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Subsurface Exploration, Geologic Hazards, and
Geotechnical Engineering Report

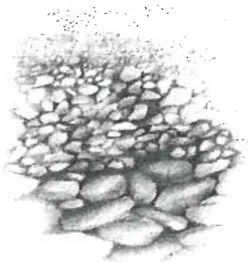
PROPOSED
BOULEVARD PLACE SENIOR HOUSING

Bothell, Washington

Prepared for

Pacific Northern Construction

Project No. KE100378A
November 10, 2014



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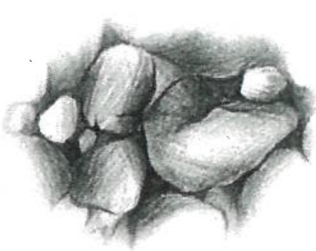

Geologic Assessments

SUBSURFACE EXPLORATION, GEOLOGIC HAZARDS, AND GEOTECHNICAL ENGINEERING REPORT

PROPOSED
BOULEVARD PLACE SENIOR HOUSING

Bothell, Washington

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I. PROJECT AND SITE CONDITIONS

1.0 INTRODUCTION

This report presents the results of our subsurface exploration, geologic hazards, and geotechnical engineering studies for the proposed new Boulevard Place Senior Housing project. The site location is shown on the "Vicinity Map," Figure 1, and approximate locations of the exploration borings completed for this study are shown on the "Site and Exploration Plan," Figure 2. Logs of the subsurface explorations completed for this study and copies of laboratory testing results are included in the Appendix.

1.1 Purpose and Scope

The purpose of this study was to provide geotechnical engineering design recommendations to be utilized in the design of the project. This study included a review of selected available geologic literature, advancing five exploration borings, installing one ground water observation well, and performing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow ground water. Grain-size analysis tests were completed on selected soil samples recovered from our exploration borings, and copies of laboratory test results are included in the Appendix. Geotechnical engineering studies were completed to establish recommendations for the type of suitable foundations and floors, allowable foundation soil bearing pressure, anticipated foundation and floor settlement, pavement recommendations, and drainage considerations. This report summarizes our fieldwork and offers recommendations based on our present understanding of the project. We recommend that we be allowed to review the recommendations presented in this report and revise them, if needed, if project plans change substantially.

1.2 Authorization

Authorization to proceed with this study was granted by Pacific Northern Construction. Our work was completed in general accordance with our scope of work and cost proposal, dated January 7, 2011. This report has been prepared for the exclusive use of Pacific Northern Construction and its 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, express or implied, is made.

2.0 PROJECT AND SITE DESCRIPTION

The site will be developed with a new multifamily senior housing building with commercial space at street level, paved driveway and parking areas, buried utilities, a surface water management system, and other typical improvements. The new building and paving will be constructed close to existing grades on the east side of the site. A cut along the west side of the site will be provided with soldier pile shoring.

The project site consists of an existing Safeway store. The existing store occupies the north central part of the lot, with paved parking and driveways on the east, south, and west. The west edge of the site includes a cast-in-place concrete retaining wall that forms a grade separation of up to approximately 6 feet in height, with higher elevations on the west adjacent to existing homes, and lower elevations on the existing Safeway store site. East of the retaining wall, the site slopes gently down to the southeast, with overall vertical relief of approximately 15 feet. The site is surrounded by existing commercial and residential buildings. We are aware of previous projects in the site vicinity that have encountered compressible soils, and used deep foundations and/or preloading to facilitate construction where compressible soils were an issue.

A Phase II Environmental Site Assessment (ESA) was completed for the project by another consultant. We are aware that the ESA identified ground water contamination that originates off-site and extends below the project. The known contaminant is chlorinated dry cleaning solvent, and the presence of chlorinated solvents in the ground water will be a factor in determining appropriate foundation types and construction methods. We anticipate that any excavations that require temporary or permanent dewatering will encounter ground water that contains chlorinated solvents and will require special handling.

The site does not appear to contain slopes that will be treated as critical areas. The subsurface conditions may be susceptible to liquefaction during a design-level seismic event; however the foundation systems appropriate for this site and recommended in this report would mitigate liquefaction hazards, if they are present.

3.0 SUBSURFACE EXPLORATION

Our subsurface exploration completed for this project included advancing five exploration borings and installing one ground water observation well. The conclusions and recommendations presented in this report are based 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

The exploration borings were completed by advancing hollow-stem auger tools with a trailer-mounted drill rig. During the drilling process, samples were obtained at generally 5-foot-depth intervals. The exploration borings were continuously observed and logged by a representative 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 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 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, visual reconnaissance of the site, and review of selected applicable geologic literature. 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.

4.1 Stratigraphy

Fill

Existing fill was encountered in all but one of our explorations. At the boring locations, the depth of existing fill ranged from approximately 5 to 11 feet. Existing fill was observed to consist of loose to very dense granular sediments, with organic material in two exploration borings, and trace plaster or sheetrock construction waste in one exploration boring. Existing fill is not suitable for structural support. Existing fill should be removed from below planned building areas, and should be re-worked under paving. Excavated existing fill material is not expected to be suitable for reuse in structural fill applications due to high moisture content, organic content, and possible construction waste content. Existing fill is discussed in greater detail in the "Site Preparation" section of this report.

Recessional Outwash Sediments

Below the existing fill, two of our exploration borings encountered variable interbedded sand, sand with silt, and silt. These sediments were typically observed to be loose to medium dense. These native sediments are interpreted to represent recessional outwash sediments. Recessional outwash sediments were deposited from meltwater streams from a receding glacier, and resemble alluvial deposits. The recessional outwash sediments observed in our exploration borings for this project are silty and are considered highly moisture-sensitive. Recessional outwash sediments are not suitable for support of moderately heavy foundation loads expected for the new multistory building, and will require the use of deep foundations, or aggregate piers prior to foundation construction. Excavated recessional outwash sediments are expected to be above optimum moisture content for compaction purposes. If reuse of recessional outwash sediments is explicitly allowed by project plans and specifications, they will need to be dried during favorable dry site and weather conditions to allow their reuse in structural fill applications.

Advance Outwash

Each of the exploration borings encountered dense to very dense granular sediments below surficial fill and recessional outwash sediments. The deeper, dense granular sediments are interpreted to represent advance outwash sediments. Advance outwash was deposited at the base of an advancing glacier, and was subsequently compacted by the weight of the overlying glacial ice. Advance outwash is suitable for support of shallow foundations, deep foundation elements, and paving with proper preparation. Excavated advance outwash sediments from above the ground water level are expected to be suitable for reuse in structural fill applications if specifically allowed by project plans and specifications, and are expected to be moisture-sensitive. It should be noted that due to the depth below existing grade where advance outwash sediments were observed, they will provide direct foundation support for a portion of the project. It is unlikely that excavated advance outwash sediments will be available for reuse in structural fill applications in significant quantity.

Published Geologic Map

We reviewed a published geologic map of the area (*Geologic Map of King County, Washington*, by Derek B. Booth, Kathy A. Troost, and Aaron P. Wisher, 2006). The referenced map indicates that the site is underlain by recessional outwash sediments, with wetland sediments and lodgement till also mapped nearby.

4.2 Hydrology

Ground water was encountered in each of our explorations at the time of drilling. Observed ground water conditions are presented on exploration logs included in the Appendix. We installed one ground water observation well in exploration boring EB-5. Well construction details and observed ground water levels are presented on the exploration log for EB-5. We also noted an existing observation well that had been installed by others prior to our work on-site, near the southeast site corner on the edge of the Bothell Way Northeast right-of-way. Associated Earth Sciences, Inc. (AESI) measured ground water levels in existing wells over an extended period of time. Ground water level measurements are summarized in a graph included in the Appendix.

Ground water conditions should be expected to vary due to changes in season, 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 and surface water conditions, as observed and discussed herein. The discussion will be limited to slope stability, seismic, and erosion issues. The site does not contain subsurface and slope conditions that are likely to trigger City of Bothell steep slope critical areas regulations. The site does not appear to contain soils and slope inclinations that would lead to classification as an erosion hazard area under City of Bothell code. The site may contain subsurface conditions that constitute a seismic hazard area as a result of liquefaction risks, though the foundation support alternatives recommended in this report will mitigate liquefaction risks if properly designed and implemented.

5.0 SLOPE HAZARDS AND MITIGATIONS

The entire site is hard-surfaced and gently sloping with the exception of the west retaining wall that protects a grade separation of up to approximately 6 feet in height. The site does not appear to contain slopes that constitute a slope stability hazard, in our opinion, and does not contain slopes that meet the definition for landslide hazard areas as contained in *Bothell Municipal Code* (BMC) Section 14.04. No quantitative slope stability analysis was completed for this study, and none is warranted for the currently proposed project, in our opinion.

6.0 SEISMIC HAZARDS AND MITIGATIONS

The following discussion is a general assessment of seismic hazards that is intended to be useful to the owner in terms of understanding seismic issues, and to the structural engineer for structural design.

Earthquakes occur regularly in the Puget Lowland. The 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 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

Generally, the largest earthquakes that have occurred in the Puget Sound area are sub-crustal events with epicenters ranging from 50 to 70 kilometers in depth. Earthquakes that are generated at such depths usually do not result in fault rupture at the ground surface. However current research indicates that surficial ground rupture is possible in the South Whidbey Island Fault Zone. The South Whidbey Island Fault Zone is an area of active research. Our current understanding of this fault zone is poor, and actively evolving. The site is located south and west of the currently mapped limits of the South Whidbey Island Fault Zone. Due to the fact that the site lies outside of the currently understood limits of the South Whidbey Island Fault Zone, the risk of damage to the project as a result of surficial ground rupture is low, in our opinion.

6.2 Seismically Induced Landslides

The site does not contain substantial slopes, and does not appear to have significant risk of seismically induced landslides, in our opinion. We did not complete a quantitative slope stability analysis as part of this study, and none is warranted for the currently proposed project, in our opinion.

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 site contains some shallow subsurface soils that are relatively granular, and below the ground water table. These sediments are relatively thin, and are within the zone where ground water is expected to be contaminated with chlorinated solvents. Due the presence of weak soils, and the presence of impacted ground water that would make excavating weak soils difficult, this report recommends use of a deep foundation system. Deep foundation systems are one of the most common measures used to mitigate liquefaction risks. Because the project will use a deep foundation system, any liquefaction risks would be mitigated and therefore a detailed liquefaction analysis was not completed for this study.

6.4 Ground Motion

Structural design of buildings should follow 2012 *International Building Code* (IBC) standards using Site Class "E".

7.0 EROSION HAZARDS AND MITIGATIONS

The BMC Section 14.04 refers to *Washington Administrative Code* (WAC) Section 365-190-120 for definition of erosion hazard areas, which in turn cites United States Department of Agriculture Soil Conservation Service (SCS) map designations. The SCS has mapped the site as EvC, Everett Gravelly Sandy Loam with inclinations of 5 to 15 percent. Because the entire site is gently sloping and hard-surfaced, it appears likely that the site does not meet the applicable definition for an erosion hazard area. The following discussion addresses Washington State Department of Ecology (Ecology) erosion control regulations that will be applicable to the project. In our opinion, implementation of the following recommendations should be adequate to address City of Bothell requirements.

As of October 1, 2008, the 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 significant when the soils are exposed. 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 1st through March 31st), 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.

Some fine-grained surface soils are the result of natural weathering processes that have broken down parent materials into their mineral components. These mineral components can have an inherent electrical charge. Electrically charged mineral fines will attract oppositely charged particles and can combine (flocculate) to form larger particles that will settle out of suspension. The sediments produced during the recent glaciation of Puget Sound are, however, most commonly the suspended soils that are carried by site storm water. The fine-grained fraction of the glacially derived soil is referred to as "rock flour," which is primarily a silt-sized particle with no electrical charge. These particles, once suspended in water, may have settling times in periods of months, not hours.

Therefore, the flow length within a temporary sediment control trap or pond has virtually no effect on the water quality of the discharge, since it will not settle out of suspension in the time it takes to flow from one end of the pond to the other. Reduction of turbidity from a construction site is almost entirely a function of cover measures and flow control. Temporary sediment traps and ponds are necessary to control the release rate of the runoff and to provide a catchment for sand-sized and larger soil particles, but are very ineffective at reducing the turbidity of the runoff.

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 1st and March 31st, 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 [BMPs]) during construction, as recommended by the erosion control inspector, the potential adverse impacts from erosion hazards on the project may be mitigated.

III. DESIGN RECOMMENDATIONS

8.0 INTRODUCTION

Some portions of the site are underlain by a layer of surficial existing fill that is loose and variable. Existing fill is not suitable for support of new foundations, and warrants remedial preparation where it occurs below paving and similar lightly loaded structures. Existing native sediments observed at shallow depths on the south and east parts of the site are also relatively weak, and are not suitable to support foundation loads in their current condition. The surficial fill and weak sediments are underlain at depth by relatively dense native soils that are suitable for foundation support. The depth to suitable support soils ranges from approximately 4.5 to 17 feet below existing grade.

The weak soils on the south and west parts of the site extend below the ground water level. Because of known ground water contamination that originates off-site and extends below the project, we anticipate that it is not feasible to remove the weak soils to allow the use of conventional shallow foundations. The preferred foundation alternative is stone columns. Stone columns could be advanced to varying depths that accommodate variations in subsurface conditions across the site. The resulting building pad will consist of areas where suitable soils are exposed at planned foundation depth, and areas where stone columns of various depths have been installed. A conventional shallow foundation system is then constructed above the finished building pad. Stone columns offer direct foundation support, as well as ground improvement effects for weaker soils that remain between stone columns. At this site, a stone column construction method that does not generate drill cuttings is required.

Project plans make use of 6-inch-diameter pipe piles for foundation support in a limited area on the east part of the site. A soldier pile wall is planned along the west side of the building. Geotechnical engineering recommendations for these structures are presented in this report.

9.0 SITE PREPARATION

Existing buildings, foundations, buried utilities, vegetation, topsoil, and any other deleterious materials should be removed where they are located below planned construction areas. We installed one ground water observation well for this study, and may eventually install additional wells before the design process is complete. Where existing wells are not compatible with future site development plans, they should be decommissioned in accordance with WAC Section 173-160 by a Washington State licensed well driller. 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.

Once demolition has been completed, existing fill should be addressed. The observed fill depth in our exploration borings was up to approximately 12 feet below existing grade. We recommend that where existing fill does not extend to the ground water table, existing fill be removed from below areas of planned foundations to expose underlying, undisturbed native sediments, followed by restoration of the planned foundation grade with structural fill. Removal of existing fill should extend laterally beyond the building footprint by a distance equal to the depth of overexcavation. For example, if existing fill is removed to a depth of 2 feet below a planned footing area, the excavation should also extend laterally 2 feet beyond the building footprint in that area. Where existing fill is removed and replaced with structural fill, conventional shallow foundations may be used for building support. Where existing fill extends below ground water level, we recommend that existing fill be excavated, as needed, to construct a building pad working surface for stone column installation. Subgrade protection is discussed in Section 9.2 of this report.

Below areas of planned flexible paving, it would be possible to leave existing fill in place with some remedial preparation. We recommend that paving areas be stripped of existing topsoil, and proof-rolled and compacted as described later in this report for preparation of paving subgrades. If the resulting surface is firm and unyielding and compacted to 95 percent or more of the modified Proctor maximum dry density, no further preparation is required. If the subgrade is wet or yielding, we recommend that a portion of the existing fill be removed and replaced with material that is capable of being compacted under field conditions that are present at the time the work is completed. Decisions on appropriate preparation procedures should be made in the field at the time of construction when site, soil, and weather conditions are known. A typical scenario might include replacement of the upper 2 feet of existing fill with new structural fill. During wet site or weather conditions, select fill may be needed for this application. A geotextile separation fabric may be required between the prepared subgrade and new compacted structural fill. It should be noted that leaving existing fill in place below planned paving carries some risks of future settlement. Such risks are offset by a substantial saving in initial construction costs. We are available to answer questions regarding cost savings and risks associated with leaving the existing fill in place below planned paving.

9.1 Site Drainage and Surface Water Control

The site should be graded to prevent water from ponding in construction areas and/or flowing into excavations. Exposed grades should be crowned, sloped, and smooth drum-rolled at the end of each day to facilitate drainage. Accumulated water must be removed from subgrades and work areas immediately prior to performing further work in the area. Equipment access may be limited, and the amount of soil rendered unfit for use as structural fill may be greatly increased, if drainage efforts are not accomplished in a timely sequence. If an effective drainage system is not utilized, project delays and increased costs could be incurred due to the greater quantities of wet and unsuitable fill, or poor access and unstable conditions.

We anticipate that ground water could be encountered in excavations completed during construction. We recommend that the project be constructed in such a way that excavations below ground-water level will be minimized. If excavation dewatering becomes necessary, we should be allowed to offer recommendations for collecting and testing water prior to discharge. We should also be allowed to offer situation-specific recommendations if deeper excavations with dewatering systems are considered.

Final exterior grades should promote free and positive drainage away from the building at all times. Water must not be allowed to pond, or to collect adjacent to foundations or within the immediate building area. We recommend that a gradient of at least 3 percent for a minimum distance of 10 feet from the building perimeter be provided, except in paved locations. In paved locations, a minimum gradient of 1 percent should be provided, unless provisions are included for collection and disposal of surface water adjacent to the structure.

9.2 Subgrade Protection

To the extent that it is possible, the existing paving should be used for construction staging. If building construction will proceed during the winter, we recommend the use of a working surface of sand and gravel, crushed rock, or quarry spalls to protect the building pad and any other exposed soils, particularly in areas supporting concentrated equipment traffic. In winter construction staging areas and areas that will be subjected to repeated heavy loads, such as those that occur during construction of masonry walls, a minimum thickness of 12 inches of quarry spalls or 18 inches of pit run sand and gravel is recommended. If subgrade conditions are soft and silty, a geotextile separation fabric, such as Mirafi 500x or approved equivalent, should be used between the subgrade and the new fill. For building pads where floor slabs and foundation construction will be completed in the winter, a similar working surface should be used, composed of at least 6 inches of pit run sand and gravel or crushed rock. Construction of working surfaces from advancing fill pads could be used to avoid directly exposing the subgrade soils to vehicular traffic.

Foundation subgrades may require protection from foot and equipment traffic and ponding of runoff during wet weather conditions. Typically, compacted crushed rock or a lean-mix concrete mat placed over a properly prepared subgrade provides adequate subgrade protection. Foundation concrete should be placed and excavations backfilled as soon as possible to protect the bearing surface.

9.3 Proof-Rolling and Subgrade Compaction

Following the recommended demolition, site stripping, and planned excavation, the stripped subgrade within the building 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 or foundation excavation. 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. Subgrade preparation and selection, placement, and compaction of structural fill should be performed under engineering-controlled conditions in accordance with the project specifications.

9.4 Overexcavation/Stabilization

Construction during extended wet weather periods could create the need to overexcavate exposed soils if they become disturbed and cannot be recompacted due to elevated moisture content and/or weather conditions. Even during dry weather periods, soft/wet soils, which may need to be overexcavated, may be encountered in some portions of the site. If overexcavation is necessary, it should be confirmed through continuous observation and testing by AESI. Soils that have become unstable may require remedial measures in the form of one or more of the following:

1. Drying and recompaction. Selective drying may be accomplished by scarifying or windrowing surficial material during extended periods of dry and warm weather.

2. Removal of affected soils to expose a suitable bearing subgrade and replacement with compacted structural fill.

3. Mechanical stabilization with a coarse crushed aggregate compacted into the subgrade, possibly in conjunction with a geotextile.

4. Soil/cement admixture stabilization.

9.5 Wet Weather Conditions

If construction proceeds during an extended wet weather construction period and the moisture-sensitive site soils become wet, they will become unstable. Therefore, the bids for site grading operations should be based upon the time of year that construction will proceed. It is expected that in wet conditions, additional soils may need to be removed and/or other stabilization methods used, such as a coarse crushed rock working mat to develop a stable condition if silty subgrade soils are disturbed in the presence of excess moisture. The severity of construction disturbance will be dependent, in part, on the precautions that are taken by the contractor to protect the moisture- and disturbance-sensitive site soils. If overexcavation is necessary, it should be confirmed through continuous observation and testing by a representative of our firm.

9.6 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 unsaturated existing fill and unsaturated shallow native sediments can be made at a maximum slope of 1.5H:1V (Horizontal:Vertical) or flatter. Temporary slopes in native soils described in exploration logs as dense to very dense may be planned at 1H:1V. Unshored excavations below the ground water level should not be attempted. 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 vegetation cover can be established during favorable weather.

9.7 Frozen Subgrades

If earthwork takes place during freezing conditions, all exposed subgrades should be allowed to thaw and then be recompacted prior to placing subsequent lifts of structural fill or foundation components. Alternatively, the frozen material could be stripped from the subgrade to reveal unfrozen soil prior to placing subsequent lifts of fill or foundation components. The frozen soil should not be reused as structural fill until allowed to thaw and adjusted to the proper moisture content, which may not be possible during winter months.

10.0 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 90 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. For fill placed below foundation elements designed with an allowable foundation soil bearing pressure higher than 3,000 pounds per square foot (psf), only crushed rock or controlled density fill (CDF) may be used to raise grades. In the case of roadway and utility trench filling, the backfill should be placed and compacted in accordance with current City of Bothell 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. The native and existing fill soils present on-site contained significant amounts of silt and are considered highly moisture-sensitive. On-site soils may only be reused in structural fill applications if specifically allowed by project plans and specifications, and if moisture conditions can be achieved that allow compaction to a firm and unyielding condition and to the specified minimum density for the application where they are used. 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.

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

The subsurface explorations completed for this study encountered highly variable subsurface conditions, some of which are not capable of supporting conventional shallow foundations. We recommend that the site use a conventional shallow foundation system above a system of stone columns. We anticipate that below the west part of the building suitable shallow foundation support soils will be exposed at planned footing grade. Stone columns will be used below foundations on the west part of the site. Pipe piles will be used for foundation support in an isolated area on the east part of the site to reduce the potential for adverse effects on an existing culvert nearby.

11.1 Shallow Foundations

Spread footings may be used for building support when founded directly on a building pad that is prepared by installation of stone columns. We recommend that the design of the stone columns target a design foundation soil bearing pressure of 5,000 psf. Higher foundation soil bearing pressures are possible with stone columns, but are not expected to be needed for this project. We should be allowed to offer additional recommendations if foundation soil bearing pressures higher than 5,000 psf are needed. For those portions of the site underlain by suitable native sediments, stone columns may not be needed. In this case, any fill placed to raise grades below foundations must consist of crushed rock or CDF.

Perimeter footings should be buried at least 18 inches into the surrounding soil for frost protection. However, all footings must penetrate to the prescribed bearing stratum, and no footing should be founded in or above organic or loose soils. All footings should have a minimum width of 18 inches.

It should be noted that the area bound 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 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 ¾ inch or less. 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 to 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.

11.2 Stone Columns

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 provide direct foundation support. 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. At this site, a stone column system that does not generate drill cuttings is required. Material excavated from below the ground water level is expected to contain chlorinated solvents that would require special treatment, and therefore the foundation construction method that is used should not result in bringing significant quantities of soil from below the ground water level to the surface.

The stone columns should be installed after the site is excavated 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 stone columns contractor should review exploration logs contained in this report carefully. Some of our explorations encountered existing fill. Existing fill was not observed to contain substantial demolition waste, stumps, or other deleterious materials that would hinder stone column installation, though such obstacles are always possible. Where drilling obstacles are encountered, the contractor should be prepared to relocate stone columns, or remove obstacles, as needed. It should be noted that Geopiers were used to support a recently completed building on the site adjacent to the north. On that project, which was characterized by subsurface conditions similar to the subject site, pre-drilling of Geopier holes was used to speed up Geopier installation in existing fill. If Geopier Northwest thinks pre-drilling is warranted for this project it should be included in project scheduling and budgeting. The contractor should expect ground water below a depth of approximately 7 feet at the location of EB-5, and as noted on other exploration logs in the Appendix. The contractor should not assume that the site is suitable for use of uncased open holes.

A site-specific stone column design has been prepared for this project by Geopier Northwest. AESI has reviewed the Geopier plan and finds it consistent with our recommendations.

11.3 Baseline Survey

Installation of stone columns will cause vibration that could trigger complaints from adjacent properties. We recommend completion of a detailed photographic survey of adjacent buildings, sidewalks, and paving prior to constructing stone columns. Existing utilities including the Horse Creek culvert should be considered for pre-construction surveying. Particular attention should be paid to documenting any existing cracks prior to stone column construction. The owner and construction team should consider placing crack gauges or other monitoring devices on significant pre-existing cracks. If the owner or construction team feels that vibration-related complaints are likely, additional measures, such as survey monitoring and vibration monitoring, should be considered.

11.4 Pin Piles

Pin piles should be installed by a local contractor with demonstrated expertise in pin pile installations. Pin pile materials and equipment capable of achieving the needed compressive capacity are not standardized. Different contractors will have different pipe materials, different driving equipment, and different driving refusal criteria to meet the design capacity. We should be allowed to review the specific materials and procedures the contractor proposes to use before they mobilize on-site.

In general, pin piles are installed with an air or hydraulic impact hammer until the specified refusal criteria are met. If multiple pipe sections are required, the pipes should be joined with an extension pin inside the pipe, and/or a sleeve on the outside. If uplift loads are expected to be placed on the piles at any time, the connections should also be securely welded. We recommend that at least one pile load test be performed to verify that the design compressive capacity is achieved, and that the load test be observed by AESI. Pile load tests may also be required as a condition of permitting. Piles may be battered up to 15 degrees to develop lateral capacity. Lateral capacity of battered piles may be taken as the horizontal component of the axial pile load 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 pin piles can provide small uplift and lateral capacities, we recommend that these contributions be neglected in designing the new foundation system. The structural engineer should provide pile spacing, locations, splicing details, foundation connection details, and any other structural design recommendations that are needed.

Pin piles are driven until specific refusal criteria are achieved. Pile lengths are difficult to estimate in advance. At this site in addition to achieving the required driving resistance, piles are required to reach a minimum depth. We recommend that piles achieve a minimum penetration depth of 5 feet below the lowest adjacent planned excavation. With proper selection of driving equipment and refusal criteria 6-inch-diameter pipe piles should be able to achieve allowable axial compressive capacities of 20 kips per pile.

We recommend that we be allowed to observe the installation of pin piles. We would observe materials, equipment, and procedures, and confirm refusal for each pile. The purpose of our observations is to confirm that the conditions observed in our explorations and assumed in preparation of our recommendations are consistent with those encountered at the time of construction, and to confirm that the materials, procedures, and refusal criteria are consistent with those we assumed while formulating our recommendations contained in this report.

11.5 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 building. 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 structure to achieve surface drainage. Depending on the locations and final grades that are selected for the building, subfloor drains may be appropriate. As a general guide, building locations that are 5 feet or less above observed ground water locations would warrant subfloor drains. In our experience, subfloor drains are a relatively low cost measure that reduces the potential for expensive, long-term moisture problems. In general, subfloor drains consist of a thickened layer of capillary break material or drainage material of similar gradation that freely communicates with perforated drain lines that are typically on the order of 20 feet on–center.

12.0 FLOOR SUPPORT

Floor slabs can be supported on suitable native sediments, on structural fill placed above suitable native sediments, or on a subgrade improved by stone columns. 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. Areas of subgrade that are disturbed (loosened) during construction should be compacted to a non-yielding condition prior to placement of capillary break material. Floor slabs should also be protected from dampness by an impervious moisture barrier at least 10 mils thick. The moisture barrier should be placed between the capillary break material and the concrete slab.

13.0 FOUNDATION WALLS

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

As required by the 2012 IBC, retaining wall design should include a seismic surcharge pressure in addition to the equivalent fluid pressures presented above. Considering the site soils and the recommended wall backfill materials, we recommend a seismic surcharge pressure of 5H and 10H psf, where H is the wall height in feet for the "active" and "at-rest" loading conditions, respectively. The seismic surcharge should be modeled as a rectangular distribution with the resultant applied at the midpoint of the walls.

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

It is imperative that proper drainage be provided so that hydrostatic pressures do not develop against the walls. This would involve installation of a minimum 1-foot-wide blanket drain to within 1 foot of finish grade for the full wall height using imported, washed gravel against the walls.

13.1 Passive Resistance and Friction Factors

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

- Passive equivalent fluid = 250 pcf
- Coefficient of friction = 0.30

14.0 SOLDIER PILE WALL

A soldier pile wall is planned on the west side of the site. AESI previously provided geotechnical engineering recommendations for the shoring wall in a letter dated June 19, 2014. We have reviewed structural engineering plan sheet S0.1 and verified that the values recommended in our June 19, 2014 letter have been incorporated.

15.0 PAVEMENT RECOMMENDATIONS

Pavement areas should be prepared in accordance with the "Site Preparation" section of this report. If the stripped native soil or existing fill pavement subgrade can be compacted to 95 percent of ASTM:D 1557 and is firm and unyielding, no additional overexcavation is required. Soft or yielding areas should be overexcavated to provide a suitable subgrade and backfilled with structural fill.

The pavement sections included in this report section are for driveway and parking areas on-site, and are not applicable to right-of-way improvements. At this time, this report does not address right-of-way improvements; however, if any new paving of public streets is required, we should be allowed to offer situation-specific recommendations.

The exposed ground should be recompacted to 95 percent of ASTM:D 1557. If required, structural fill may then be placed to achieve desired subbase grades. Upon completion of the recompaction and structural fill, a pavement section consisting of 2½ inches of asphaltic concrete pavement (ACP) underlain by 4 inches of 1¼-inch crushed surfacing base course is the recommended minimum in areas of planned passenger car driving and parking. In heavy traffic areas, a minimum pavement section consisting of 3 inches of ACP underlain by 2 inches of $^5/_8$-inch crushed surfacing top course and 4 inches of 1¼-inch crushed surfacing base course is recommended. The crushed rock courses must be compacted to 95 percent of the maximum density, as determined by ASTM:D 1557. All paving materials should meet gradation criteria contained in the current Washington State Department of Transportation (WSDOT) Standard Specifications.

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

16.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

Our report is based on plans that were current when it was written. We recommend that AESI be allowed to review and revise our recommendation if plans change substantially.

We are also available to provide geotechnical engineering and monitoring 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 our currently approved scope of work.

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.

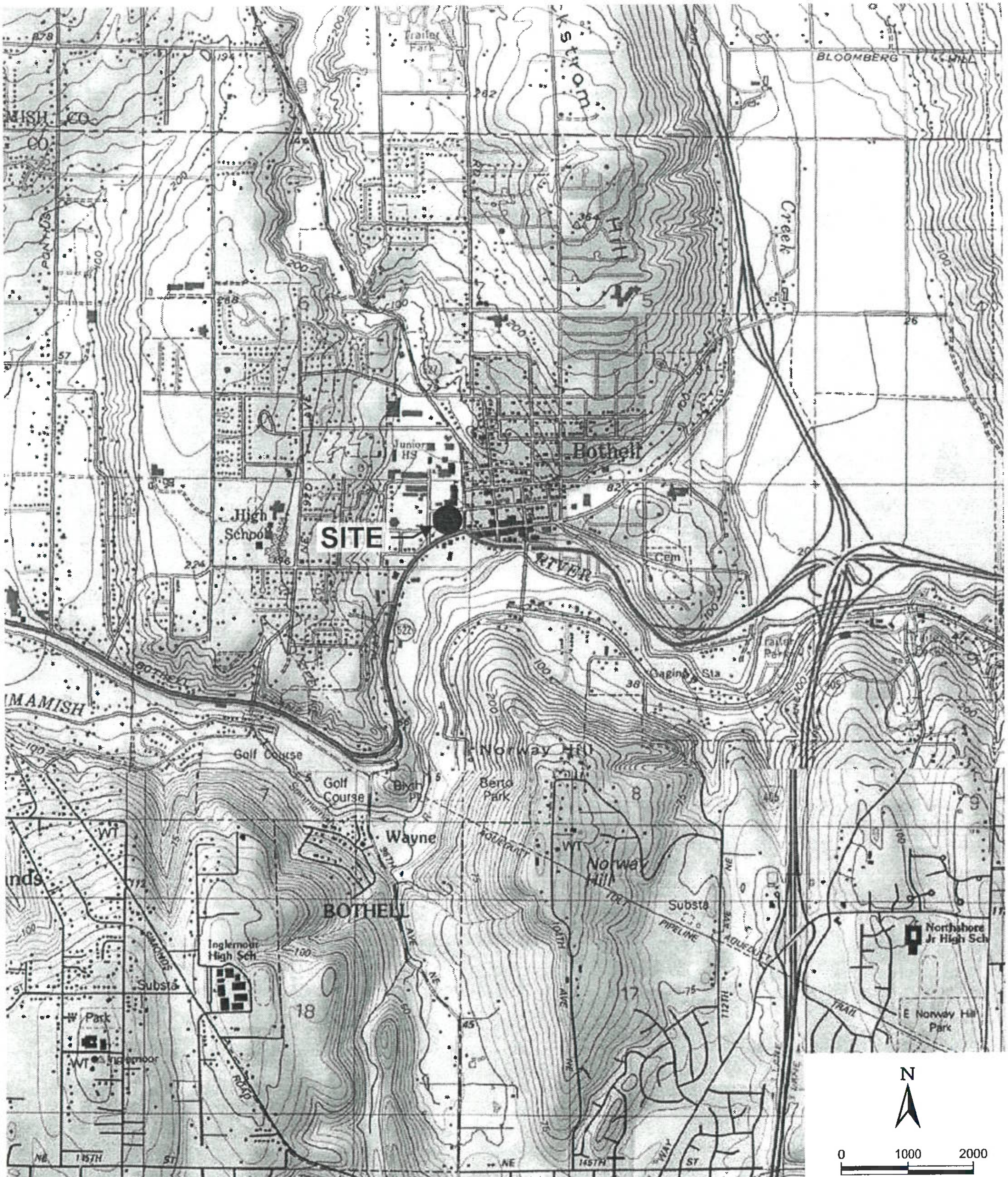
Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

KURT D. MERRIMAN
STATE OF WASHINGTON
23580
REGISTERED
PROFESSIONAL ENGINEER
11/10/14

Bruce W. Guenzler, L.E.G.
Project Geologist

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

Attachments: Figure 1: Vicinity Map
Figure 2: Site and Exploration Plan
Appendix: Exploration Logs
Laboratory Testing Results
Water Levels Graph



REFERENCE: USGS TOPO!

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VICINITY MAP

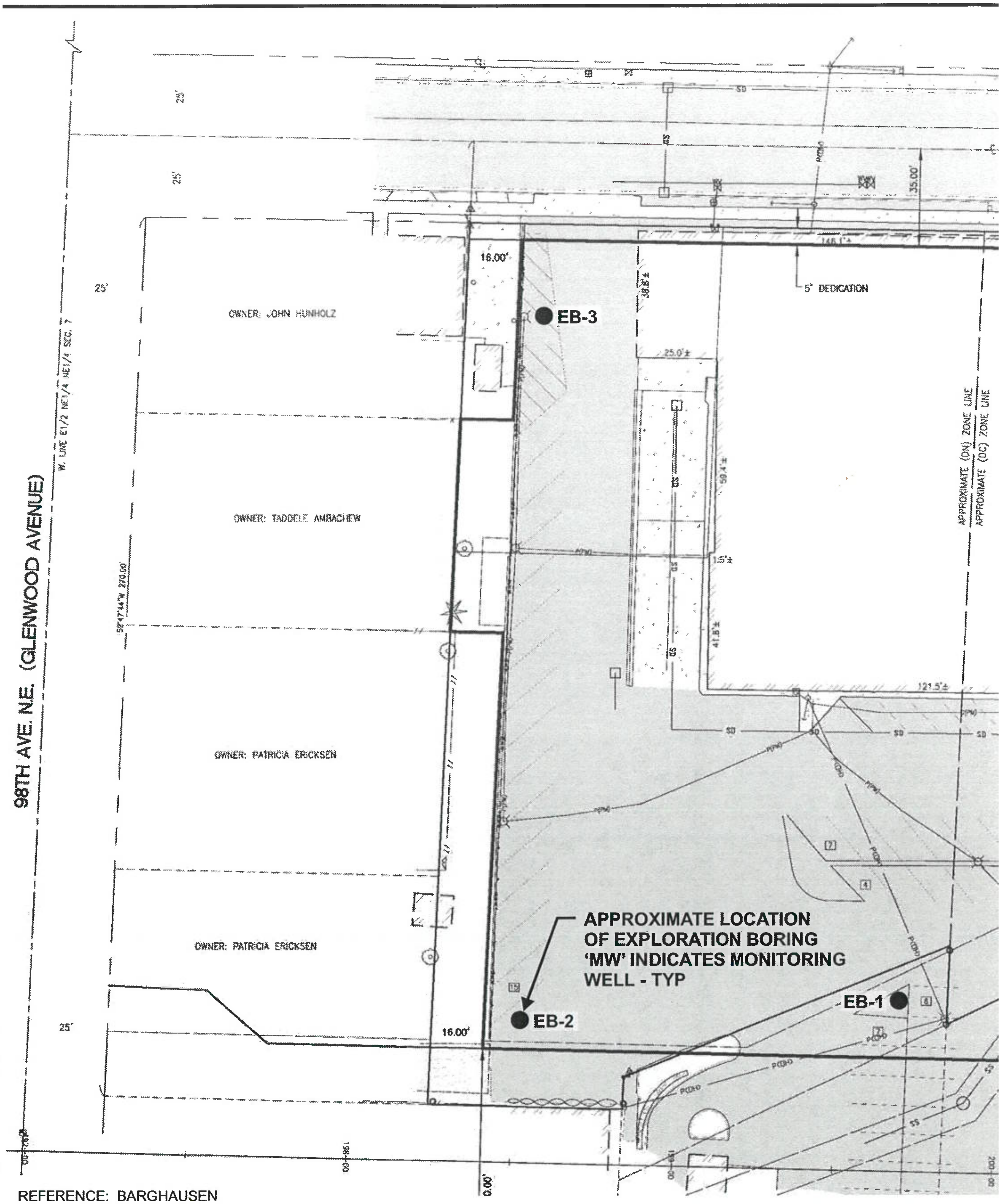
BOTHELL SENIOR HOUSING
BOTHELL, WASHINGTON

FIGURE 1

DATE 12/10

PROJ. NO. KE100378A

100378 Bothell Senior Housing \ 100378 Vicinity.cdr



REFERENCE: BARGHAUSEN

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SITE AND EX

BOTHELL

BOTHEL

100378 Bothell Senior Housing \ 100378 Site and Expl.cdr

APPENDIX

Coarse-Grained Soils - More than 50%[1] Retained on No. 200 Sieve	Gravels - More than 50%[1] of Coarse Fraction Retained on No. 4 Sieve	≤5% Fines[5]		GW	Well-graded gravel and gravel with sand, little to no fines
				GP	Poorly-graded gravel and gravel with sand, little to no fines
		≥15% 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
		≥15% 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)

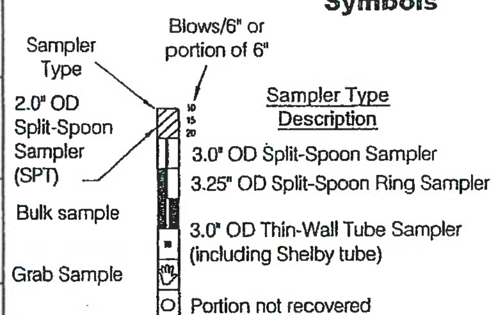
(3) Estimated Percentage

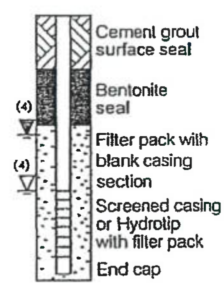
Component	Percentage by Weight
Trace	<5
Few	5 to 10
Little	15 to 25
With	- Non-primary coarse constituents: ≥ 15% - Fines content between 5% and 15%

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 15%

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

blocks\log_key.dwg LAYOUT: Layout2

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EXPLORATION LOG KEY

FIGURE A1

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Exploration Log

Project Number	Exploration Number	Sheet
KE100378A	EB-1	1 of 1

Project Name: Bothell Senior Housing
Location: Bothell, WA
Driller/Equipment: Geologic Drill/XL
Hammer Weight/Drop: 140# / 30"

Ground Surface Elevation (ft): 43 feet
Datum: Barghausen
Date Start/Finish: 12/16/10,12/16/10
Hole Diameter (in): 6 inches

Depth (ft)	Samples	DESCRIPTION	Water Level	Blows/6"	Blows/Foot
		Asphalt - two layers, 1.25 inches and 3.25 inches.			
		Recessional Outwash			
	S-1	6 inches recovery. Medium dense/very stiff, very moist, brown and mottled brown, interbedded, fine SAND, little silt and SILT (SP and ML).		8 8 10	18
5	S-2	8 inches recovery. Loose/stiff, very moist to wet (varies, no water showing on rods), mottled brown, interbedded SILT and fine SAND, few silt (ML and SP).	▽	5 5 5	10
10	S-3	18 inches recovery. Medium dense, very moist to wet (6 inches of water on rods), gray, fine SAND, few to little silt (SP to SM).		6 7 12	19
15	S-4	18 inches recovery. Medium dense, very moist to wet, brownish gray, with strongly oxidized red zone 14.5 to 14.8 feet, fine SAND, with silt and silt stringers, slight gravelly drilling action (SM).		6 11 12	23
		Advance Outwash			
20	S-5	18 inches recovery. Dense, wet, gray, fine SAND, few silt (SP).		12 17 32	49
25	S-6	12 inches recovery. Very dense (blowcount overstated), wet, gray, fine SAND, few silt, with silt stringers (SP with ML).		30 50/5"	50/5"
		Begin drilling with bentonite gel			
30	S-7	Blowcount overstated. Gradation as above, with trace fine gravel.		20 50/6"	70
35	S-8	18 inches recovery. Very dense, wet, gray with red oxidized stringers, fine SAND, few silt (SP).		28 38 40	78
		Bottom of exploration boring at 35 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 (12/22/10 & 1/6/11)
- ▼ Water Level at time of drilling (ATD)

Logged by: BWG
Approved by:

Associated Earth Sciences, Inc.

Exploration Log

Project Number	Exploration Number	Sheet
KE100378A	EB-2	1 of 1

Project Name: Bothell Senior Housing
Location: Bothell, WA
Driller/Equipment: Geologic Drill/XL
Hammer Weight/Drop: 140# / 30"

Ground Surface Elevation (ft): 48 feet
Datum: Barghausen
Date Start/Finish: 12/16/10,12/16/10
Hole Diameter (in): 6 inches

Depth (ft)	Samples	DESCRIPTION	Water Level	Blows/6"	Blows/Foot
		Asphalt - one layer, 2 inches.			
		Fill			
	S-1	14 inches recovery. To 3.5 feet: Medium dense, moist, interbedded dark brown, brown, and black, fine to coarse SAND, with silt and gravel (SW-SM). Below 3.5 feet: Medium dense, moist, yellowish brown, fine to coarse SAND, few silt, little fine gravel (SM).		9 5 7	12
5		**Recessional Outwash**			
	S-2	14 inches recovery. Medium dense, moist, gray, fine to medium SAND, few silt, with silt stringers (SP with ML).		7 7 8	15
10			ATD		
	S-3	18 inches recovery. Medium dense, very moist to wet, gray, fine to medium SAND, few silt, few fine gravel (SW to SP).		10 15 11	26
15		*Begin drilling with bentonite gel*			
	S-4	18 inches recovery. Grades with trace fine gravel, subtle gradational stratification of silt fraction.		7 10 11	21
20		**Advance Outwash**			
	S-5	18 inches recovery. Dense, wet, gray, fine SAND, silt ranges from with to few (varies) (SM). Gradational stratification.		19 20 18	38
25					
	S-6	18 inches recovery. Very dense, wet, gray, fine to medium SAND, few silt (SP). Red oxidized stringers. Gradational stratification of silt fraction.		18 30 30	60
30		Bottom of exploration boring at 29 feet			
35					

Sampler Type (ST):

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

Logged by: BWG
Approved by:

Associated Earth Sciences, Inc.

Exploration Log

Project Number	Exploration Number	Sheet
KE100378A	EB-3	1 of 1

Project Name	Bothell Senior Housing	Ground Surface Elevation (ft)	52 feet
Location	Bothell, WA	Datum	Barghausen
Driller/Equipment	Geologic Drill/XL	Date Start/Finish	12/16/10,12/16/10
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	6 inches

Depth (ft)	Samples	DESCRIPTION	Water Level	Blows/6"	Blows/Foot
		Asphalt - 2.75 inches			
		Fill			
5	S-1	18 inches recovery. Above 3.5 feet: Very dense, moist, brown, fine to coarse SAND, with fine gravel, few silt (SM). Below 3.5 feet: Very dense, moist, gray, fine SAND, with trace silt (SP).		46 35 39	74
		Advance Outwash			
10	S-2	18 inches recovery. Dense, moist, gray and oxidized gray, fine SAND, few silt (SP). Gradational stratification.		10 16 20	36
15	S-3	18 inches recovery. Dense, moist, gray with oxidized stringers, fine to medium SAND, few to trace silt (varies) (SP). Gradational stratification.		17 23 22	45
		Begin drilling with bentonite gel	ATD		
20	S-4	18 inches recovery. As above, except 1 inch thick silt stringer at 18.5 feet.		12 21 25	46
25	S-5	12 inches recovery. Very dense/hard, wet, gray, interbedded fine to medium SAND, few fine gravel and silt, and SILT (SP-SW and ML).		22 50/6"	72
	S-6	12 inches recovery. As above, but silty layers predominant.		28 50/6"	78
30		Bottom of exploration boring at 28.5 feet			
35					

Sampler Type (ST):

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

Logged by: BWG
Approved by:

Associated Earth Sciences, Inc.

Exploration Log

Project Number	Exploration Number	Sheet
KE100378A	EB-4	1 of 1

Project Name: Bothell Senior Housing
Location: Bothell, WA
Driller/Equipment: Geologic Drill/XL
Hammer Weight/Drop: 140# / 30"

Ground Surface Elevation (ft): 43 feet
Datum: Barghausen
Date Start/Finish: 12/17/10,12/17/10
Hole Diameter (in): 6 inches

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
				Asphalt paving 3 inches.					
				Fill					
		S-1		12 inches recovery. Loose, very moist, brown, fine to medium SAND, with few to little fine gravel and silt, trace sheetrock or plaster (SW to SM).			11 5 3	▲8	
5									
		S-2		14 inches recovery. Soft, wet, brown, SILT, with fine organics and small pieces of wood fiber, trace fine gravel (OL).		▼	4 2 1	▲3	
10									
				Advance Outwash					
		S-3		3 inches recovery. High SPT N-value and low recovery; possible obstruction. Very dense, wet, gray, fine SAND, with little silt, trace fine gravel (SM).			22 41 27	▲68	
15				*Begin drilling with bentonite gel*					
		S-4		10 inches recovery. Very dense, wet, gray, fine to coarse SAND, with fine gravel, few to little silt (SW).			13 27 33	▲60	
20									
		S-5		12 inches recovery. Very dense, wet, gray, fine to coarse SAND, few to with fine gravel (varies), few to with silt (varies) (SW to SM). Significant gradational stratification.			31 50/6"	▲81	
25									
		S-6		18 inches recovery. Gradation generally as above, but lower gravel content overall.			28 41 50/6"	▲91	
				Bottom of exploration boring at 29 feet					
30									
35									

Sampler Type (ST):

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

Logged by: BWG
Approved by:

Associated Earth Sciences, Inc.

Geologic & Monitoring Well Construction Log

Project Number	Well Number	Sheet
KE100378A	EB-5	1 of 1

Project Name: Bothell Senior Housing
Elevation (Top of Well Casing): N/A
Water Level Elevation:
Drilling/Equipment: Geologic Drill/XL
Hammer Weight/Drop: 140# / 30"

Location: Bothell, WA
Surface Elevation (ft): 41 feet
Date Start/Finish: 12/17/10,12/17/10
Hole Diameter (in): 6 inches

Depth (ft)	Water Level	WELL CONSTRUCTION	S T	Blows/ 6"	Graphic Symbol	DESCRIPTION
		Flush monument Concrete to surface 1-inch PVC Bentonite chips 4 to 1 1/2 feet Blank riser to surface Colorado #10/20 sand 20 to 4 feet				Asphalt - 2.75 inches. **Fill**
				3 3 3		14 inches recovery. Medium dense, moist, brown, fine SAND, few silt (SP). Thin dark brown stringers.
5	▼					
				3 3 2		Loose, wet, brownish gray, fine SAND, with silt (SM). One dark stringer may have fine organics.
10		0.020-inch slot screen 20 to 5 feet				**Advance Outwash**
				11 13 18		18 inches recovery. Dense, wet, brown, fine to medium SAND, few silt, trace fine gravel (SP). No stratification.
15						*Begin drilling with bentonite gel*
				11 15 44		13 inches recovery. Gradation as above, but gravel fraction is finer - fine gravel to coarse sand.
20		Well ID: BBB 688				
		Note: Well constructed in a separate boring +/- 4 feet away from EB-5		31 27 33		12 inches recovery. Very dense, wet, brown, fine to coarse SAND, little to with fine gravel, few silt (SW).
25						
				50/3" 16 39 50/6"		First sample attempt at 27 1/2 feet, 50/3" bouncing on a rock. Second sample attempt at 28 feet, 16/39/50/6" 89. Textural description: Above 28 1/2 feet, same as Sample 5. Below 28 1/2 feet, very dense, wet, grayish brown, fine SAND, with silt (SM).
						Boring terminated at 29.5 feet on 12/17/10
30						
35						

Sampler Type (ST):
- 2" OD Split Spoon Sampler (SPT)
- 3" OD Split Spoon Sampler (D & M)
- Grab Sample
- No Recovery
- Ring Sample
- Shelby Tube Sample

M - Moisture
▽ Water Level ()
▼ Water Level at time of drilling (ATD)

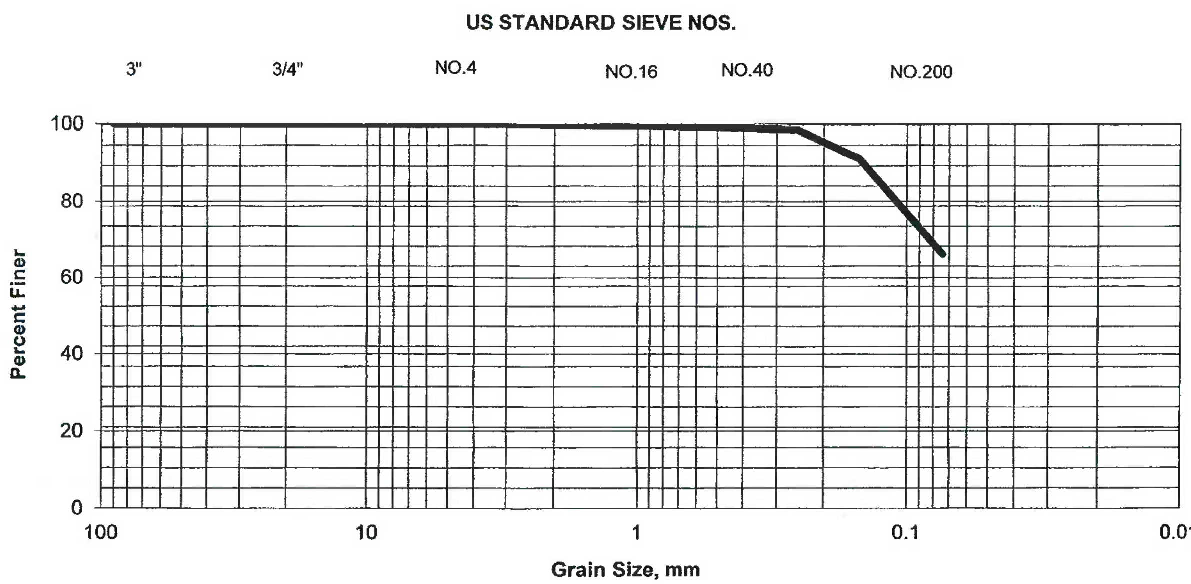
Logged by: BWG
Approved by:

GRAIN SIZE ANALYSIS - MECHANICAL

Date	Project	Project No.		Soil Description
12/17/2010	**Bothell Senior Housing**	KE100378A		
Tested By	Location	EB/EP No	Depth	**Silt With Sand**
MS	**Onsite**	**EB-1 S-2**	**5'**	

Wt. of moisture wet sample + Tar	390	Total Sample Tare	459.52
Wt. of moisture dry Sample + Tare	329.89	Total Sample wt + tare	829.42
Wt. of Tare	98.74	Total Sample Wt	369.9
Wt. of moisture Dry Sample	231.15	Total Sample Dry Wt	293.6
Moisture %	26%		

					Specification Requirements	
Sieve No.	Diam. (mm)	Wt. Retained (g)	% Retained	% Passing	Minimum	Maximum
3.5	90		-	100.00		
3	76.1		-	100.00		
2.5	64		-	100.00		
2	50.8		-	100.00		
1.5	38.1		-	100.00		
1	25.4		-	100.00		
3/4	19		-	100.00		
3/8	9.51		-	100.00		
#4	4.76		-	100.00		
#8	2.38	0.35	0.12	99.88		
#10	2	0.61	0.21	99.79		
#20	0.85	1.72	0.59	99.41		
#40	0.42	3.07	1.05	98.95		
#60	0.25	4.81	1.64	98.36		
#100	0.149	26.16	8.91	91.09		
#200	0.074	99.44	33.87	66.13		



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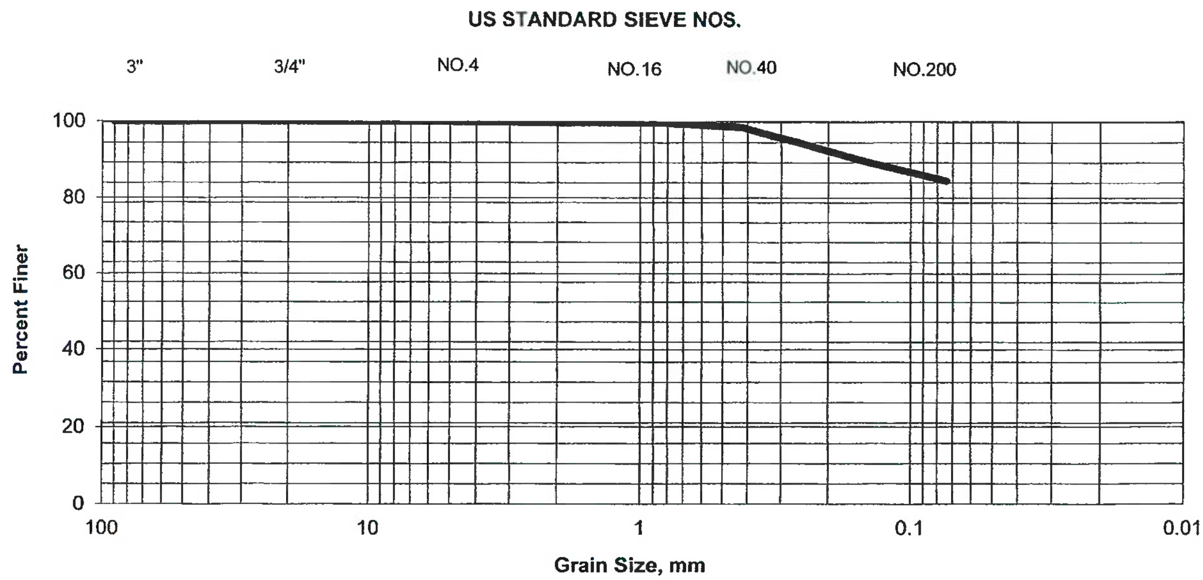
GRAIN SIZE ANALYSIS - MECHANICAL

Date	Project	Project No.		Soil Description
12/17/2010	**Bothell Senior Housing**	KE100378A		
Tested By	Location	EB/EP No	Depth	**Silt few sand**
MS	**Onsite**	**EB-1 S-3**	**8.5'**	

Wt. of moisture wet sample + Tar	327.06	Total Sample Tare	518.71
Wt. of moisture dry Sample + Tare	277	Total Sample wt + tare	1076.14
Wt. of Tare	100.71	Total Sample Wt	557.4
Wt. of moisture Dry Sample	176.29	Total Sample Dry Wt	434.1
Moisture %	28%		

					Specification Requirements	
Sieve No.	Diam. (mm)	Wt. Retained (g)	% Retained	% Passing	Minimum	Maximum
3.5	90		-	100.00		
3	76.1		-	100.00		
2.5	64		-	100.00		
2	50.8		-	100.00		
1.5	38.1		-	100.00		
1	25.4		-	100.00		
3/4	19		-	100.00		
3/8	9.51		-	100.00		
#4	4.76		-	100.00		
#8	2.38	1.44	0.33	99.67		
#10	2	1.55	0.36	99.64		
#20	0.85	1.7	0.39	99.61		
#40	0.42	6.94	1.60	98.40		
#60	0.25	25.02	5.76	94.24		
#100	0.149	44.45	10.24	89.76		
#200	0.074	67.45	15.54	84.46		



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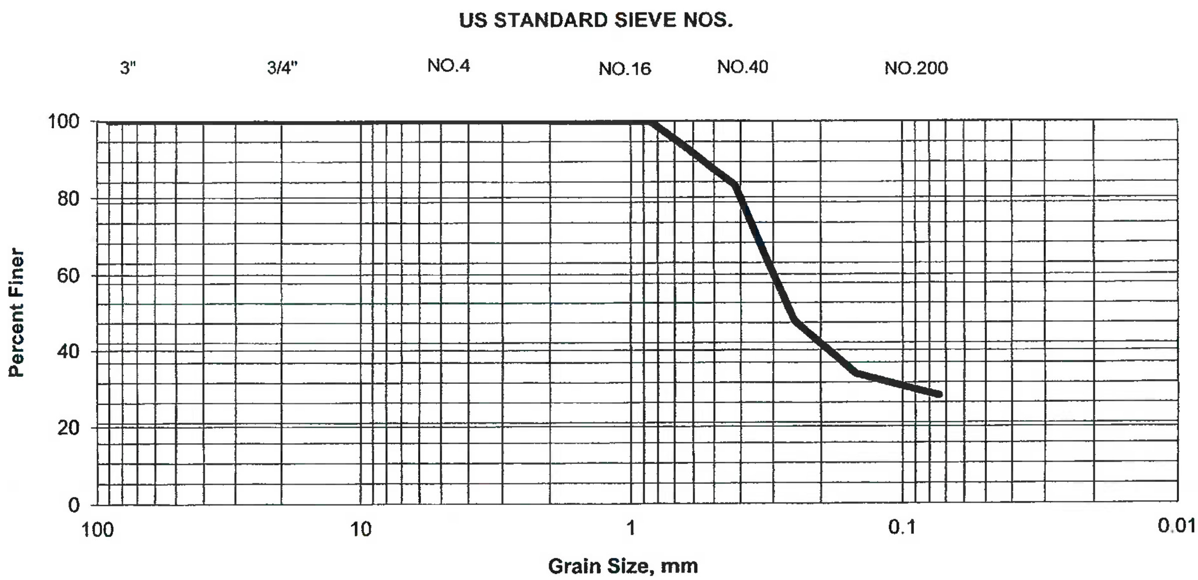
GRAIN SIZE ANALYSIS - MECHANICAL

Date	Project	Project No.		Soil Description
12/17/2010	**Bothell Senior Housing**	KE100378A		
Tested By	Location	EB/EP No	Depth	**Sand little silt**
MS	**Onsite**	**EB-1 S-4**	**13.5'**	

Wt. of moisture wet sample + Tar	437.82	Total Sample Tare	311.66
Wt. of moisture dry Sample + Tare	388.38	Total Sample wt + tare	792.4
Wt. of Tare	101.7	Total Sample Wt	480.7
Wt. of moisture Dry Sample	286.68	Total Sample Dry Wt	410.0
Moisture %	17%		

					Specification Requirements	
Sieve No.	Diam. (mm)	Wt. Retained (g)	% Retained	% Passing	Minimum	Maximum
3.5	90		-	100.00		
3	76.1		-	100.00		
2.5	64		-	100.00		
2	50.8		-	100.00		
1.5	38.1		-	100.00		
1	25.4		-	100.00		
3/4	19		-	100.00		
3/8	9.51		-	100.00		
#4	4.76		-	100.00		
#8	2.38		-	100.00		
#10	2	0.51	0.12	99.88		
#20	0.85	1.19	0.29	99.71		
#40	0.42	69.2	16.88	83.12		
#60	0.25	214.21	52.24	47.76		
#100	0.149	271.52	66.22	33.78		
#200	0.074	294.83	71.90	28.10		



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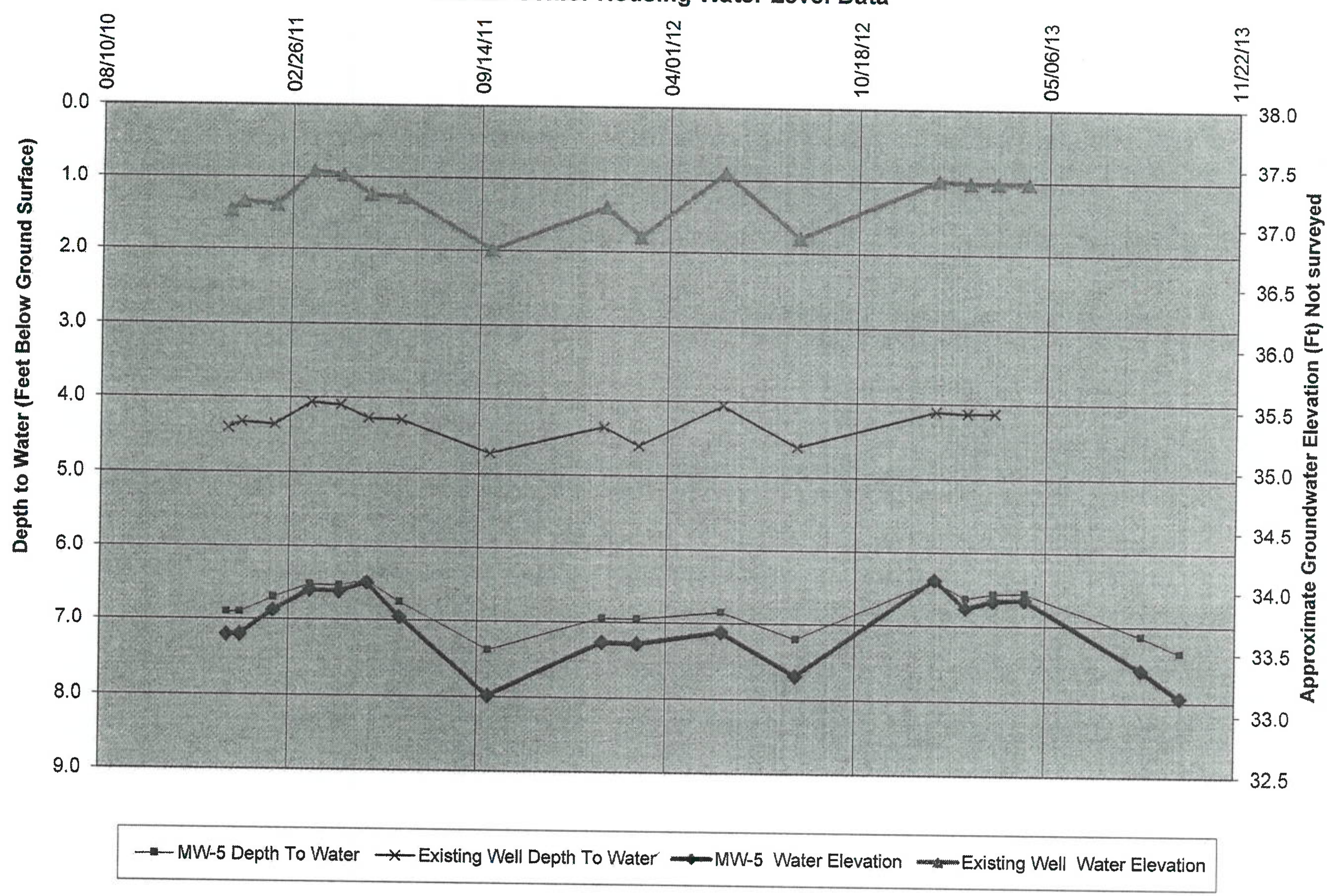
Bothell Senior Housing Water Level Data



11/10/2014 Bothell Senior Housing KE100378A