation of pile lengths is not possible. We recommend that AESI be retained to observe pile installation to confirm that our recommendations have been implemented, to verify that appropriate installation procedures are used, and that the appropriate refusal criteria are achieved.

Grade beams and pile caps that are backfilled with structural fill may be designed for passive resistance against lateral translation using an equivalent fluid equal to 250 pounds per cubic foot (pcf).

12.0 LATERAL WALL PRESSURES

All backfill behind 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 which are free to yield laterally at least 0.1 percent of their height may be designed using an equivalent fluid equal to 35 pcf. Fully restrained, horizontally backfilled rigid walls which cannot yield should be designed for an equivalent fluid of 50 pcf. If parking areas or other areas subject to vehicular traffic are adjacent to walls, a surcharge equivalent to 2 feet of soil should be added to the wall height in determining lateral design forces. Walls with sloping backfill at a maximum angle of 2H:1V should be designed for 52 pcf for yielding conditions, and 75 pcf for restrained conditions.

The lateral pressures presented above are based on the conditions of a uniform backfill consisting of the on-site granular, glacial sediments, or imported sand and gravel 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 wall. Surcharges from adjacent footings, heavy construction equipment, or sloping ground 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 wall. This would involve installation of a minimum 1-foot-wide blanket drain to within 2 feet of the ground surface using imported, washed gravel against the walls placed to be continuous with the perimeter footing drain.

12.1 Passive Resistance and Friction Factors

Lateral loads can be resisted by friction between the foundation and the natural glacial sediments or supporting structural fill soils, or by passive earth pressure acting on the buried portions of the foundations. The foundations must be backfilled with compacted structural fill to achieve the passive resistance provided below. We recommend the following design parameters.

- Passive equivalent fluid = 300 pcf
- Coefficient of friction = 0.35

The above values include a factor of safety of 2.0.

For foundations or retaining walls supported on drilled piers, parameters provided under the "Foundations" section of this report for vertical and lateral pier capacities should be used to design walls for resistance against overturning and lateral transition.

13.0 FLOOR SUPPORT

For those areas where conventional spread footings will be utilized for foundation support, slab-on-grade floors may be constructed either directly on the medium dense to very dense, natural glacial sediments, on recompacted, natural recessional outwash sediments, or on structural fill placed over these materials. Areas of the slab subgrade that are disturbed (loosened) during construction should be compacted to a non-yielding condition prior to placing the pea gravel, as described below.

In order to limit moisture intrusion through the floor slabs, the slabs should be constructed atop a capillary break material, and a vapor barrier. The capillary break should consist of a minimum thickness of 4 inches of washed pea gravel, with a moisture barrier on top of the capillary break. In addition, as per American Concrete Institute (ACI) recommendations, a minimum of 2 inches of clean sand should cover the moisture barrier to protect the integrity of the moisture barrier during concrete placement, and to aid in the curing of the concrete.

In those areas where drilled piers are used for foundation support, two options are available for support of slab-on-grade floors. Option one fully mitigates the potential for total and differential settlement caused by existing, uncontrolled fill soils and debris flow sediments. Option two will provide some partial mitigation, but the owner should expect that cracking and possible differential movement could occur.

Option One: Support of the Floor Atop Drilled Piers or Pipe Piles

Option one would be to support the slab-on-drilled piers or pipe piles. The spacing of the drilled piers or piles would be determined by a structural engineer, based on the amount of reinforcement included in the floor slab design and the amount of acceptable settlement for deflection of the slab.

Option Two: Floating Floors

Another alternative would be to “float” the slab on a structural fill mat. After removing 2 feet of the existing fill soils, and recompacting the exposed soils to a firm and unyielding condition with a minimum 20-ton vibratory roller, a structural fill mat would be placed. After the structural fill placement is completed and approved, the capillary break material, moisture barrier, and sand layer may be placed, and the floor slab cast. The floor slab should not be tied into the building’s foundation, but should be free to settle independently of the footings. Floating floor slabs should contain bar-reinforcement to reduce differential movement across any cracks that might develop.

14.0 DRAINAGE CONSIDERATIONS

Site stratigraphy is relatively variable, but generally consists of layered sediments with significant silt content. We expect that thin, perched, ground water horizons could be encountered during construction, and seepage zones could be encountered in cuts or in undisturbed areas. Therefore, prior to site work and construction, the contractor should be prepared to provide drainage as necessary.

All retaining and footing walls should be provided with a drain at the footing elevation. Drains should consist of rigid, perforated, polyvinyl chloride (PVC) pipe surrounded by washed pea gravel. The level of the perforations in the pipe should be set approximately 2 inches below the bottom of the footing at all locations and the drains should be constructed with sufficient gradient to allow gravity discharge away from the buildings. In addition, all retaining walls should be lined with a minimum 12-inch-thick washed gravel blanket provided over the full height of the wall, and which ties into the footing drain. 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 structures to achieve surface drainage.

15.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

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 final design completion. 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 foundation 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.

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

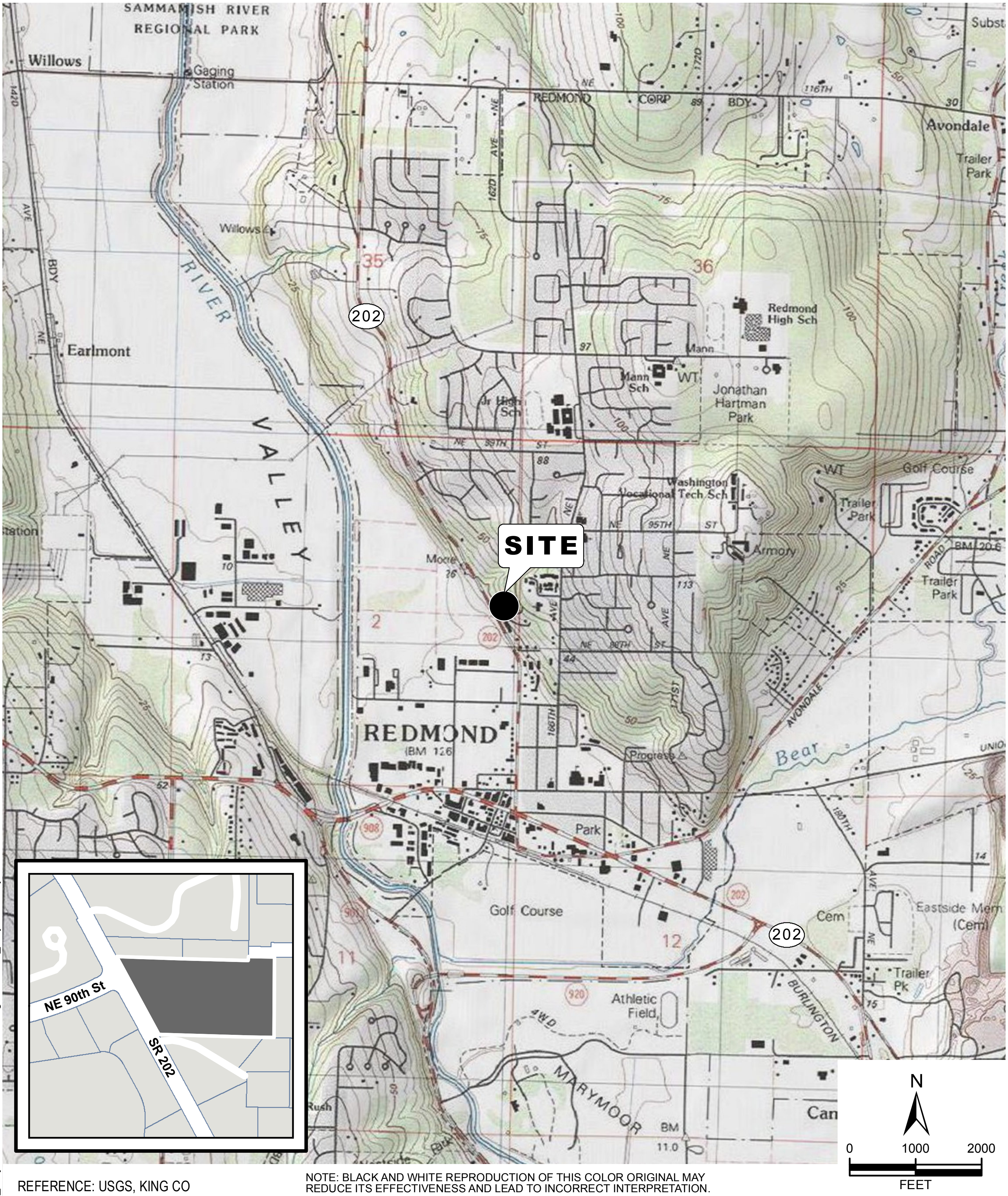
MATTHEW A. MILLER
STATE OF WASHINGTON
30284
REGISTERED
PROFESSIONAL ENGINEER

11-14-14

Danika M. Globokar, E.I.T., G.I.T.
Senior Staff Engineer

Matthew A. Miller, P.E.
Principal Engineer

Attachments:	Figure 1:	Vicinity Map
	Figure 2:	Site and Exploration Plan
	Figure 3:	Geologic Cross-Section A - A'
	Figure 4:	Geologic Cross-Section B - B'
	Appendix:	Exploration Logs



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

RAINSONG DEVELOPMENT PHASE I AND II
REDMOND, WASHINGTON

FIGURE 1

DATE 11/14

PROJ. NO. KE140442A

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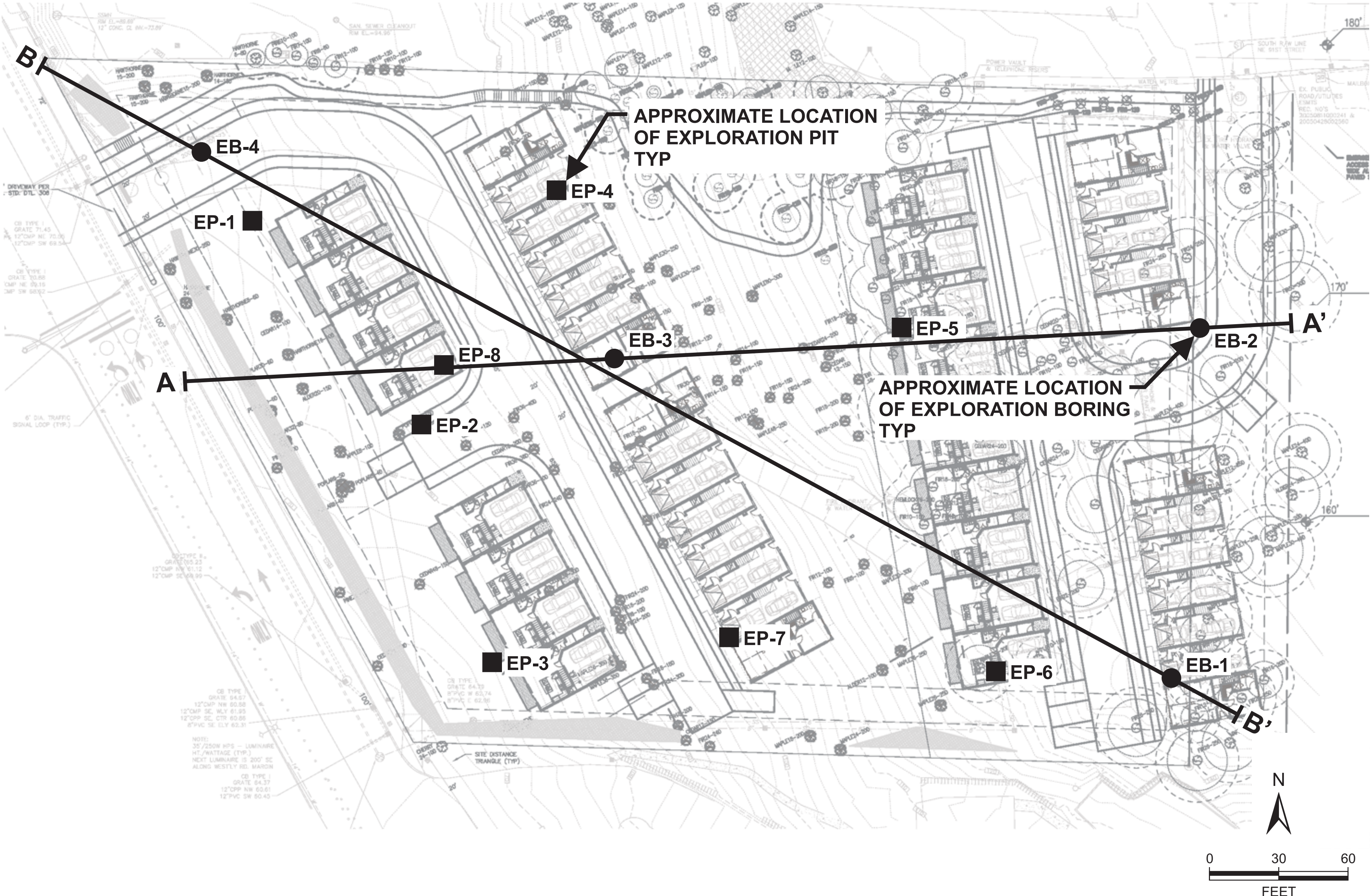

REFERENCE: FREIHEIT & HO ARCHITECTS

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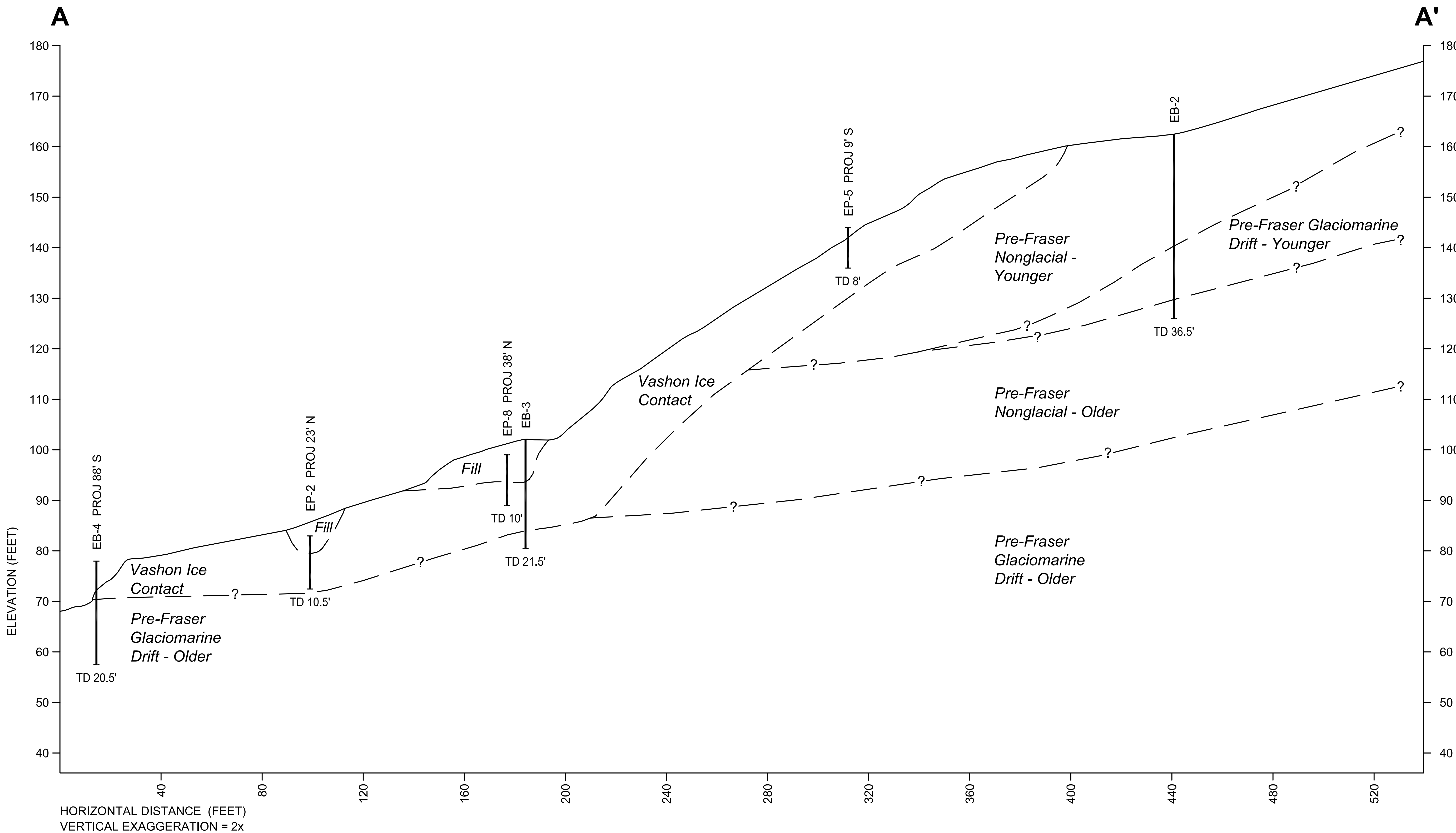
SITE AND EXPLORATION PLAN

RAINSONG DEVELOPMENT PHASE I AND II
REDMOND, WASHINGTON

FIGURE 2
DATE 11/14
PROJ. NO. KE140449A

140449 Rainsong Development Phase I and II\Site and Exploration Plan Phase A.cdr



GEOLOGIC CROSS-SECTION A - A'

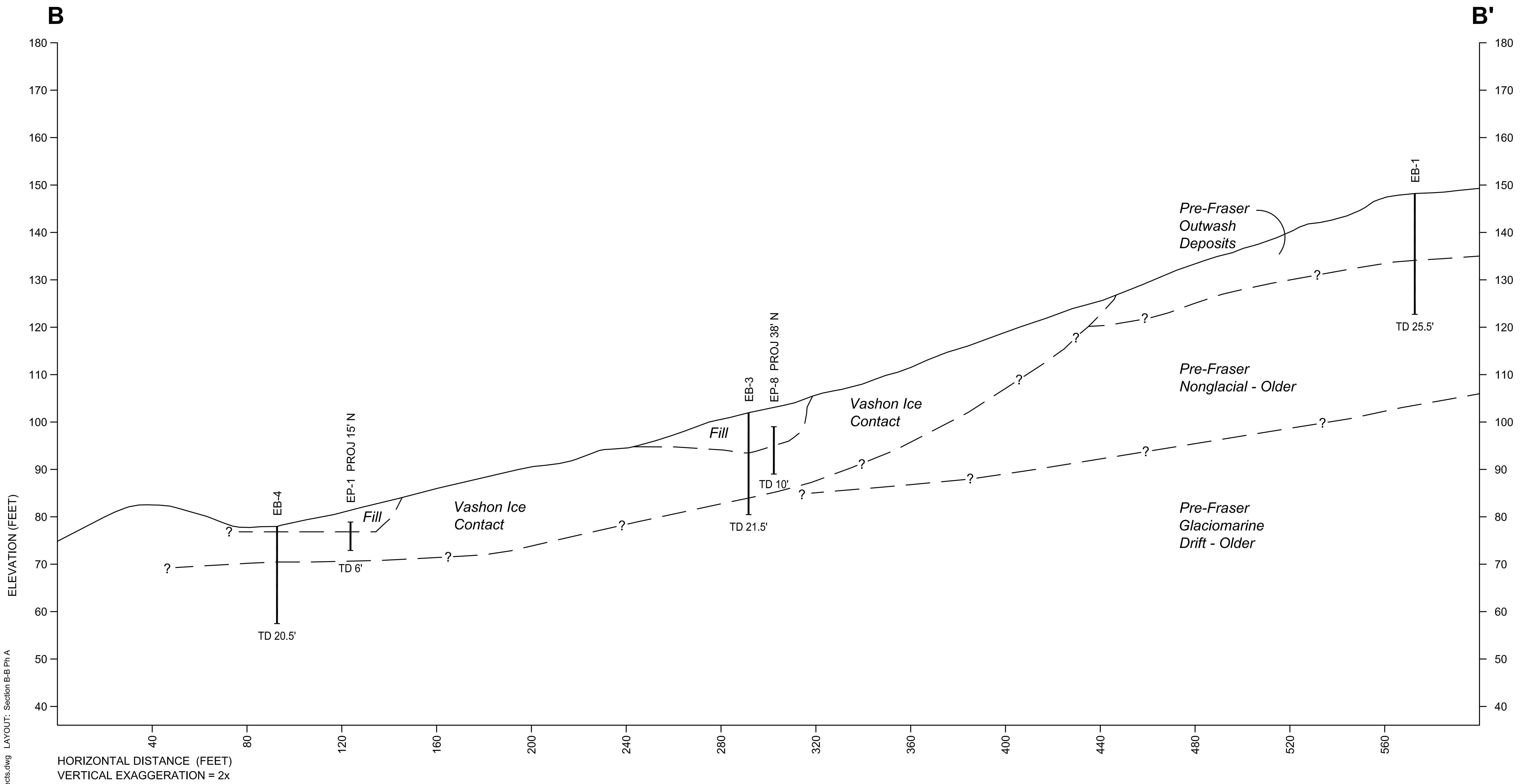
RAINSONG DEVELOPMENT PHASE I AND II
REDMOND, WASHINGTON

FIGURE 3

DATE 11/14

PROJECT NO. KE140449A

140449 Rainsong \ 140449 Geo Sects.dwg LAYOUT: Section A-A Ph A



GEOLOGIC CROSS-SECTION B - B'
RAINSONG DEVELOPMENT PHASE I AND II
REDMOND, WASHINGTON

FIGURE 4
DATE 11/14
PROJECT NO. KE140449A

140449 Rainsong \ 140449 Geo Sects.dwg LAYOUT: Section B-B Ph A

APPENDIX

Group			Symbol	Description
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)

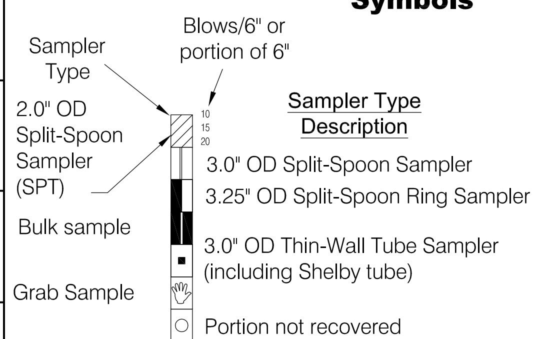
(3) Estimated Percentage

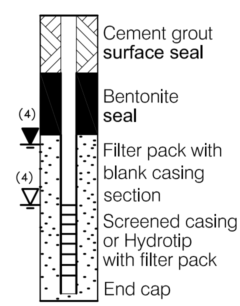
Component	Percentage by Weight
Trace	<5
Some	5 to <12
Modifier (silty, sandy, gravelly)	12 to <30
Very *modifier* (silty, sandy, gravelly)	30 to <50

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.

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EXPLORATION LOG KEY

FIGURE A1

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Exploration Log

Project Number	Exploration Number	Sheet
KE140449A	EB-1	1 of 1

Project Name: Rainsong
Location: Redmond, WA
Driller/Equipment: Geologic Drill, Inc.
Hammer Weight/Drop: 140# / 30"

Ground Surface Elevation (ft): 149
Datum: N/A
Date Start/Finish: 10/30/14,10/30/14
Hole Diameter (in): 6 inches

Depth (ft)	ST	Samples	DESCRIPTION	Blows/6"	Blows/Foot
0			**Asphalt / Crushed Gravel Base Course**		
			Pre-Fraser Outwash Deposits Cuttings: Moist, brown becoming lighter brown with depth, very gravelly fine to medium SAND, some silt (SM-SP).		
5		S-1	Dense, slightly moist, light brown, silty, gravelly fine SAND; stratified (SM).	10 16 20	36
10		S-2	Very dense, slightly moist to moist, light grayish brown, silty, gravelly fine to medium SAND; faintly stratified (SM). Continues to be gravelly / crunchy drilling.	50/6"	50/6"
			Cobbles at 13 feet; hard drilling.		
			Pre-Fraser Nonglacial Deposits		
15		S-3	Very dense, slightly moist to moist, gray with occasional orangish brown mottling, silty fine SAND, some gravel; unsorted (SM). Cuttings become gray, silty fine to medium SAND, some gravel 16 to 17 feet.	50/5.5"	50/5.5"
20		S-4	Very dense, slightly moist to moist, gray with occasional brown mottling, silty, gravelly fine SAND, some medium to coarse sand; unsorted (SM). Driller notes drilling has smoothed out.	50/5"	50/5"
25		S-5	Very dense, moist, gray, silty fine to medium SAND, some gravel; faintly stratified (SM).	33 50/5.5"	83/11.5"
			Bottom of exploration boring at 25.5 feet No ground water encountered.		
30					
35					

Column headings also printed: Graphic Symbol; Well Completion; Water Level; Other Tests. Blows/Foot scale: 10 20 30 40

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: DMG
Approved by:

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Exploration Log

Project Number	Exploration Number	Sheet
KE140449A	EB-2	1 of 1

Project Name	Rainsong	Ground Surface Elevation (ft)	162
Location	Redmond, WA	Datum	N/A
Driller/Equipment	Geologic Drill, Inc.	Date Start/Finish	10/30/14,10/30/14
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	6 inches

Depth (ft)	Samples	DESCRIPTION	Water Level	Blows/6"	Blows/Foot
0		**Forest Duff / Topsoil**			
		Weathered Pre-Fraser Nonglacial Deposits			
		Cuttings: moist, brown, silty, very gravelly fine to medium SAND, occasional organics (Snoqualmie Provenance) (SM).			
5	S-1	Dense, moist, light brown, very sandy GRAVEL, some silt, abundant mica; gravel fractured; stratified (Snoqualmie Provenance) (GM-GP).		22 22 22	44
		Transition from cobbles to gravelly material at 7 feet.			
10	S-2	Very dense, slightly moist, gray, silty, very gravelly SAND, gravel fractured; unsorted (SM).		50/5"	50/5"
		Cuttings: Very moist (perched ground water ?), and gray.			
		Driller notes change in drilling action (almost no gravel) at 13.5 feet			
		Pre-Fraser Nonglacial Deposits			
15	S-3	Dense, moist to very moist, bluish gray with slight green hue, very silty fine SAND, thin bed (~1 inch thick) of silt near sampler tip; stratified (SM).		20 22 24	46
20	S-4	Very dense, very moist, bluish gray, very silty fine SAND, occasional organics, thinly bedded with alternating layers of fine to medium sand and sandy silt (SM).		20 40 50	90
		Cuttings: Wet from seepage of perched zone above; auger / casing not wet from 20 to 35 feet:	▼		
		Pre-Fraser Glaciomarine Deposits			
25	S-5	Hard, slightly moist, gray, fine sandy SILT; laminated; HCl reaction (ML).		24 33 50/5"	83/11"
30	S-6	Hard, slightly moist, gray, SILT, some fine sand, occasional laminations (<1/4 inch thick) of silty sand; fractured texture; laminated; HCl reaction (ML).		10 22 47	69
		Pre-Fraser Nonglacial Deposits			
35	S-7	Hard, moist to very moist, sandy SILT, occasional organics; stratified with beds (1 to 4 inches thick) of silt, sandy silt, very silty fine sand, coarse sand (ML).		15 21 49	70
		Bottom of exploration boring at 36.5 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)

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Approved by:

AESIBOR 140449.GPJ November 13, 2014

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Exploration Log

Project Number	Exploration Number	Sheet
KE140449A	EB-3	1 of 1

Project Name: Rainsong
Location: Redmond, WA
Driller/Equipment: Geologic Drill, Inc.
Hammer Weight/Drop: 140# / 30"

Ground Surface Elevation (ft): 102
Datum: N/A
Date Start/Finish: 10/30/14,10/30/14
Hole Diameter (in): 6 inches

Depth (ft)	Samples	DESCRIPTION	Water Level	Blows/6"	Blows/Foot
0		**Topsoil \ Sod**			
		Fill			
		Drilling relatively smooth.			
5	S-1	Medium dense, very moist to wet, light brown with orange and dark brown mottling, silty, gravelly fine to medium SAND, occasional organics; unsorted (SM).		7 6 12	18
		Some gravel; crunchy drilling.			
		Vashon Ice Contact Deposits			
10	S-2	Hard, wet to very moist, light grayish brown with orange oxidation, very sandy SILT, some gravel (clay coated); unsorted (ML).		11 13 17	30
		Drill action very gravelly / cobbly 12 to 14 feet.			
15	S-3	Hard, wet to very moist, light tan to gray with orange oxidation, fine sandy SILT, trace medium sand, trace gravel (clay coated), thin (~1/2 inch thick) diagonal beds of fine to medium sand; otherwise unsorted (ML).		12 19 17	36
			▼		
		Pre-Fraser Glaciomarine Deposits			
20	S-4	Hard, slightly moist, gray, SILT, some to trace sand, occasional dropstones and laminations of sandy silt; laminated; HCl reaction (ML).		14 21 30	51
		Bottom of exploration boring at 21.5 feet			
25					
30					
35					

Other Tests / Well Completion: (none)

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)

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Approved by:

Exploration Log

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Project Number	Exploration Number	Sheet
KE140449A	EB-4	1 of 1

Project Name	Rainsong	Ground Surface Elevation (ft)	88
Location	Redmond, WA	Datum	N/A
Driller/Equipment	Geologic Drill, Inc.	Date Start/Finish	10/30/14,10/30/14
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	6 inches

Depth (ft)	ST	Samples	DESCRIPTION	Water Level	Blows/6"	Blows/Foot
0			**Gravelly Fill**			
			Vashon Ice Contact Deposits			
5	2" OD Split Spoon	S-1	Dense, wet, brown with abundant orange oxidation, silty, fine to medium sandy GRAVEL; unsorted (GM-GP). Gravelly drilling.		19 21 19	40
			Drilling smoothes out at 7.5 feet.			
			Pre-Fraser Glaciomarine Deposits			
10	2" OD Split Spoon	S-2	Very stiff, very moist, gray, SILT, some to trace sand, dropstones present; laminated; HCl reaction (ML).		7 9 12	21
15	2" OD Split Spoon	S-3	Very stiff, slightly moist to moist, gray, SILT; fractured texture; laminated (ML).	▼ (ATD)	9 9 10	19
20	2" OD Split Spoon	S-4	As above, hard.		19 36 50/5"	86/11"
			Bottom of exploration boring at 20.5 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: DMG
Approved by:

EXPLORATION PIT LOG

Number EP-1

Depth (ft)	Description
0	Loose, moist to wet, dark brown, silty SAND with some gravel; contains substantial roots and scattered wood debris. (Fill)
	Loose, wet, tan, silty SAND with gravel, some cobbles and scattered boulders; becomes medium dense below 3'; contains lumps of silt. (Debris Flow Deposits)
	Stiff, wet, tan, sandy SILT. (Ice Contact Sediments)
5	Dense, wet, tan, silty SAND with gravel and some cobbles. (Ice Contact Sediments)
	BOH @ 6' Note: No seepage; no caving.
10	
15	

Number EP-2

Depth (ft)	Description
0	Topsoil mixed with compost and miscellaneous debris (sticks, concrete, lumber, etc.) (Fill)
5	Medium dense, moist, tan, silty SAND with some gravel and scattered cobbles; contains a bed of stiff, wet, blue-gray with rust mottling, silty CLAY with moderate amounts of organic debris and a sheared appearance at 8' - 9-1/2'. (Debris Flow Deposits)
10	Dense, wet, gray, silty SAND with gravel and scattered cobbles. (Ice Contact Sediments)
	BOH @ 10-1/2' Note: No seepage; no caving.
15	

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Redmond, Washington
Project No. KE98506G
October 1998

EXPLORATION PIT LOG

Number EP-3

0	Topsoil.
	Loose, moist, reddish-brown, silty SAND with some gravel; becomes medium dense and tan below 2'. (Weathered Ice Contact Sediments)
5	Dense to very dense, moist, grayish-tan, silty SAND with some gravel. (Ice Contact Sediments)
	Hard, moist, blue-gray, sandy SILT with some gravel. (Lodgement till)
10	BOH @ 10' Note: No seepage; no caving.
15	

Number EP-4

0	Topsoil.
	Loose, moist, reddish-brown, silty SAND with some gravel. (Weathered Recessional Outwash)
	Medium dense, moist, gray SAND with some gravel, trace silt. (Recessional Outwash)
5	Dense to very dense, moist, light tan, silty SAND with gravel interbedded with silty GRAVEL with sand; becomes grayish-brown with no gravel interbeds below 6-1/2'. (Advance Outwash)
10	BOH @ 8' Note: No seepage; no caving.
15	

Subsurface conditions depicted represent our observation at the time and location of this exploratory hole, modified by geologic interpretation, engineering analysis, and judgment. They are not necessarily representative of other times and locations. We will not accept responsibility for the use or interpretation by others of information presented on this log.

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Project No. KE98506G
October 1998

EXPLORATION PIT LOG

Number EP-5

Depth (ft)	Description
0	Topsoil.
	Loose, moist, reddish-brown, silty SAND with gravel. (Weathered Recessional Outwash)
	Loose to medium dense, moist, gray, gravelly SAND, trace silt. (Recessional Outwash)
5	Dense to very dense, moist, gray to grayish-brown SAND AND GRAVEL with trace silt. (Advance Outwash)
10	BOH @ 8' Note: No seepage; no caving.
15	

Number EP-6

Depth (ft)	Description
0	Topsoil.
	Loose, moist, reddish-brown, silty SAND with gravel. (Weathered Recessional Outwash)
	Medium dense, moist, tan, SAND AND GRAVEL, some silt. (Recessional Outwash)
	Very dense, moist, tan SAND AND GRAVEL, some silt. (Advance Outwash)
5	BOH @ 4' Note: No seepage; no caving.
10	
15	

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October 1998

EXPLORATION PIT LOG

Number EP-7

Depth (ft)	Description
0	Topsoil.
	Loose, moist, reddish-brown, silty SAND with some gravel. (Weathered Recessional Outwash)
5	Loose to medium dense, moist, brownish-gray, gravelly SAND, some silt; stratified. (Recessional Outwash)
	Dense to very dense, moist, grayish-brown, silty SAND with gravel. (Lodgement Till)
	Dense to very dense, moist, grayish-brown, gravelly SAND, trace to some silt. (Advance Outwash)
10	BOH @ 10' Note: No seepage; minor caving 0 to 7-1/2'.
15	

Number EP-8

Depth (ft)	Description
0	Topsoil.
	Loose, moist, reddish-brown, silty SAND with gravel. (Weathered Recessional Outwash)
	Medium dense, moist, grayish-tan to gray, gravelly SAND; contains silt lenses, trace silt. (Recessional Outwash)
5	Very dense, moist, brownish-gray, silty, fine SAND with some gravel; grades to a silty SAND with gravel below 4-1/12'. (Advance Outwash)
10	BOH @ 10' Note: No seepage; no caving.
15	

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