Appendix III. Geotechnical Report

HWA GEOSCIENCES INC.
Geotechnical & Pavement Engineering • Hydrogeology • Geoenvironmental • Planning & Permitting • Inspection & Testing

August 2, 2011
HWA Project No. 2011-073-21 Task Order No. 1

City of Sammamish
801 228th Avenue SE
Sammamish, Washington 98075

Attention: Anjali Myer

Subject: **PRELIMINARY GEOTECHNICAL EVALUATION – KELLMAN SITE**
Proposed Sammamish Community Center
Sammamish, Washington

Dear Ms. Myer:

As requested, HWA GeoSciences Inc. (HWA) has undertaken a preliminary geotechnical engineering site evaluation for the proposed Sammamish Community Center, located on the Kellman Property, in Sammamish, Washington. Our preliminary investigation for this task consisted of performing a limited site reconnaissance, site explorations, laboratory testing, geotechnical analyses, and preparation of this letter report. The field exploration program consisted of advancing three exploratory borings at selected locations. Soils information obtained from previous engineering reports on nearby facilities, together with our current field explorations and laboratory testing, were used to develop the preliminary recommendations provided herein.

PROJECT UNDERSTANDING

The City of Sammamish (City) is considering construction of a Community Center on the Kellman Property, located near the intersection of SE 8th Street and 228th Avenue NE in Sammamish, Washington (see Figure 1 – Vicinity Map). Based on information provided to us and discussions with the City and the project architect, Barker Rinker Seacat Architecture, we understand the current project concept includes a two to four-story underground parking garage, with additional building structures that will provide space for a gymnasium, a community center area, administration rooms, service areas, and a meeting space, (see Figure 2 – Concept Site Plan). Two pools will also be constructed; one will be a leisure pool and the second will be a lap pool with a diving well. The proposed Community Center is to be located to the west of the recently completed Library, which in turn is located to the south of the City Hall. In general, the complex is arranged so as to conform to the contours of the site which slopes upward to the east.

FIELD EXPLORATIONS

Subgrade soils on the Kellman Property were explored by advancing three exploratory borings on the site, on June 29 and 30, 2011. The boreholes, designated BH-1 through BH-3, were advanced within the limits of the proposed building footprint. The locations of the borings are presented on Figure 3 – Site and Exploration Plan. The depths of the borings ranged from 21.5 to 66.5 feet below ground surface (bgs).

The boreholes were drilled by Holocene Drilling Inc., of Puyallup, under subcontract to HWA, using a truck-mounted Mobile B-65 drill rig. The borings were advanced using hollow-stem auger and employing Non-Standard Penetration Test (N-SPT) sampling methods. The N-SPT sampling was performed using a 3.25 inch outside diameter sampler, with brass rings, which was advanced using a 300 pound automatic-trip hammer. During the test, a sample was obtained by driving the sampler 18 inches into the soil with the hammer free-falling 30 inches. The number of blows required for each 6 inches of sampler penetration was recorded. The N-value (or resistance in terms of blows per foot) is defined as the number of blows recorded to drive the sampler the final 12 inches. This resistance provides an indication of the relative density of granular soils and the relative consistency of cohesive soils. If a total of 50 blows was recorded within a single 6-inch interval, the test was terminated, and the blow count was recorded as 50 blows for the number of inches of penetration achieved.

The undersigned HWA project geotechnical engineer monitored all subsurface explorations. Soil samples obtained from the explorations were classified in the field and representative portions were placed in plastic bags to prevent moisture loss and returned to our laboratory in Bothell, Washington, for further examination and testing.

A Legend of Terms and Symbols Used on Exploration Logs is presented on Figure A-1, Appendix A. Summary soil exploration logs are presented on Figures A-2 though A-4. It should be noted that the stratigraphic contacts shown on the individual exploration logs represent the approximate boundaries between soil types; actual transitions may be more gradual. Moreover, the soil and ground water conditions depicted are only for the specific date and locations reported and, therefore, are not necessarily representative of other locations and times.

LABORATORY TESTING

Laboratory tests were conducted on selected soil samples to characterize relevant engineering properties of the on-site materials. The laboratory testing program was performed in general accordance with appropriate ASTM Standards as outlined below.

- **Moisture Content of Soil:** The moisture content (percent by dry mass) of selected soil samples was determined in accordance with ASTM D 2216. The results are shown at the sampled intervals on the boring logs in Appendix A.
- **Particle Size Analysis of Soils:** Selected samples were tested to determine the particle size distribution of material in accordance with ASTM D 422. The results are summarized on Figures B-1 and B-2, Appendix B, which also provide information regarding the classification of the samples and the moisture content at the time of testing.
- **Liquid Limit, Plastic Limit, and Plasticity Index of Soils (Atterberg Limits):** Selected samples were tested using method ASTM D 4318, multi-point method. The results are reported on the attached Liquid Limit, Plastic Limit, and Plasticity Index reports found on Figure B-3.

SITE CONDITIONS

The site is located to the southwest of the intersection of SE 8th Street and 228th Avenue NE and currently consists of the Sammamish Commons and the Sammamish Library complex. The overall topography is rolling and slopes downward to a large basin located to the west. It appears that the area has been graded in the past to create relatively level building pads for the existing library building and the Sammamish Commons, as well as for the residential structure located in the center of the site. Based on the Concept Site Plan, provided by the City and created by the architect, the elevation difference over the proposed Community Center footprint extends from Elev. 525 to 485 feet, or approximately 40 feet.

The site is accessed from SE 8th Street using the paved access road that passes south of the library and turns to head northward along the west side of the library. Between the library building and the road, a swale has been constructed to hold storm water. We noted that the swale had standing water in it at the time of our explorations. The elevation of the access road where it enters the site is approximately Elev. 525 feet. A gravel parking area has also been constructed west of the access road to hold over-flow parking for the Sammamish Commons.

West of the gravel parking/access road, is a relatively flat area that, according to the City, was used as a staging area during construction of the library building. West of the staging area, the slope increases to about 3H:1V (horizontal:vertical), until it reaches the level of the residential structure, which is at about Elev. 490 feet. West of the house, the site slopes at about 5H:1V. The slope continues down to about Elev. 390 feet where it intersects a stream to the west. A steep slope measured to have a gradient of about 30 degrees borders the site along its south side. This slope is about 15 feet in height.

Except along the southern edge, and where the residential structure with landscaping is located, the site is covered by tall grasses. Standing water in a drainage ditch was noted east of the detached garage that comprises part of the residential development on site. The drainage ditch extended to the south where the upslope end of a culvert was noted near the garage. The upslope end of the culvert is shown on Figure 3. Based on the existing vegetation it appears this area is typically wet most of the year. This vegetation also continues down slope along the south edge of the Kellman Property.

GENERAL GEOLOGICAL CONDITIONS

The geologic maps for the project area (Booth and Minard, 1992 & USGS, 1995) indicate the project site is underlain by Vashon glacial till and Vashon advance outwash deposits. Advance outwash materials are typically sand and gravel deposited by melt water rivers and streams issuing from the advancing ice sheet. Glacial till is a compact, unsorted, mixture of clay, silt, sand, and gravel, and is know to also contain cobbles and boulders. Vashon glacial till was glacially transported and deposited during the last glacial advance. Below the glacial till, we observed glaciolacustrine materials, which were deposited by standing melt water, in a proglacial lake environment. Glaciolacustrine deposits generally consist of silts and clays, with laminations of varying thickness. Given that the glaciolacustrine deposits in the project area were observed to be very stiff to hard in consistency, it appears that these glaciolacustrine materials were deposited prior to the Vashon glaciation of the area. All of the deposits underlying the project site have been over-ridden by up to 3,000 feet of ice and, therefore, have been highly compressed, giving them very high strength suitable for support of foundations. The glacial till and glaciolacustrine deposits are also relatively impermeable, except where sandy and/or highly weathered zones are encountered. Generally, the till and glaciolacustrine deposits form an impervious layer below which surface water cannot penetrate. Where sand overlies the till, ground water is often perched within the sand or weathered soils on top of the dense to very dense till.

SUBSURFACE SOIL CONDITIONS

Our interpretations of subsurface conditions are based on results of our field explorations, review of available geologic and geotechnical data, and our general experience in similar geologic settings. In general, soil conditions throughout the project site consist of imported surficial fill over areas of weathered and non-weathered glacial till, over glaciolacustrine deposits. Each major soil unit is described below, with materials interpreted as being youngest in origin and nearest to the surface described first.

- **Fill** – Fill materials were observed in each of the three borings, and ranged in thickness from 2.5 feet at BH-2 and BH-3 to 7.5 feet in BH-1. The fill consisted of a few inches of topsoil in borings BH-2 and BH-3, grading to light brown to dark brown, sandy silt or silty sand. BH-2 also encountered a gravelly layer. Boring BH-1 encountered 3 inches of hot-mix asphaltic pavement over a few inches of angular gravel base course. A mixture of gravel and cobbles was observed to a depth of about 7.5 feet in BH-1. The gravel and cobbles observed in BH-1 and BH-2 were likely placed as fill during construction of the Sammamish Commons and Library buildings.

- **Disturbed/Weathered Glacial Till** – Disturbed and weathered glacial till was encountered in each of our current borings. This material consisted of medium dense to very dense, slightly gravelly to gravelly, sandy to very sandy, silt. This material extended to depths ranging from 2.5 to 12.5 feet bgs where encountered in our borings. This deposit was the thickest in BH-2, at 10 feet, and the thinnest in BH-3 at 2.5 feet. Weathered glacial till is typically encountered directly above undisturbed glacial till. This derivative of glacial till is a direct result of weathering of the underlying glacial till. Generally, weathered glacial till is looser and more pervious than the underlying glacial till.

- **Glacial Till** – Glacial till was encountered in borings BH-1 and BH-2, but not BH-3. A distinct transition from weathered till to unweathered till was not evident in BH-1; however, the observation of ground water at about 12.5 feet, suggests a more impermeable layer at this depth and extended to about 15 feet bgs. In BH-2, the glacial till was observed at about 10.75 feet bgs and extended to about 17.5 feet bgs. The glacial till was observed to consist of very dense sandy silt with varying amounts of gravel.

- **Glaciolacustrine Deposits** –Glaciolacustrine deposits were observed in each of our three borings. The unit was observed at depths ranging from 5.5 feet bgs at boring BH-3 to 17.5 feet bgs at boring BH-2. All of our borings were terminated in this layer. The glaciolacustrine deposits were observed to transition from typical glacial till to typical glaciolacustrine materials somewhat gradually. The glaciolacustrine materials are characterized as very stiff to hard, gray, lean clay with varying amounts of gravel. Some shear laminations and slickensides were observed in some of our samples, suggesting that the movement of the glacier above applied a drag shearing force on the glaciolacustrine materials below. At greater depths, the samples indicated laminations typical of a quiet glacial lake environment where the water is still enough to allow the sediments to settle out and be deposited at the bottom of the lake. Typically, the laminations can be indicative of seasonal deposition sequencing where the coarser silt bands were deposited more quickly in the summer months and the lighter suspended clay sizes were deposited in the winter months when sediment load to the lake was reduced significantly.

  In BH-3, an approximately 7.5 feet thick zone of permeable silty sand to sandy silt was observed within the glaciolacustrine layer, which indicates a seam of sand and silt was deposited within the glaciolacustrine sequence; possibly in a stream delta setting. The extent of the sand seam is not known: however, it is typical to find these types of permeable zones within glacial till and glaciolacustrine deposits.

GROUND WATER CONDITIONS AND PRELIMINARY HYDROGEOLOGICAL ASSESSMENT

Ground water in the site vicinity occurs in the Vashon till and underlying Vashon advance outwash. Because of the fine-grained matrix of the till, ground water in till is typically concentrated within lenses of coarse-grained material included in the till, and/or ground water perched on top of the dense till surface. In general, advance outwash is considered a major aquifer in east King County, and numerous ground water supply wells utilize this aquifer (USGS, 1995). However, the three borings completed at the site only encountered fill over disturbed and undisturbed till over glaciolacustrine soils. No deposits which could be a significant aquifer were encountered in our limited site exploration program.

At boring BH-1, apparent perched ground water was encountered at the fill-till interface, or within the disturbed till. Standing water was not observed in the borehole or soil samples, but the split-spoon sampler was observed to be wet at 12 to 14 feet bgs. Perched ground water or saturated soil conditions were not observed at a similar elevation at the location of boring BH-2. Previous borings completed approximately 400 to 600 feet east of the site encountered perched ground water within till soils at an elevation of approximately 495 feet (Kleinfelder, 2003). Perched ground water was encountered at a similar elevation in BH-1, indicating that this ground water may occur locally at this elevation, but may not be significant or continuous due to its presence within apparent till soils.

At boring BH-3, located west and down gradient of borings BH-1 and BH-2, ground water was encountered in a sand layer within the glaciolacustrine soils. The sand layer was encountered at approximately 15 feet bgs (Elev. 474 feet). A piezometer was installed in the boring and ground water subsequently rose to 6.5 feet bgs (Elev. 484 feet). This boring was drilled at the location and to the depth of the proposed swimming and diving pool. The sand layer does not appear to be continuous, and likely represents a coarse-grained lens within the glaciolacustrine soils, rather than advance outwash or a separate formation.

Insufficient specific information is available to assess ground water flow direction and gradients in the site area at this time. However, ground water flow at the site is expected to follow surface topography to the west towards drainages and wet areas located west of the site.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Foundations

The current concept plan for the proposed underground parking structure shows the base of the structure to be at Elev. 477 feet, while the lower level of the gymnasium floor is to be at about Elev. 480 feet. Foundations for these components of the complex will be well below the existing site grades, and are likely to be supported by either glacial till or glaciolacustrine deposits, similar to those encountered in our borings BH-1 and BH-2 at these elevations. These materials will provide adequate bearing capacity for shallow foundations such as spread footings, slabs-on-grade, or mat foundations. If properly installed, settlement of shallow foundations in the materials encountered is expected to be negligible. Depending on plasticity conditions in the clay materials, swelling pressures may be a significant design consideration for foundations, floor slabs and walls retaining this material. The single Atterberg Limit test on a sample of this material (see Figure B-3) indicates that the clay soil is a low plasticity material and would not be expected to have a high swelling potential. However, the natural moisture content of the sample was determined to be several percent lower than the plastic limit of the soil, which indicates that the soil is in a moisture deficient state and has the potential to take on moisture with some possible potential to increase in volume. Further testing will be necessary to determine the swelling potential of this material and significance to retaining wall, foundation and floor slab design.

Drilled piers integrated with grade beams may also be a suitable foundation option. Drilled piers or drilled shafts of the type suitable for foundation support for project structural components would have diameters that range between about 16 and 36 inches. They would be constructed by drilling open shafts without casing and installing reinforcing cages and concrete in the open holes. In our experience, they are similar in cost to spread footings and would provide restraint if swelling pressure is an issue for foundation heave.

The cross-sections and concept plan provided to us by the architect indicate that the floor level of portions of the complex will be above existing site grades. Where this occurs, we recommend the footings for the structure in these areas be founded on the dense glacial till materials encountered within approximately 2 to 8 feet bgs. The foundations could then be extended up to the desired floor elevation. During construction the upper topsoil and fill materials should be removed and the dense glacial soils exposed before constructing the footings on the dense glacial materials. Imported structural fill soil would be required to be placed to provide suitable subgrade support for floor slabs on grade. Suitable structural fill would comprise well graded sand and gravel materials compacted in lifts to not less than 95% of the Modified Proctor (ASTM D1557) maximum dry density for the material.

Shoring and Basement Retaining Walls

With a proposed parking garage lower floor elevation of approximately Elev. 477 feet, permanent basement walls up to 50 feet tall will be required to deal with existing site grades. These walls will need to support excavations extending into native very dense, silty sand and hard, lean clay. Based on our evaluations, we conclude that either permanent or temporary shoring up to 50 feet high could be accomplished using a soldier pile and tieback system, or a soil nail system.

A soldier pile and tieback shoring system consists of wide flange beams set into concrete-filled shafts positioned on 6- to 10-foot centers, depending on structural elements selected. Tiebacks are soil anchors extending back from the soldier piles at a low declination from horizontal; typically, about 20 degrees. Timber lagging is used for temporary shoring to span between the soldier piles and support the excavated soils, as well as serve as a back form for the permanent wall. Basement walls are then proportioned and constructed as permanent soil retaining elements, followed by abandonment or removal of the temporary shoring system. Alternatively, precast concrete panels may be substituted for timber lagging for permanent wall systems, and the basement walls designed primarily as a facing element under limited to no soil lateral loading.

Soil nail shoring consists of a series of soil anchors installed on a regular grid pattern, with a shotcrete fascia. Soil nail shoring is completed in top-down steps. The first step is to create a relatively shallow cut, of the order of 3 to 5 feet deep. The next step is to install a series of soil anchors into the face of the cut at a uniform horizontal spacing, typically of the order of 6 to 8 feet. Typically, the soil anchors are installed at a declination of about 20 degrees from horizontal. Next, the face of the cut is covered with drainage mats (geodrains) and a welded-wire reinforcing mesh, which is covered with a shotcrete fascia. The soil anchors are secured to the shotcrete with bearing plates set into the wet shotcrete. When the shotcrete is cured sufficiently the soil nail or anchor is loaded to the level necessary to resist the design soil pressures. The excavation is then taken down another 6 to 8 feet, and the process is continued until the desired depth is achieved. We anticipate that a typical design would entail up to 10 horizontal rows of soil nails.

Due to the density and high clay content of the glacial till, and more so the glaciolacustrine soils, shallow cuts will stand nearly vertical for short durations. These soils are suitable for soil nail wall construction. In our experience, a soil nail and shotcrete shoring system is more economical than a soldier pile, lagging, and tieback shoring system. Moreover, required soil nail lengths will be typically shorter than larger higher-capacity tiebacks, which will reduce soil anchor encroachment onto adjacent properties. For these reasons, we consider soil nail and shotcrete shoring appropriate for this project, where deeper excavations are required and sloping, or long anchors, are not an option due to space limitations. However, care will have to be exercised in shoring of the upper fill and reworked native soils, as some sloughing of loose materials may occur.

For planning purposes, soil nail lengths of the order of 75% of the retained height of the basement should be considered. Thus, for the 50-foot deep basement excavation along the southern side of the proposed garage, 38-foot long soil nails should be planned for. We understand that the City prefers to keep all permanent ground anchors on the City's property and not encroach onto the adjacent private properties. This would require the permanent shoring wall be kept at least 40 feet north of the property line.

The shotcrete-covered soil nail shoring walls can be designed as either temporary shoring, or can be incorporated into the design of the permanent basement walls. In either case, we envision the permanent basement retaining walls would consist of cast-in-place, reinforced, concrete. In either scenario, a good drainage system must be installed between the soil and the shotcrete and the permanent basement walls.

Pool Foundation and Retaining Walls

The proposed elevation of the pool deck is Elev. 490 feet, which is near the existing ground surface. The base of the diving well is planned to be at Elev. 477 feet. Based on our explorations at BH-3, the foundation for the pool will be supported on glacial deposits with adequate bearing capacity. However, shallow ground water was observed at Elev. 485 feet at the time of drilling in late June and may be higher during the wet season. Therefore, design of the pool should include a system to temporarily dewater the area around the pool when it is emptied for cleaning and maintenance. Alternatively, the design will need to compensate for buoyant loading effects.

Temporary shoring may also be needed around the pool structure; here soil nails are not considered appropriate. Soldier piles would be more appropriate in this area. The shafts of the soldier piles would likely extend below the ground water table. Temporary casing will likely be necessary to facilitate installation of the soldier piles, and dewatering will be necessary to facilitate construction of the pools in relatively dry conditions.

Dewatering

Dewatering during construction is expected to be required. In the parking garage area, the dewatering is likely to take place within the excavation, as no continuous water bearing strata were observed that would be amenable to the use of dewatering well systems. At the pool location, construction may require the use of dewatering wells around the excavation, or at least on the uphill side to lower the ground water table during construction of the pool. At the pool, no settlement issues are expected by dewatering the area around the pool.

Earthwork

Glacial materials, and particularly the clayey glaciolacustrine deposit, being highly consolidated and hard, will likely be difficult to excavate using standard excavation equipment, and may require ripping. Cobbles and gravel were observed in the glacial till encountered in our explorations and it is likely boulders could also be encountered during excavation. The contractor should be prepared to deal with these materials.

Wet Weather Earthwork

Existing site soils are highly moisture sensitive (i.e. softening and losing strength on wetting) and will prove to be difficult to handle or traverse with construction equipment during periods of wet weather. Therefore, limiting wet weather earthwork is recommended for this project. Excavations and shoring construction should be completed in the late summer months to minimize ground water seepage and precipitation runoff. Moreover, water will have a substantial destabilizing effect on silty glacial till and glaciolacustrine soils and should not be permitted to accumulate in the basement area and foundation excavations. Thin concrete mud slabs may be utilized to protect exposed soil bearing surfaces after they are prepared and approved for support.

Utilities

During our site exploration, we observed that several utilities extend north and south along the western edge of the access road. Thus, design should provide for the relocation of several utilities. If desired, the location of the structure could be selected to limit or minimize the number of utilities being impacted.

Temporary Excavations

Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. In accordance with Part N of Washington Administrative Code (WAC) 296-155, latest revisions, all temporary cuts in excess of 4 feet in height must be either sloped or shored prior to entry by personnel. The existing fill and weathered glacial till are generally classified as Type C soil, per WAC 296-155. Where shoring is not used, temporary cuts in Type C should be sloped no steeper than 1½H:1V (horizontal:vertical). The existing non-weathered glacial till and glaciolacustrine soils classify as Type A Soil, and temporary unsupported cut slopes in these materials should be inclined no steeper than 3/4H:1V (horizontal:vertical). Composite slopes are permissible, where material types vary with depth.

These recommendations are applicable to excavations above the water table only; flatter side slopes and/or shoring will be required if significant ground water seepage is encountered. Temporary slopes should be protected from erosion, as necessary, by covering the cut face with well-anchored plastic sheets. Heavy construction equipment, construction materials, excavated soil, and vehicular traffic should not be allowed any nearer the cut slope crest than half the height of slope, measured from the edge of the excavation, unless there is a shoring system in place that has been designed for support of the additional lateral pressure. Exposure of personnel beneath temporary cut slopes should be kept to a minimum.

CONDITIONS AND LIMITATIONS

We prepared this preliminary geotechnical report for the City of Sammamish for use in preliminary evaluation of this site for the intended purpose. This report is not a detailed geotechnical engineering design report; and geotechnical engineering evaluations were not conducted as part of this work.

Our work scope did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or ground water at this site.

Experience has shown that soil and ground water conditions can vary significantly over small distances. Inconsistent conditions can occur between exploration locations and may not be detected by a preliminary geotechnical evaluation of this nature. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, HWA should be notified for review of the recommendations of this report, and revision of such if necessary.

Within the limitations of scope, schedule and budget, HWA attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology in the area at the time the report was prepared. No warranty, express or implied, is made.

We trust that the foregoing meets with your present requirements for a preliminary geotechnical engineering evaluation of the proposed project site. However, if any questions arise, or if we may be required to provide for more detailed geotechnical engineering design, please contact our office at your convenience.

Thank you for this opportunity to have been of service.

Sincerely,

HWA GEOSCIENCES INC.

JOANNA LYN GILLIE
45832

Gillie

JoLyn Gillie, P.E.
Geotechnical Engineer

Lorne Balanko, P.E.
Principal Geotechnical Engineer

ATTACHMENTS

Figure 1	Vicinity Map
Figure 2	Concept Site Plan
Figure 3	Site and Exploration Plan
Appendix A	Field Explorations
Appendix B	Laboratory Testing

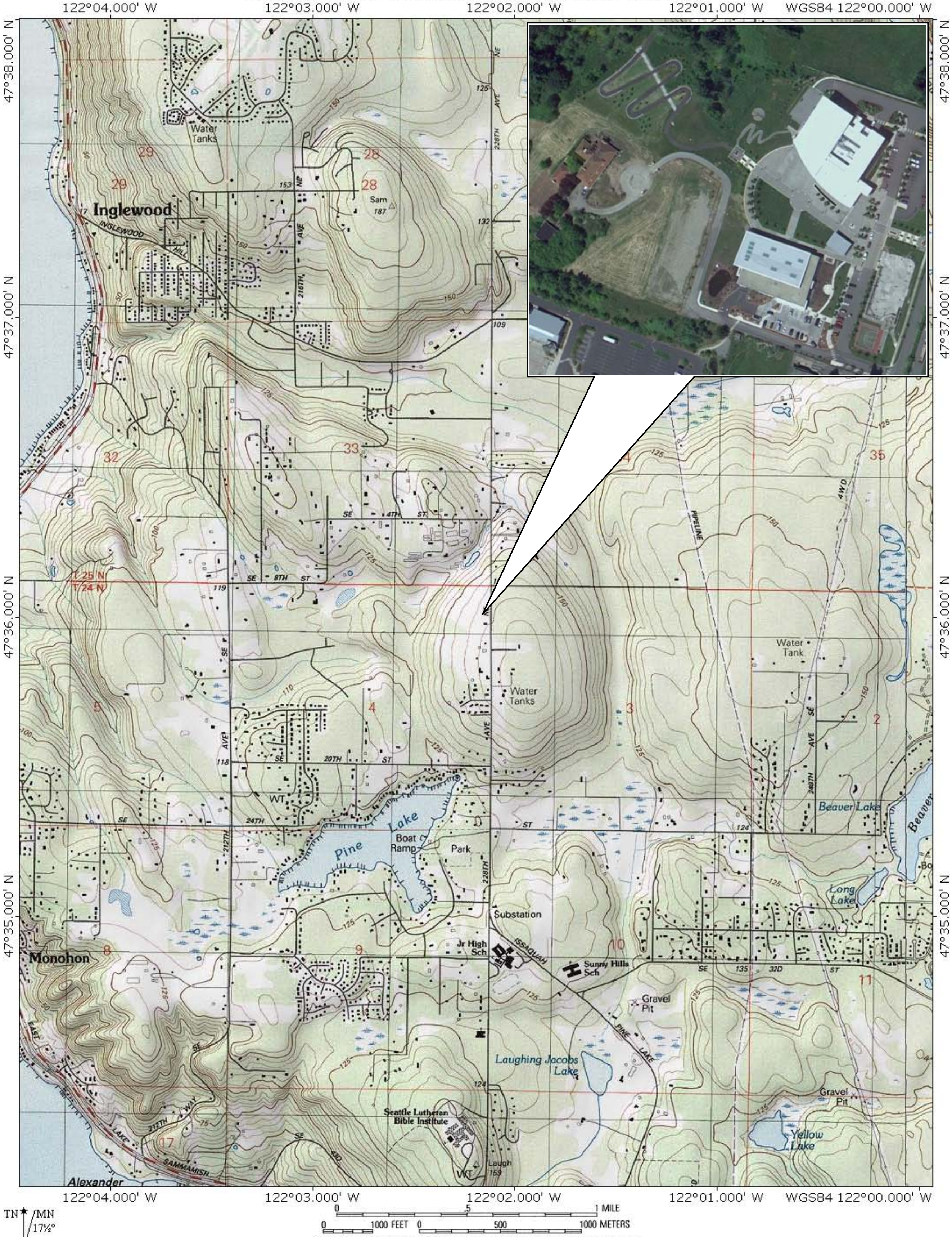
REFERENCES:

Booth, Derek B., Minard, James P., *"Geologic Map of the Issaquah 7.5' Quadrangle, King County, Washington"*, U.S. Department of the Interior, 1992

Kleinfelder Inc., 2003, *Geotechnical Engineering Study, Sammamish Commons, Sammamish, Washington,* submitted June 26, 2003, Kleinfelder Project No.: 30327

U.S. Geological Survey, 1995, *Geohydrology and Ground-Water Quality of East King County, Washington,* Water-Resources Investigations Report 94-4082.

TOPO! map printed on 12/09/10 from "Washington.tpo" and "Untitled.tpg"



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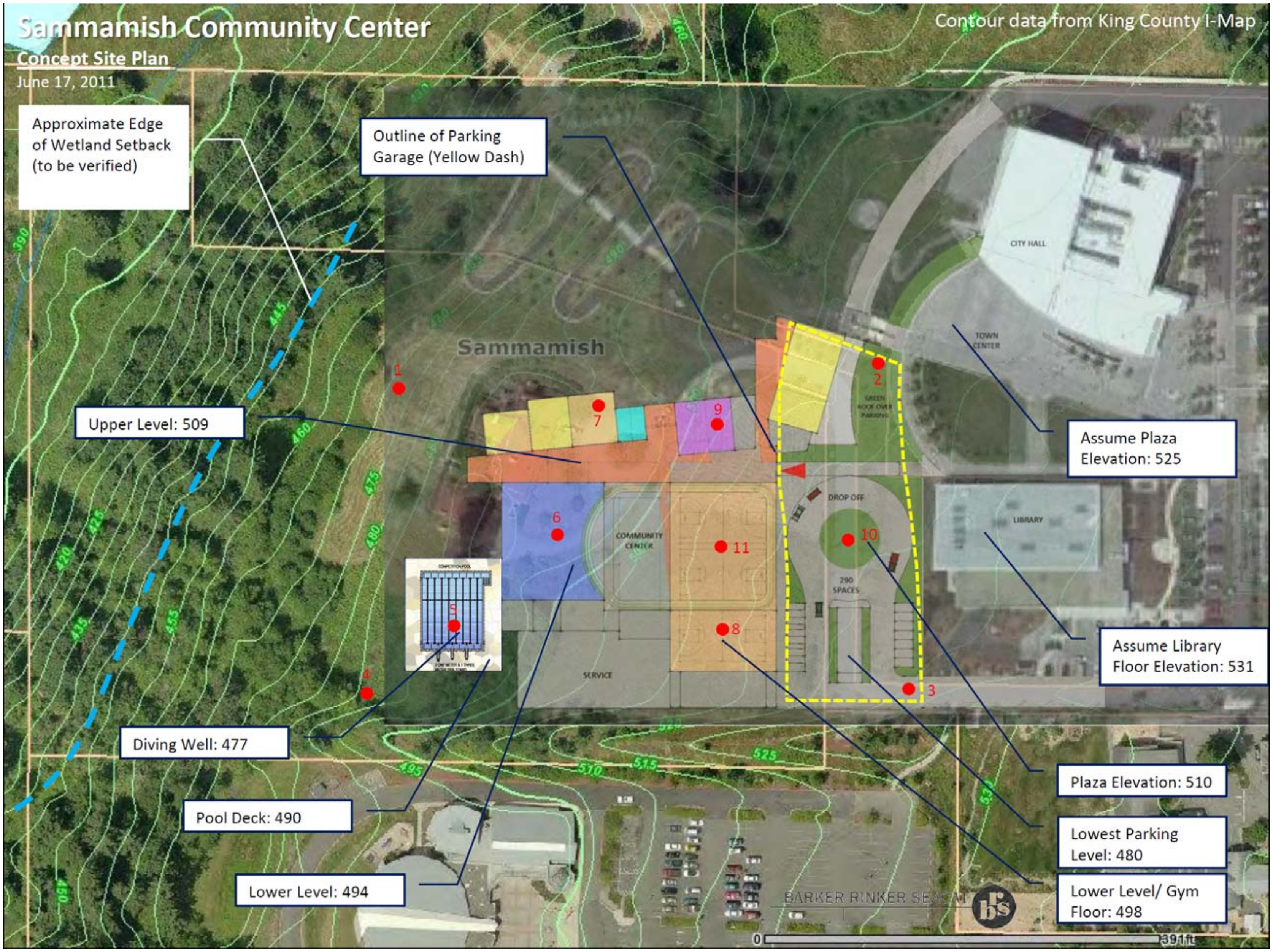
HWA GEOSCIENCES INC.

VICINITY MAP

SAMMAMISH COMMUNITY CENTER
PRELIMINARY GEOTECHNICAL SITE EVALUATION
FOR THE KELLMAN SITE
SAMMAMISH, WASHINGTON

FIGURE NO. **1**

PROJECT NO.
2011-073-21
T100



Concept plan developed by Barker Rinker Seacat Architecture and provided to HWA by City of Sammamish on June 21, 2011

HWA GEOSCIENCES INC.

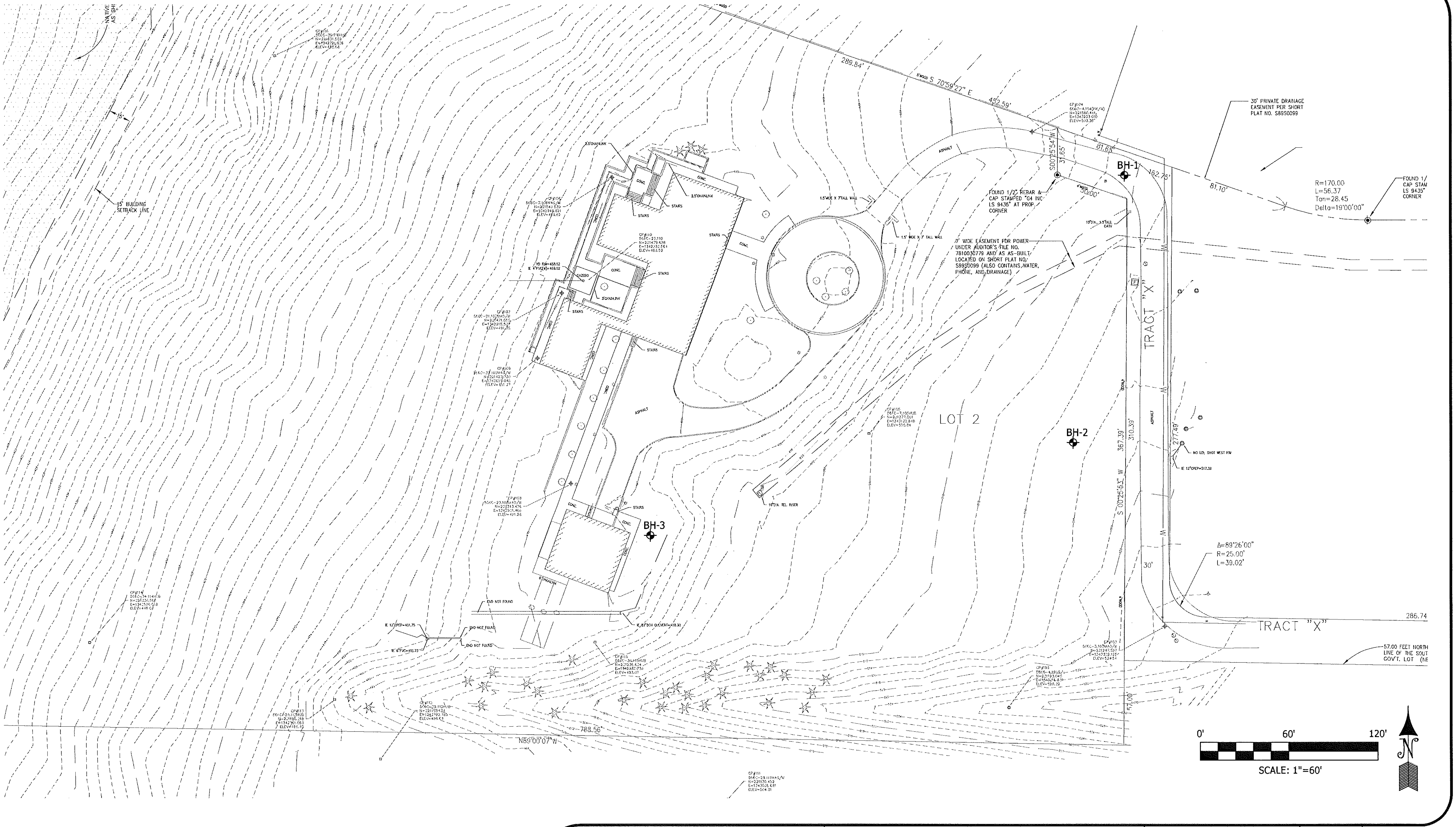
July 8, 2011

CONCEPT SITE PLAN

SAMMAMISH COMMUNITY CENTER
PRELIMINARY GEOTECHNICAL SITE EVALUATION
FOR THE KELLMAN SITE
SAMMAMISH, WASHINGTON

FIGURE NO.
2

PROJECT NO.
2011-073-21
T100



LEGEND

BH-1

⌖ BOREHOLE DESIGNATION AND APPROXIMATE LOCATION

BASE MAP PROVIDED BY CITY OF SAMMAMISH
SITE SURVEY BY PACE ENGINEERING

HWA GEOSCIENCES INC.

SAMMAMISH COMMUNITY CENTER
PRELIMINARY GEOTECHNICAL SITE
EVALUATION FOR THE KELLMAN SITE
SAMMAMISH, WASHINGTON

SITE AND
EXPLORATION
PLAN

DRAWN BY	EFK
CHECK BY	JG
DATE	07.6.11
FIGURE NO.	3
PROJECT NO.	2011-073-21 Task 0100

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APPENDIX A

FIELD EXPLORATION

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
Density	N (blows/ft)	Approximate Relative Density(%)	Consistency	N (blows/ft)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	0 - 15	Very Soft	0 to 2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 - 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	>4000

USCS SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP DESCRIPTIONS		
Coarse Grained Soils More than 50% Retained on No. 200 Sieve Size	Gravel and Gravelly Soils More than 50% of Coarse Fraction Retained on No. 4 Sieve	Clean Gravel (little or no fines)	●	GW	Well-graded GRAVEL
			○	GP	Poorly-graded GRAVEL
		Gravel with Fines (appreciable amount of fines)	○	GM	Silty GRAVEL
			▨	GC	Clayey GRAVEL
	Sand and Sandy Soils 50% or More of Coarse Fraction Passing No. 4 Sieve	Clean Sand (little or no fines)	⁘	SW	Well-graded SAND
			⁘	SP	Poorly-graded SAND
		Sand with Fines (appreciable amount of fines)	▥	SM	Silty SAND
			▨	SC	Clayey SAND
Fine Grained Soils 50% or More Passing No. 200 Sieve Size	Silt and Clay	Liquid Limit Less than 50%	▥	ML	SILT
			▨	CL	Lean CLAY
			▤	OL	Organic SILT/Organic CLAY
	Silt and Clay	Liquid Limit 50% or More	▥	MH	Elastic SILT
			▨	CH	Fat CLAY
			≋	OH	Organic SILT/Organic CLAY
Highly Organic Soils			⁂	PT	PEAT

TEST SYMBOLS

%F	Percent Fines
AL	Atterberg Limits: PL = Plastic Limit LL = Liquid Limit
CBR	California Bearing Ratio
CN	Consolidation
DD	Dry Density (pcf)
DS	Direct Shear
GS	Grain Size Distribution
K	Permeability
MD	Moisture/Density Relationship (Proctor)
MR	Resilient Modulus
PID	Photoionization Device Reading
PP	Pocket Penetrometer Approx. Compressive Strength (tsf)
SG	Specific Gravity
TC	Triaxial Compression
TV	Torvane Approx. Shear Strength (tsf)
UC	Unconfined Compression

SAMPLE TYPE SYMBOLS

☒	2.0" OD Split Spoon (SPT) (140 lb. hammer with 30 in. drop)
I	Shelby Tube
▣	3-1/4" OD Split Spoon with Brass Rings
○	Small Bag Sample
▚	Large Bag (Bulk) Sample
▮	Core Run
▱	Non-standard Penetration Test (3.0" OD split spoon)

GROUNDWATER SYMBOLS

▽	Groundwater Level (measured at time of drilling)
▼	Groundwater Level (measured in well or open hole after water level stabilized)

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No 4 (4.5mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No 4 (4.5mm)
Sand	No. 4 (4.5 mm) to No. 200 (0.074 mm)
Coarse sand	No. 4 (4.5 mm) to No. 10 (2.0 mm)
Medium sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and Clay	Smaller than No. 200 (0.074mm)

COMPONENT PROPORTIONS

PROPORTION RANGE	DESCRIPTIVE TERMS
< 5%	Clean
5 - 12%	Slightly (Clayey, Silty, Sandy)
12 - 30%	Clayey, Silty, Sandy, Gravelly
30 - 50%	Very (Clayey, Silty, Sandy, Gravelly)

Components are arranged in order of increasing quantities.

NOTES: Soil classifications presented on exploration logs are based on visual and laboratory observation.
Soil descriptions are presented in the following general order:

Density/consistency, color, modifier (if any) GROUP NAME, additions to group name (if any), moisture content. Proportion, gradation, and angularity of constituents, additional comments. (GEOLOGIC INTERPRETATION)

Please refer to the discussion in the report text as well as the exploration logs for a more complete description of subsurface conditions.

MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch.
MOIST	Damp but no visible water.
WET	Visible free water, usually soil is below water table.

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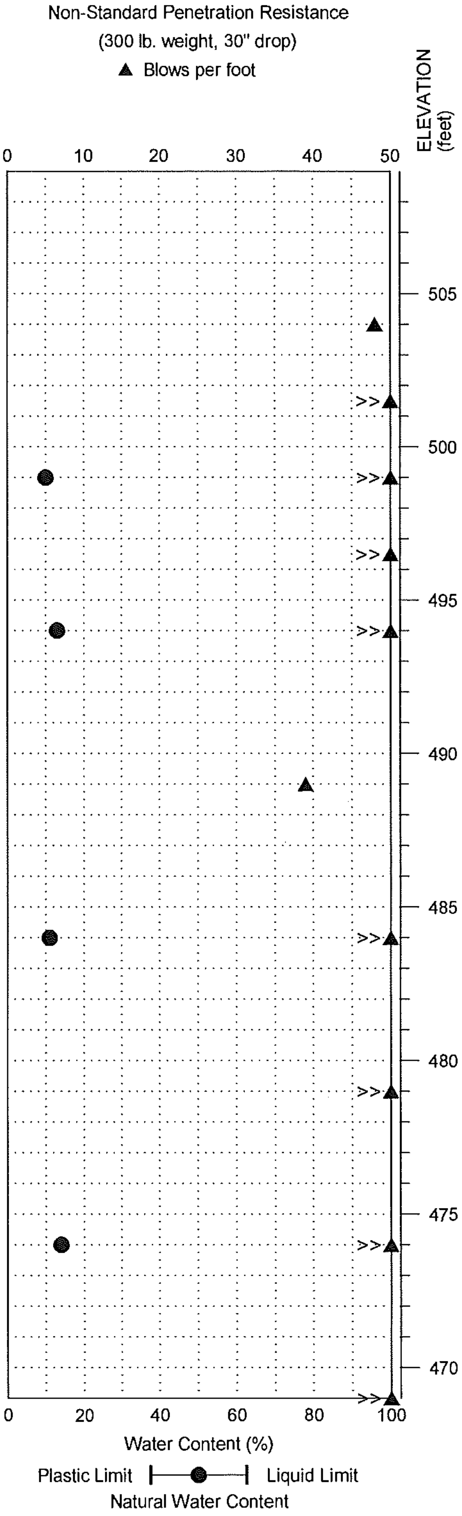
LEGEND OF TERMS AND SYMBOLS USED ON EXPLORATION LOGS

PROJECT NO.: 2011-073-21 T100 FIGURE: A-1

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Truck-mounted Mobile B-65, 4 1/4" HSA
SAMPLING METHOD: D&M sampler with 300 lb autohammer
LOCATION: See Figure 2, Northeast corner of proposed parking structure

DATE STARTED: 6/29/2011
DATE COMPLETED: 6/29/2011
LOGGED BY: J. Gillie
SURFACE ELEVATION: 509.0 ± feet

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0	GM	Three inches of HMA pavement.					
		Loose of medium dense, brown, slightly silty, gravelly, SAND, over slightly silty, sandy, angular to sub-rounded GRAVEL, moist. **(FILL)** Bumpy drilling from 2.5' to 5' below ground surface (bgs).					
5		No recovery in S-1 at 5' bgs, cobble stuck in sampler tip.	■	S-1	11-18-30		
	ML	Very dense, dark brown, sandy SILT, moist, with rust mottling. **(WEATHERED GLACIAL TILL)**	■	S-2	17-25-50		
10	ML	Very dense, dark brown, gravelly, very sandy SILT, moist, with multi-colored gravel particles, and dark brown to black subvertical partings. Outside of sampler wet at 12.5' to 14' bgs.	■	S-3	18-34-50/3"	GS	
	ML	Very dense, dark gray, slightly sandy, SILT, moist, with distorted shear laminations. **(GLACIAL TILL)**	■	S-4	15-42-50/4"		
15	CL	Hard, dark gray, slightly sandy, lean CLAY, moist. **(GLACIOLACUSTRINE DEPOSITS)**	■	S-5	18-32-50		
20		Broken cobble noted at top of sample S-6 at 20' bgs. Thin distorted shear laminations noted in sample at 21' bgs. Auger steaming from heat generated by drilling. Very slow advance of auger.	■	S-6	5-15-24		
25	CL	Hard, dark gray, slightly sandy, slightly gravelly, lean CLAY, moist, with diamicton appearance and shiny pockets formed around coarse sand and gravel particles. Bumpy drilling from about 25' to 26' bgs.	■	S-7	3-17-45		
30	CL	Hard, dark gray, sandy, lean CLAY, moist, crumbly texture with distorted horizontal shear laminations.	■	S-8	15-21-50/5"		
35	CL	Hard, dark gray, slightly gravelly, sandy, lean CLAY, with scattered particles of fine to coarse sand and fine gravel.	■	S-9	9-22-50/5"	GS	
40							



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

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FOR THE KELLMAN SITE
SAMMAMISH, WASHINGTON

BORING:
BH-1

PAGE: 1 of 2

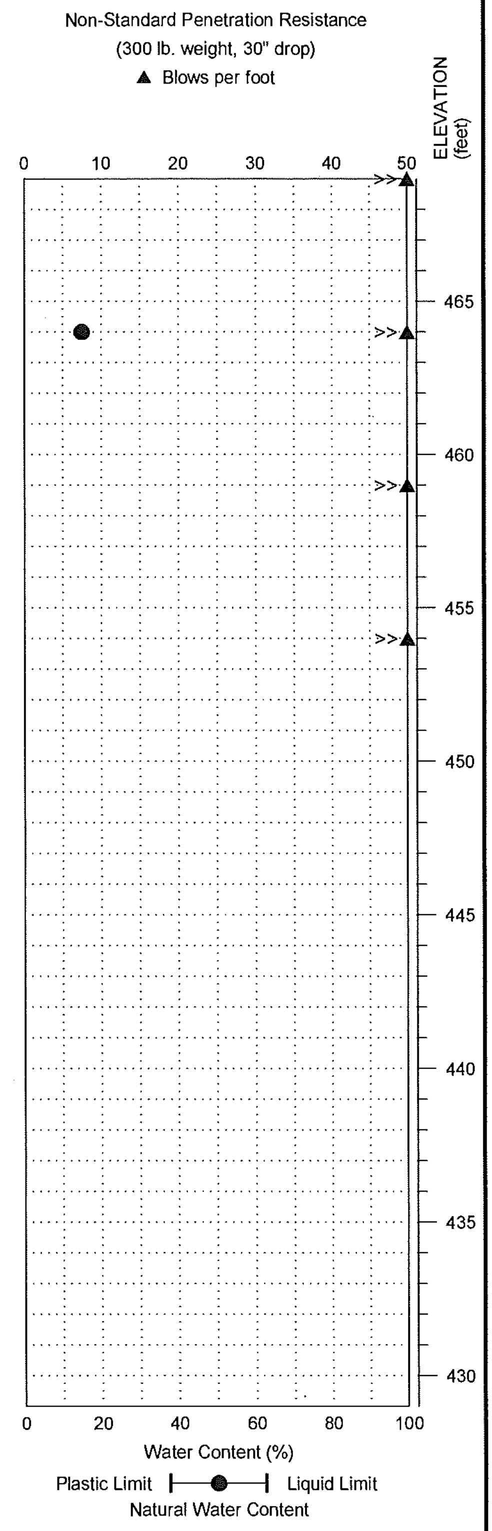
PROJECT NO.: 2011-073-21 T100 FIGURE: A-2

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Truck-mounted Mobile B-65, 4 1/4" HSA
SAMPLING METHOD: D&M sampler with 300 lb autohammer
LOCATION: See Figure 2, Northeast corner of proposed parking structure

DATE STARTED: 6/29/2011
DATE COMPLETED: 6/29/2011
LOGGED BY: J. Gillie
SURFACE ELEVATION: 509.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
40	▨		Distorted horizontal shear laminations and crumbly texture noted in sample S-10 at 40' bgs.	▪	S-10	9-25-50/5"		
45	▨		Shear laminations observed in sample S-11 at 45' bgs.	▪	S-11	8-16-50/5"		
50	▨			▪	S-12	9-21-50		
	▨		Auger making scraping sounds while drilling at 53' bgs.					
55	▨		Low recovery in sample S-13 at 55' bgs, with broken cobble observed in sampler.	▣	S-13	50/4"		

Borehole BH-1 terminated at 55.5' bgs. Only ground water observed was noted on outside of sampler at 12.5' bgs.



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

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SAMMAMISH, WASHINGTON

BORING:
BH-1

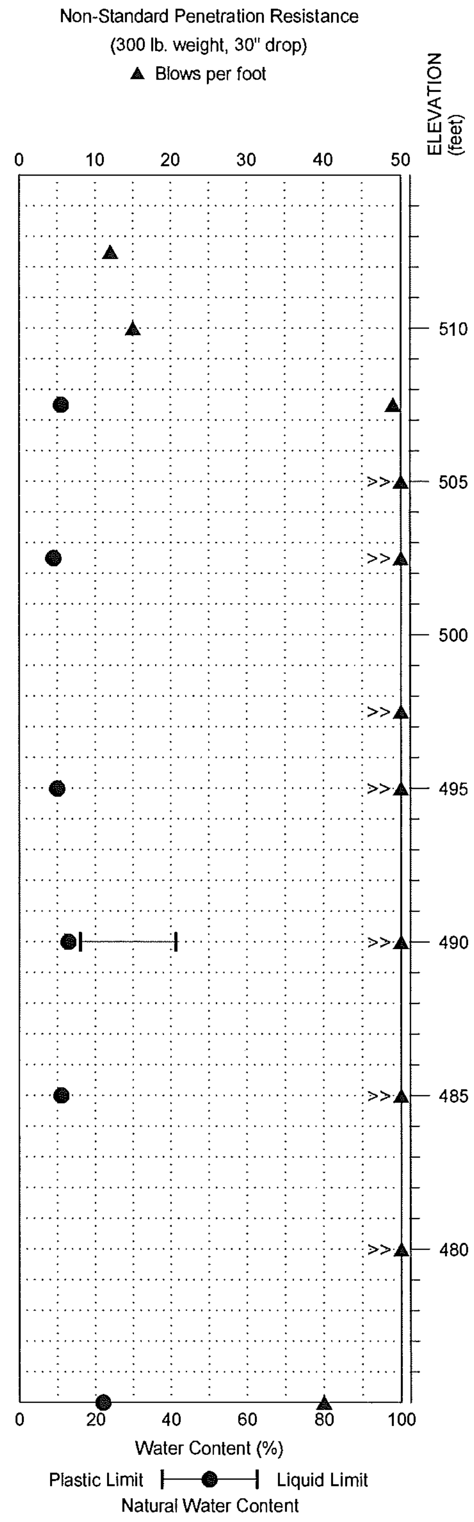
PAGE: 2 of 2

PROJECT NO.: 2011-073-21 T100 FIGURE: A-2

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Truck-mounted Mobile B-65, 4 1/4" HSA
SAMPLING METHOD: D&M sampler with 300 lb autohammer
LOCATION: See Figure 2, West side of proposed parking structure

DATE STARTED: 6/30/2011
DATE COMPLETED: 6/30/2011
LOGGED BY: J. Gillie
SURFACE ELEVATION: 515.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0		GM	Loose to medium dense, brown, silty, sandy GRAVEL, moist, with layers of sandy silt. **(FILL)**					
		ML	Medium dense, olive brown to yellow brown, sandy, SILT, moist, with rust mottling and scattered gravel. **(DISTURBED/WEATHERED GLACIAL TILL)**	■	S-1	4-5-7		
5			No bedding observed in sample S-2 at 5' below ground surface (bgs).	■	S-2	3-6-9		
		ML	Dense, yellow brown, gravelly, very sandy, SILT, moist. Faint bedding noted in a silty, fine sand lense. Broken cobble/gravel in sampler at 8.5' bgs.	■	S-3	13-20-29	GS	
10				■	S-4	20-50/6"		
		ML	Very dense, olive brown to olive gray, gravelly, sandy, SILT, moist. **(GLACIAL TILL)**					
15		ML	Hard, dark gray, slightly gravelly, sandy, SILT, moist, with horizontal, distorted shear laminations. Slow occasional bumpy drilling.	■	S-5	10-36-50/4"		
		CL	Hard, dark gray, slightly gravelly, slightly sandy, lean CLAY, moist, with crumbly texture, and shear laminations. Small pocket of saturated sand noted. **(GLACIOLACUSTRINE DEPOSIT)** Added water to auger to aid drilling.	■	S-6	20-50/6"		
20				■	S-7	21-50/6"		
25		CL	Hard, dark gray, gravelly, sandy, lean CLAY, moist.	■	S-8	20-22-34	AL	
30			Thin sand seam visible due to water introduced during drilling. Shear laminations observed in portions of the sample and massive clay in others. Smooth drilling.	■	S-9	14-27-34	GS	
35			Scattered black gravel observed in sample S-10 at 35' bgs.	■	S-10	7-21-33		
40								



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

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FOR THE KELLMAN SITE
SAMMAMISH, WASHINGTON

BORING:
BH-2

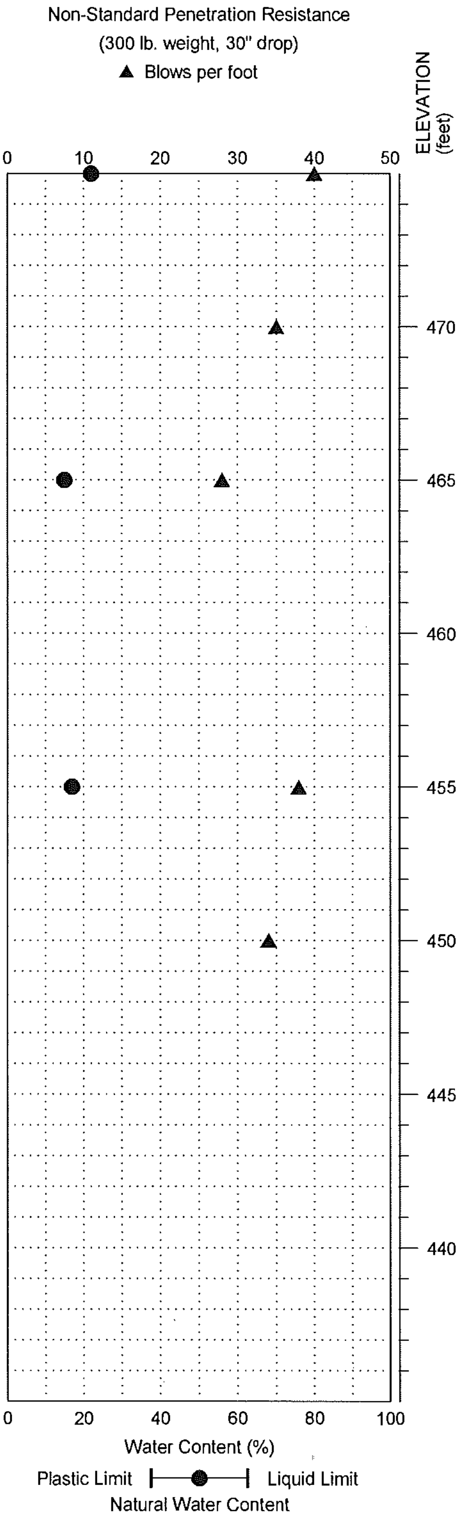
PAGE: 1 of 2

PROJECT NO.: 2011-073-21 T100 FIGURE: A-3

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Truck-mounted Mobile B-65, 4 1/4" HSA
SAMPLING METHOD: D&M sampler with 300 lb autohammer
LOCATION: See Figure 2, West side of proposed parking structure

DATE STARTED: 6/30/2011
DATE COMPLETED: 6/30/2011
LOGGED BY: J. Gillie
SURFACE ELEVATION: 515.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
40	▨	CL	Hard, dark gray, lean CLAY, moist, with laminations.	▪	S-11	12-13-27		
45	▨	CL	Hard, dark gray, sandy, lean CLAY, with scattered coarse sand and fine gravel particles.	▪	S-12	10-14-21		
50	▨			▪	S-13	6-11-17		
55	▨		No recovery of sample S-14 at 55' bgs. Blows overstated due to obstruction.	▪	S-14	26-50/4"		
60	▨	CL	Hard, dark gray, slightly sandy, lean CLAY, moist, with shear laminations and slickensides, scattered gravel.	▪	S-15	13-17-21		
65	▨		Vertical partings observed in sample S-16 at 65' bgs.	▪	S-16	8-15-19		
			Borehole BH-2 terminated at 66.5' bgs.					
70								
75								
80								



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

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BORING:
BH-2
PAGE: 2 of 2

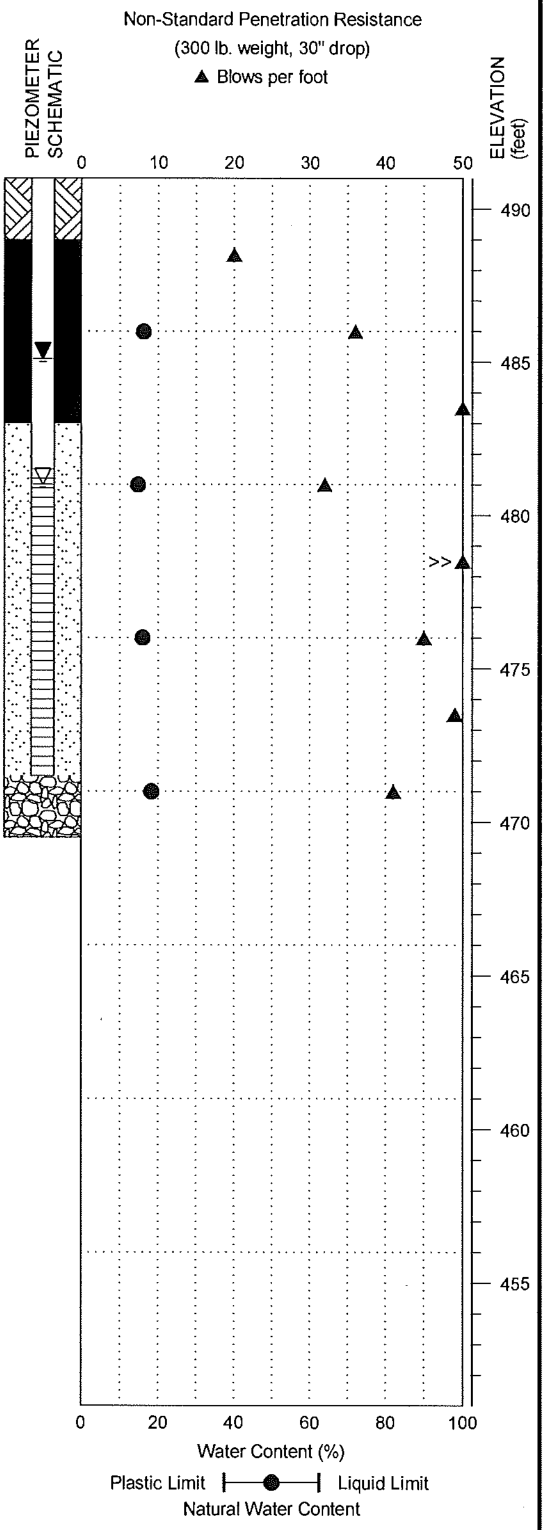
PROJECT NO.: 2011-073-21 T100 FIGURE: A-3

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Truck-mounted Mobile B-65, 4 1/4" HSA
SAMPLING METHOD: D&M sampler with 300 lb autohammer
LOCATION: See Figure 2, Near proposed pool with diving well

DATE STARTED: 6/30/2011
DATE COMPLETED: 6/30/2011
LOGGED BY: J. Gillie
SURFACE ELEVATION: 491.0 ± feet

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0	ML	Loose to medium dense, dark brown to brown, gravelly, sandy SILT, moist to very moist. **(FILL)**			
	ML	Medium dense, light brown, sandy SILT, moist, with 1" sand seam at 3.5' bgs, silt has faint bedding. **(WEATHERED TILL)**	S-1	6-7-13	
5	SM	Dense, olive brown, silty, fine SAND, moist.	S-2	3-14-22	
	CL	Hard, gray, fine sandy, lean CLAY, moist, with distorted horizontal shear laminations. Fine sand at 5' bgs before grading to lean CLAY without sand at 5.5'. **(GLACIOLACUSTRINE)** Crumbly texture and scattered coarse gravel in sample S-3 at 7.5' bgs.	S-3	4-14-36	
10	SM	Dense, dark gray, slightly gravelly, very silty, SAND, wet, sand observed as sand pockets.	S-4	5-13-19	GS
	ML	Very dense, dark gray, fine sandy, SILT, with crumbly texture and a 4" to 6" thick water bearing layer of medium to fine sand.	S-5	12-30-50/5"	
15	SM	Dense, dark gray, gravelly, silty, fine to medium SAND, wet, interbedded layers of fine sand, medium sand, and silty sand.	S-6	15-21-24	GS
	CL	Hard, dark gray, slightly sandy, lean CLAY, moist.	S-7	15-21-28	
20		Scattered coarse sand and fine gravel.	S-8	11-18-23	

Borehole BH-3 terminated at 21.5' bgs. Ground water observed at about 10' bgs during drilling. Ground water level measured at 5.9' bgs about 5.5 hours after well installation was completed.



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BORING:
BH-3

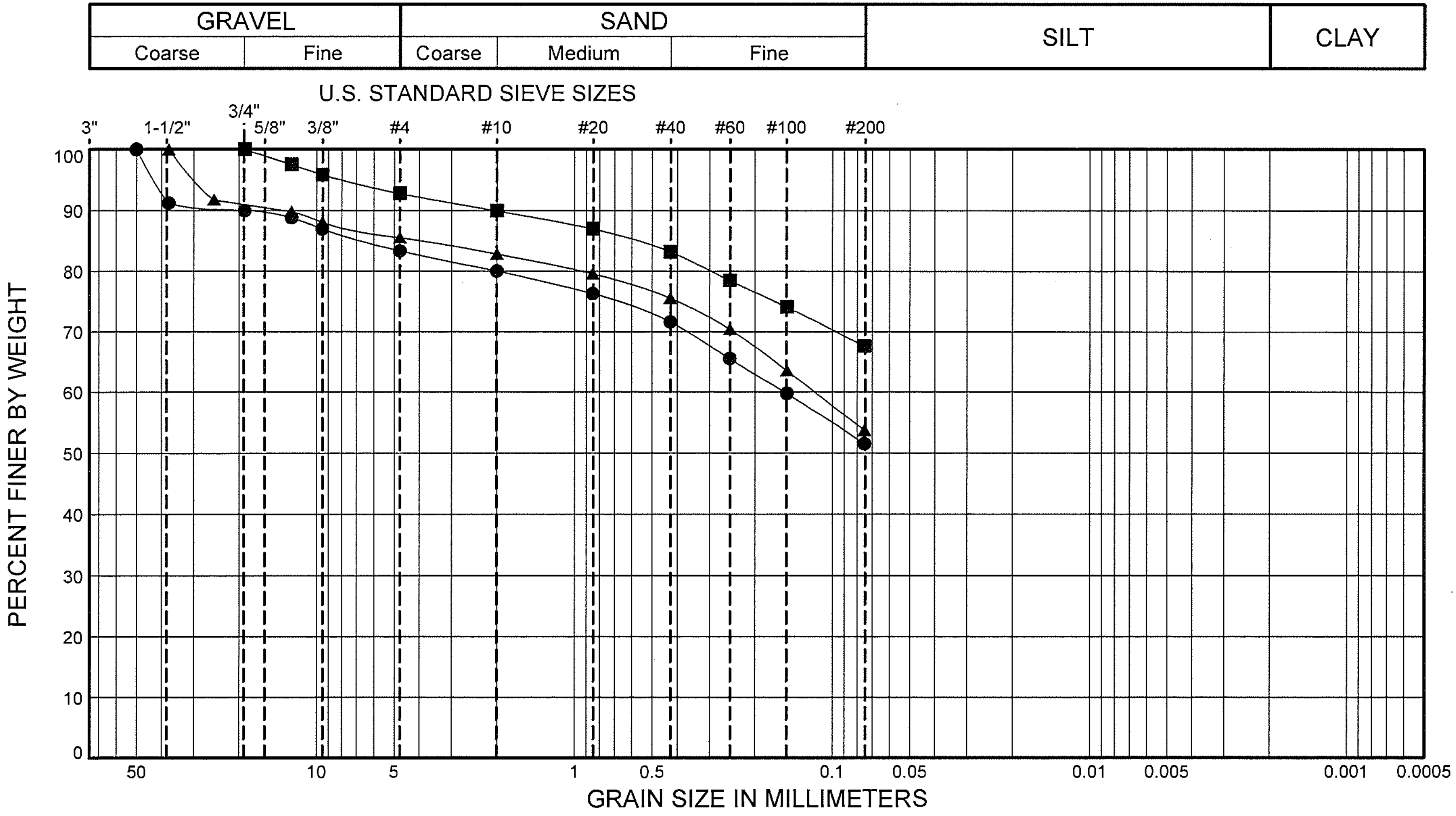
PAGE: 1 of 1

PROJECT NO.: 2011-073-21 T100 FIGURE: A-4

PZO-DSM 2011-073 T100.GPJ 8/1/11

APPENDIX B

LABORATORY TESTING



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	LL	PL	PI	Gravel %	Sand %	Fines %
●	BH-1	S-3	10.0 - 11.3	(ML) Olive brown, sandy SILT with gravel	10				16.6	31.8	51.6
■	BH-1	S-9	35.0 - 36.5	(CL) Dark gray, sandy lean CLAY	14				7.1	25.2	67.6
▲	BH-2	S-3	7.5 - 9.0	(ML) Yellowish brown, sandy SILT	11				14.4	31.9	53.8

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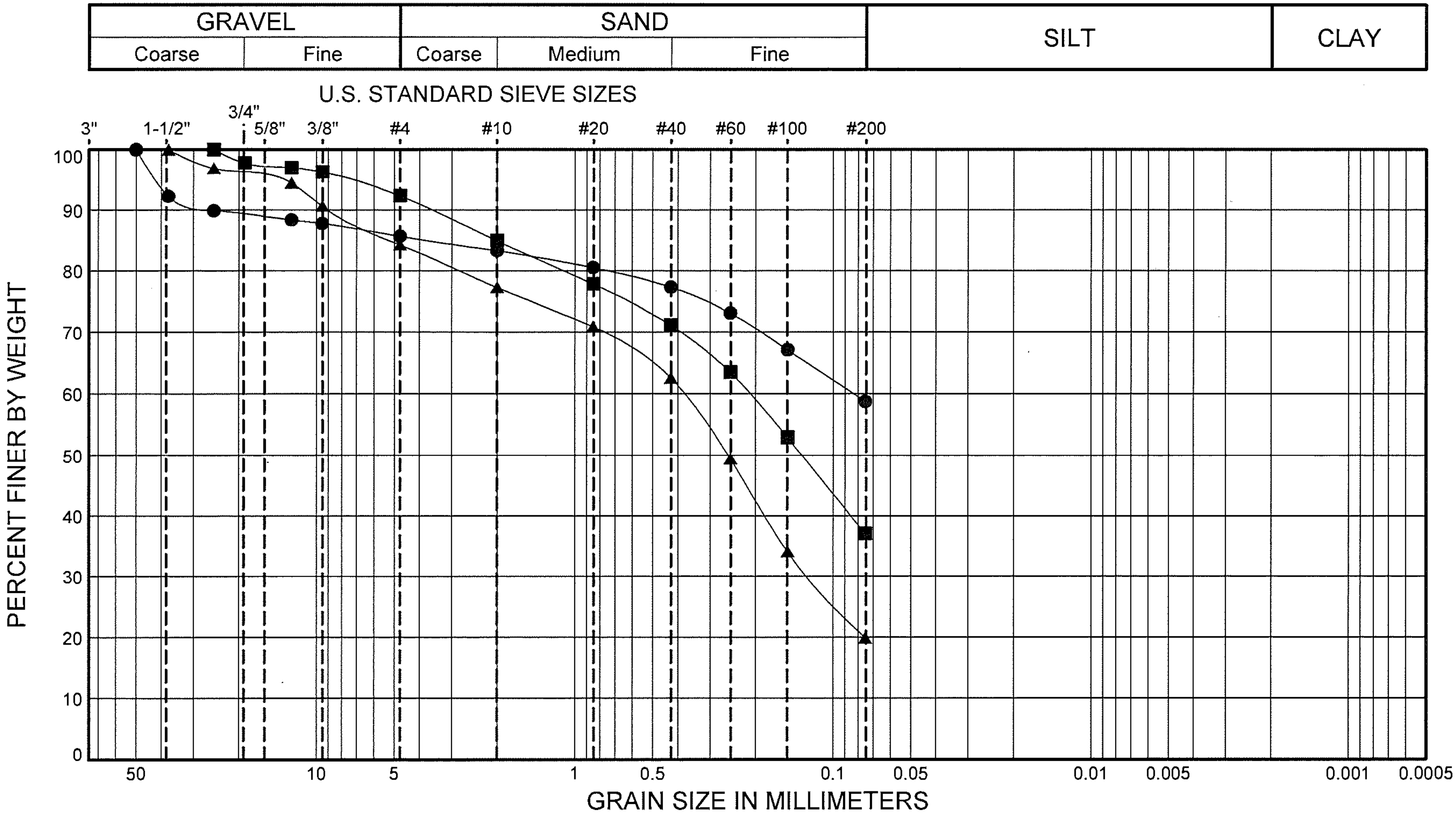
PARTICLE-SIZE ANALYSIS OF SOILS METHOD ASTM D422

PROJECT NO.: 2011-073-21 T100 FIGURE: B-1



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	LL	PL	PI	Gravel %	Sand %	Fines %
●	BH-2	S-9	30.0 - 31.5	(CL) Dark gray, sandy lean CLAY	11				14.3	26.9	58.8
■	BH-3	S-4	10.0 - 11.5	(SM) Gray, silty SAND	15				7.6	55.3	37.1
▲	BH-3	S-6	15.0 - 16.5	(SM) Gray, silty SAND with gravel	16				15.7	64.4	19.9

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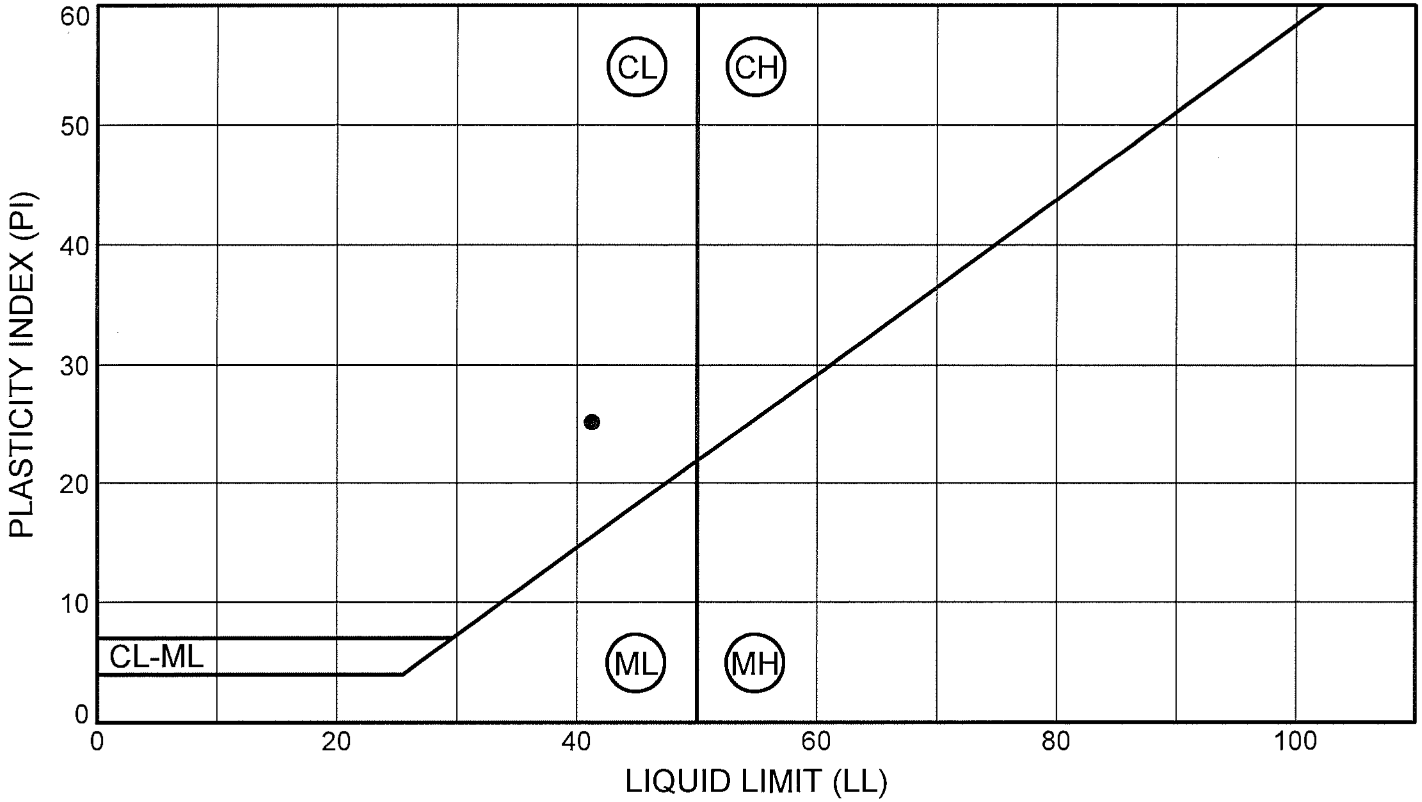
PARTICLE-SIZE ANALYSIS OF SOILS METHOD ASTM D422

PROJECT NO.: 2011-073-21 T100 FIGURE: B-2



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	BH-2	S-8	25.0 - 26.5	(CL) Dark gray, sandy lean CLAY	13	41	16	25	

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FOR THE KELLMAN SITE
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LIQUID LIMIT, PLASTIC LIMIT AND
PLASTICITY INDEX OF SOILS
METHOD ASTM D4318

PROJECT NO.: 2011-073-21 T100 FIGURE: B-3

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Celebrating 25 Years of Service

Subsurface Exploration, Geologic Hazard, and
Preliminary Geotechnical Engineering Report

SAMMAMISH LIBRARY

Sammamish, Washington

Prepared for

King County Library System

Project No. KE070718A
November 29, 2007

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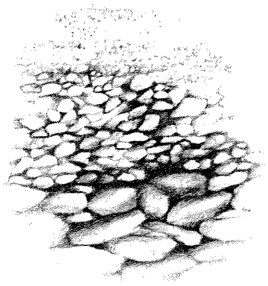
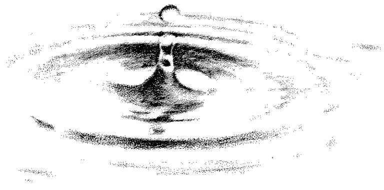
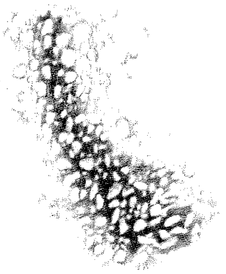


Geotechnical Engineering



Water Resources



Environmental Assessments and
Remediation

Sustainable Development Services

Geologic Assessments

B08.00470

SUBSURFACE EXPLORATION, GEOLOGIC HAZARD, AND PRELIMINARY GEOTECHNICAL ENGINEERING REPORT

SAMMAMISH LIBRARY

Sammamish, Washington

Prepared for:
King County Library System
c/o URS Corporation
1501 4th Avenue, Suite 1400
Seattle, Washington 98101

Prepared by:
Associated Earth Sciences, Inc.
911 5th Avenue, Suite 100
Kirkland, Washington 98033
425-827-7701
Fax: 425-827-5424

November 29, 2007
Project No. KE070718A

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 the subject project. Our recommendations are preliminary in that construction details have not been finalized at the time of this report. The location of the subject site is shown on the "Vicinity Map," Figure 1. The locations of the existing and proposed structures, and the approximate locations of the explorations accomplished for this study, are presented on the "Site and Exploration Plan," Figure 2. In the event that any changes occur in the nature or design of the proposed project as discussed herein, the conclusions and recommendations contained in this report should be reviewed and modified, or verified, as necessary.

1.1 Purpose and Scope

The purpose of this study was to provide subsurface data to be used in the preliminary design and development of the subject project. Our study included a review of available geologic literature, drilling exploration borings, and performing geologic studies to assess the type, thickness, distribution, and physical properties of the subsurface sediments and shallow ground water conditions. Geotechnical engineering studies were also conducted to assess the type of suitable foundation, allowable foundation soil bearing pressures, anticipated settlements, basement/retaining wall lateral pressures, floor support recommendations, temporary shoring recommendations, and drainage considerations. This report summarizes our current fieldwork and offers development recommendations based on our present understanding of the project.

1.2 Authorization

Authorization to proceed with this study was granted by the King County Library System through our scope of work/contract agreement letter dated October 22, 2007. This report has been prepared for the exclusive use of the King County Library System 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, express or implied, is made.

2.0 PROJECT AND SITE DESCRIPTION

The subject site is located on the west side of 228th Street SE, approximately ½ block north of NE 8th Street. The Sammamish City Hall (801 228th Street SE) lies adjacent to the north side of the library site (Figure 1). Although no topographic site plan was available at the time of our study, the topography of the site generally slopes gently down toward the north and east. The eastern portion of the building area consists of lawn. An area of the lawn below the southeastern portion of the building (in the area of exploration boring EB-4) is underlain by a cellular concrete pavement known as "Grasscrete". The western portion of the site appears to have been recently filled and is currently unvegetated. An existing parking lot borders the building area to the east and a private driveway to an adjacent residence borders the site to the west and south.

Our understanding of the proposed project is based on review of a site plan provided by URS Corporation, and on a conversation with the project architect, Ms. Amy Williams of Perkins and Will. It is our understanding that the new library will occupy a footprint of approximately 19,000 square feet where shown on the "Site and Exploration Plan," Figure 2. The library will include a basement level parking garage with a finish floor elevation of approximately 522 feet. The base of the foundation is estimated to be at elevation 519 feet. Exterior grades adjacent to the building are anticipated to remain similar to those of the existing grades. According to Ms. Williams, the existing grade along the north side of the proposed building is at approximately elevation 525 feet, and the existing grade at the southeast building corner is approximately elevation 534 feet. Excavation depths for the basement level parking garage are therefore anticipated to range from approximately 6 to 15 feet.

3.0 SUBSURFACE EXPLORATION

Our field study included drilling four geotechnical borings (EB-1 through EB-4) at the approximate locations shown on Figure 2. 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. Our explorations were approximately located in the field relative to known site features shown on the "Site and Exploration Plan."

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

3.1 Exploration Borings

The exploration borings were completed by advancing a 3¼-inch, inside-diameter, hollow-stem auger with a track-mounted drill rig. During drilling, soil samples were obtained at depth intervals of approximately 2.5 to 5 feet. The conditions encountered in our explorations were continuously observed and logged by an engineering geologist from our firm. 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 boring logs.

The samples obtained from the split-barrel samplers were classified in the field and representative portions placed in watertight containers. The samples were then transported to our laboratory for further visual classification.

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 applicable geologic literature. As shown on the field logs, the explorations generally encountered glacially consolidated, granular sediments overlain in places by variable thicknesses of fill. The following section presents more detailed subsurface information organized from the youngest to the oldest sediment types.

4.1 Stratigraphy

Fill

Fill soils (those not naturally placed) were encountered at the locations of exploration borings EB-1, EB-3, and EB-4. The fill generally consisted of loose to medium dense, silty sand with gravel with minor quantities of organic debris in places. Fill thicknesses of approximately 12, 13, and 6 feet were observed at the locations of exploration borings EB-1, EB-3, and EB-4, respectively. The gradation of the fill soils appears generally similar to that of the underlying native sediments.

Vashon Lodgement Till

Sediments encountered directly below the surficial sod layer at the location of exploration boring EB-2, and directly below the fill at the other boring locations, generally consisted of dense to very dense, unsorted, silty sand with moderate to high gravel content and scattered cobbles. We interpret these sediments to be representative of Vashon lodgement till. Vashon lodgement till consists of an unsorted mixture of silty, sand, and gravel that was deposited directly from basal, debris-laden glacial ice during the Vashon Stade of the Fraser Glaciation, approximately 12,500 to 15,000 years ago. The high relative density characteristic of lodgement till is due to its consolidation by the massive weight of the glacial ice from which it was deposited. At the location of exploration boring EB-4, the Vashon lodgement till extended beyond the maximum depth explored of approximately 40.5 feet. Exploration borings EB-1 through EB-3 met with refusal in the till at depths of approximately 10.5 to 20.5 feet. It should be noted that lodgement till typically contains scattered cobbles and boulders. Several small stockpiles of boulders were observed in the fill area in the western portion of the site. The contractor should be prepared to handle cobbles and boulders if encountered during construction.

Review of the regional geologic map titled *Geologic Map of the Issaquah 7.5' Quadrangle, King County, Washington*, dated 2006, prepared by Booth for the USGS, indicates that the area of the subject site is underlain by Vashon lodgement till. Our interpretation of the sediments encountered in our explorations is in general agreement with the regional geologic map.

4.2 Hydrology

Ground water seepage was not encountered in any of the explorations accomplished for our study. Although not encountered in any of our explorations, it is common in areas underlain by lodgement till for seepage to seasonally accumulate atop the surface of the dense, unweathered lodgement till sediments. This seepage, known as interflow, occurs when surface water percolates down through the surficial weathered till or fill soils and becomes perched atop the underlying, lower permeability unweathered till sediments. It is possible that interflow may periodically accumulate at the site. It should be noted that the occurrence and level of ground water seepage may vary in response to such factors as changes in season, precipitation, and site use.

II. GEOLOGIC HAZARDS AND MITIGATIONS

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

5.0 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 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- 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.

5.1 Surficial Ground Rupture

The nearest known fault traces to the project site are the South Whidbey Island Fault Zone (SWIFZ) located within approximately 1 mile to the northeast, and the Seattle Fault Zone located approximately 2 miles to the south.

In a recent study by the U.S. Geological Survey (USGS) (Sherrod et al., 2005, *Holocene Fault Scarps and Shallow Magnetic Anomalies Along the Southern Whidbey Island Fault Zone near Woodinville, Washington*, Open-File Report 2005-1136, March 2005) indicates that "strong" evidence of prehistoric earthquake activity has been observed along two fault strands thought to be part of the southeastward extension of the SWIFZ located approximately 15 miles north of the site. The study suggests as many as nine earthquake events along the SWIFZ may have occurred within the last 16,400 years. The recognition of this fault splay is relatively new, and data pertaining to it are limited with the studies still ongoing. The recurrence interval of movement along this fault system is still unknown, although it is hypothesized to be in excess of one thousand years.

Studies of the Seattle Fault Zone by the U.S. Geological Survey (USGS) (e.g., Johnson et al., 1994, *Origin and Evolution of the Seattle Fault and Seattle Basin, Washington*, Geology, v. 22, pp. 71-74; and Johnson et al., 1999, *Active Tectonics of the Seattle Fault and Central Puget Sound Washington - Implications for Earthquake Hazards*, Geological Society of

America Bulletin, July 1999, v. 111, n. 7, pp. 1042-1053) have provided evidence of surficial ground rupture along a northern splay of the Seattle Fault. According to the USGS studies, the latest movement of this fault was about 1,100 years ago when about 20 feet of surficial displacement took place. This displacement can presently be seen in the form of raised, wave-cut beach terraces along Alki Point in West Seattle and Restoration Point at the south end of Bainbridge Island. The recurrence interval of movement along this fault system is still unknown, although it is hypothesized to be in excess of several thousand years.

Due to the suspected long recurrence intervals for both fault zones, the potential for surficial ground rupture is considered to be low during the expected life of the proposed structure.

5.2 Seismically Induced Landslides

It is our opinion that the risk of damage to the proposed structure by seismically induced landsliding is low due to the lack of steep slopes in the project area. No mitigation of landslide hazards is warranted.

5.3 Liquefaction

Liquefaction is a process through which unconsolidated soil loses strength as a result of vibratory shaking, such as occurs during a seismic event. During normal conditions, the weight of the soil is supported by both grain-to-grain contacts, and by the hydraulic 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 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 sand or coarse silt with low relative densities, accompanied by a shallow water table. It is our opinion that the risk of damage to the proposed structure by liquefaction is low due to the high relative density of the underlying sediments, and the lack of adverse ground water conditions. No mitigation of liquefaction hazards is recommended.

5.4 Ground Motion

It is our opinion that any earthquake damage to the proposed structure, when founded on suitable bearing strata in accordance with the recommendations contained herein, will be caused by the intensity and acceleration associated with the event and not any of the above-discussed impacts. Structural design of the proposed building should follow the 2006 *International Building Code* (IBC). Information presented by the USGS Earthquake Hazards Program (2003 NEHRP Seismic Design Provisions) indicates a spectral acceleration for the project area for short periods (0.2 seconds) of $S_s = 1.29$ and for a 1-second period of $S_1 = 0.43$. Based on the results of subsurface exploration and on an estimation of soil properties at depth utilizing available geologic data, Site Class "C", in conformance with Table 1615.1.1 of the IBC, may be used.

6.0 EROSION HAZARDS AND MITIGATIONS

As of October 1, 2006, 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 for all sites 1 or more acres in size that discharge storm water to surface waters of the state. The TESC inspections must be completed by a Certified Erosion and Sediment Control Lead (CESCL) for the duration of the construction. TESC reports do not need to be sent to Ecology, but should be logged into the project Storm Water Pollution Prevention Plan (SWPPP). If the project does not require a SWPPP, the TESC reports should be kept in a file on-site, or by the permit holder if there is no facility on-site. Ecology also requires weekly turbidity monitoring by a CESCL of storm water leaving a site for all sites 5 acres or greater. Ecology requires a monthly summary report of the turbidity monitoring results (if performed) signed by the NPDES permit holder. If the monitored turbidity equals or exceeds 25 nephelometric turbidity units (NTU) (Ecology benchmark standard), the project best management practices (BMPs) should be modified to decrease the turbidity of storm water leaving the site. Changes and upgrades to the BMPs should be continued until the weekly turbidity reading is 25 NTU or lower. If the monitored turbidity exceeds 250 NTU, the results must be reported to Ecology within 24 hours and corrective action taken. Daily turbidity monitoring is continued until the corrective action lowers the turbidity to below 25 NTU.

In order to meet the current Ecology requirements, a properly developed, constructed, and maintained erosion control plan consistent with City of Sammamish and King County standards and best management erosion control practices will be required for this project. Associated Earth Sciences, Inc. (AESI) is available to assist the project civil engineer in developing site-specific erosion control plans. Based on past experience, it will be necessary to make adjustments and provide additional measures to the TESC plan in order to optimize its effectiveness. Ultimately, the success of the TESC plan depends on a proactive approach to project planning and contractor implementation and maintenance.

The erosion hazard of the site soils is high. 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.

Flow-control measures are also essential for collecting and controlling the site runoff. Flow paths across slopes should be kept to less than 50 feet in order to reduce the erosion and sediment transport potential of concentrated flow. Ditch/swale spacing will need to be shortened with increasing slope gradient. Ditches and swales that exceed a gradient of about 7 to 10 percent, depending on their flow length, should have properly constructed check dams installed to reduce the flow velocity of the runoff and reduce the erosion potential within the ditch. Flow paths that are required to be constructed on gradients between 10 to 15 percent should be placed in a riprap-lined swale with the riprap properly sized for the flow conditions. Flow paths constructed on slope gradients steeper than 15 percent should be placed in a pipe slope drain. AESI is available to assist the project civil engineer in developing a suitable erosion control plan with proper flow control.

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 is not going to 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.

Silt fencing should be utilized as buffer protection and not as a flow-control measure. Silt fencing is meant to be placed parallel with topographic contours to prevent sediment-laden runoff from leaving a work area or entering a sensitive area. Silt fences should not be placed to cross contour lines without having separate flow control in front of the silt fence. A swale/berm combination should be constructed to provide flow control rather than let the runoff build up behind the silt fence and utilize the silt fence as the flow-control measure. Runoff flowing in front of a silt fence will cause additional erosion and usually will cause a failure of the silt fence. Improperly installed silt fencing has the potential to cause a much larger erosion hazard than if the silt fence was not installed at all. The use of silt fencing should be limited to protect sensitive areas, and swales should be used to provide flow control.

6.1 Erosion Hazard Mitigation

To mitigate the erosion hazards and potential for off-site sediment transport, we would 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 then 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 an area other than installing the TESC features or timber harvesting. The recommended sequence of construction within a given area after timber harvesting 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 the top of steep 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 (if required) should be performed in accordance with the 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 (BMPs) 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

7.0 INTRODUCTION

Our exploration indicates that, from a geotechnical standpoint, the parcel is suitable for the proposed development provided the recommendations contained herein are properly followed. Portions of the building area are underlain by significant thicknesses of fill soils not suitable for foundation support. Excavation for construction of the basement level parking garage will remove most or all of the fill from below the building, but the fill appears to extend several feet below the elevation of the proposed footings in some areas (such as the northwestern portion of the building). In these areas, we recommend that the existing fill be overexcavated and replaced with structural fill, or deepened to bear directly on the competent, native soils. Given the depth of the proposed basement level excavation, temporary shoring may be required to achieve the proposed excavation depths without adversely impacting existing improvements to the north and east. Geotechnical recommendations for foundation and temporary shoring design and construction and provided below.

8.0 SITE PREPARATION

8.1 Clearing and Stripping

Site preparation of the planned building and pavement areas should include removal of all existing pavement, vegetation, debris, and any other deleterious materials. These unsuitable materials should be properly disposed of off-site. Additionally, any areas of organic topsoil should be removed and the remaining roots grubbed. Any buried utilities that underlie the new building area should also be removed or relocated. If the resulting depressions extend below the elevation of the new building subgrade, they should be backfilled with structural fill, as described in the "Structural Fill" section of this report. After clearing and stripping have been completed, excavation for the basement level of the structure may be conducted.

8.2 Temporary Cut Slopes

In our opinion, stable, temporary 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 or utility trenches greater than 4 feet in depth completed within the loose to medium dense fill soils can be planned at a maximum slope of 1.5H:1V (Horizontal:Vertical). Temporary cut slopes within the dense to very dense, native lodgement till sediments can be planned at a 1H:1V inclination. Flatter temporary cut slopes are recommended in any areas where seepage is encountered. 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.

In those areas where insufficient lateral room exists to construct the basement level excavation using open cuts, temporary shoring should be used. Recommendations for temporary shoring are provided subsequently in this report.

8.3 Permanent Cut Slopes

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

8.4 Site Disturbance

The on-site sediments contain a high percentage of fine-grained material that makes them moisture-sensitive and subject to disturbance when wet. The contractor must use care during site preparation and excavation operations so that the underlying soils are not softened. If disturbance occurs, the softened soils should be removed and the area brought to grade with structural fill.

Consideration should be given to protecting access and staging areas with an appropriate section of crushed rock or asphalt treated base (ATB). 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 during wet weather 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.

9.0 STRUCTURAL FILL

Placement of structural fill may be required to establish desired grades in some areas. 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.

9.1 Subgrade Compaction

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

9.2 Structural Fill Compaction

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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 at least 95 percent of the modified Proctor maximum density using ASTM:D 1557 as the standard. Roadway and utility trench backfill should be placed and compacted in accordance with applicable municipal codes and standards. The top of the compacted fill should extend horizontally a minimum distance of 3 feet beyond the perimeter footings or pavement edges before sloping down at an angle no steeper than 2H:1V. Fill slopes should either be overbuilt and trimmed back to final grade or surface-compacted to the specified density. Structural fill placed in foundation excavations must extend horizontally outward from the edges of the footings a distance greater than or equal to the thickness of the fill below the footings.

9.3 Moisture-Sensitivity

The on-site fill and native sediments are generally suitable for use as structural fill, but both contain high percentages of silt and will highly moisture sensitive and subject to disturbance when wet. If the moisture contents of these sediments are elevated at the time of construction, moisture conditioning may be required prior to their use as structural fill. Such moisture conditioning could consist of spreading out and aerating the soil during warm, dry weather. At the time of our study, the existing fill soils generally contained moisture conditions well above the optimum for achieving suitable compaction for use as structural fill. At the moisture contents observed, these soils would require extensive moisture conditioning prior to placement as structural fill. Soils encountered in our explorations that were over-optimum for structural fill placement are described on the attached boring logs as "very moist" or "wet". Although significant quantities of organic debris were not encountered in our explorations, areas of organic debris may be encountered within the existing fill during construction. Any portions of the existing fill soils that contain significant quantities of organic debris are not suitable for use as structural fill. It should be noted that the moisture content of the soil at the site likely varies with season, location, and depth.

9.4 Structural Fill Testing

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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 at least 3 business days 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 soils in structural fills should be limited to favorable dry weather conditions. 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 and at least 25 percent greater than the No. 4 sieve.

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A representative from our firm should observe 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 frequency.

10.0 TEMPORARY SHORING

10.1 Soldier Pile Wall

It is anticipated that excavation for construction of the proposed basement-level parking garage will require cut depths of up to approximately 15 feet. If excavation to the proposed depths cannot be achieved within the site constraints using open cuts, temporary shoring is recommended. This section of the report presents recommendations for design of temporary shoring for the basement excavation.

The most common method of shoring used in the Puget Sound area consists of a soldier pile/waler shoring system utilizing wide-flange steel beams (soldier piles). For excavations of approximately 15 feet or less, the soldier piles typically may be cantilevered without the use of tiebacks or bracing. Soldier piles are placed in pre-drilled holes that extend below the bottom of the excavation. The portion of each soldier pile extending below the bottom of the excavation is grouted in place with sufficient-strength concrete to transmit the load from the soldier beams into the soil below the excavation level. The upper portion of the soldier pile is then backfilled with a relatively weak grout so that it may be removed, as necessary, for placement of lagging.

10.2 "Active" and "At-Rest" Lateral Earth Pressure Conditions

In those areas where there are no settlement-sensitive structures near the top of the proposed excavation, soldier pile walls may be designed using active lateral earth pressure conditions. Active earth pressure conditions will allow a small amount of movement of the retained soil and wall to develop the shear strength within the retained soil, and lessen the shoring design loads. Under these conditions, the amount of lateral movement of the wall will be equal to approximately 0.1 percent of the wall height. If minor settlement behind the wall does occur, we estimate that it will most likely occur within a distance behind the wall equal to the height of the wall.

At-rest lateral earth pressure conditions should be used in those areas where the wall is located near settlement-sensitive structures. At-rest earth pressure conditions will allow construction of the retaining wall with little or no movement of the retained soil. It should be emphasized that in this case, the shoring wall <u>must</u> be designed to totally resist ho
such movement could result in damage to nearby sensitive structures.
design based on either at-rest or active earth pressure conditions, the
designed to withstand any horizontal surcharge pressures exerted by ad
in addition to the pressures exerted by the retained soil.

Shoring Design.

10.3 Shoring Design Parameters

For a cantilever shoring system, the applied lateral pressure can be pr
pressure distribution termed as an equivalent fluid density. We have pr
densities for shoring design based on a horizontal backslope behind the soldier pile wall. The equivalent fluid density presented subsequently does not account for stockpiled materials, buildings, or other surcharge loads within the influence zone behind the top of the wall. Based on these considerations and the anticipated soil conditions in the vicinity of the proposed retaining wall, we recommend design of the shoring with an active earth pressure condition using an equivalent fluid density of 35 pounds per cubic foot (pcf). For fully restrained, at-rest earth pressure conditions, we recommend using an equivalent fluid density of 50 pcf. The active or at-rest pressure distribution should be assumed to be applied over the pile spacing above the base of the excavation. Below the base of the excavation, the lateral pressure should be applied over one concreted soldier pile diameter. Surcharge pressures due to adjacent traffic loads should be modeled as an additional 2 feet of retained soil height. To resist lateral loads, an allowable passive equivalent fluid unit weight of 250 pcf should be used for design assuming the soldier piles are embedded in undisturbed, lodgement till sediments. The passive fluid pressure can be assumed to act over two concreted pile diameters. The passive envelope should be truncated to neglect the first 2 feet of pile penetration below the base of the lowest adjacent excavation elevation. The passive pressure presented is an allowable design value.

Embedment depths of soldier piles below the final excavation level must be designed to provide adequate lateral and/or kick-out resistance to horizontal loads and satisfy moment equilibrium. The design lateral resistance may be computed on the basis of pressures presented previously.

10.4 Lagging

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We recommend lagging be installed in all areas. Due to soil arching effects, lagging may be designed for 50 percent of lateral earth pressures used for shoring design. Prompt and careful installation and backfilling of lagging will reduce potential loss of ground. Requirements for lagging should be made the responsibility of the shoring subcontractor to prevent soil failure, sloughing, and loss of ground, and to provide a safe working condition. We recommend any voids between the lagging and the soil be backfilled. However, the backfill should not allow potential hydrostatic buildup behind the wall. Drainage behind the wall must be maintained. Washed rock or free-draining sand and gravel may be used for lagging backfill.

10.5 Caving

The sediments encountered in our borings consisted predominantly of dense to very dense lodgement till with significant thicknesses of existing fill in some areas. Although we anticipate that the portions of the borings extending into the lodgement till will generally remain open, caving could occur in areas of existing fill or seepage. The contractor should be prepared to case the soldier pile borings if necessary to prevent loss of ground and facilitate proper grout placement in the event that caving conditions are encountered.

10.6 Cobbles and Boulders

Scattered cobbles and boulders should be expected to be encountered within the lodgement till sediments, and possibly within the existing fill. Refusal on what is inferred to be cobbles and boulders was encountered in three of the four exploration borings drilled at the site. As such the shoring contractor should anticipate potentially difficult drilling conditions.

10.7 Construction Observation

Since completion of the piling takes place below ground, the judgment and experience of the geotechnical engineer or his field representative must be used for determining the acceptability of each pile. Consequently, the use of the presented design information 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 observe the contractor's operations. AESI, acting as the owner's field representative, would keep records of pertinent installation data. A final summary report would then be distributed following completion of pile installation.

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10.8 Monitoring of Surrounding Structures

A survey of the surrounding structures and other critical reference points should be performed prior to construction activities. These points should then be accurately monitored both horizontally and vertically by a licensed surveyor until the excavation is complete and permanent walls are constructed. A photographic and/or video survey is also recommended for surrounding structures to document their condition prior to development. This monitoring would act to provide early notice of slope movement or site settlement and provide an accurate record of pre-construction site conditions.

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11.0 FOUNDATIONS

11.1 Allowable Soil Bearing Pressure

Spread footings may be used for building support when founded either directly on the medium dense to very dense lodgment till sediments or on structural fill placed over these materials. The existing fill soils are not suitable for foundation support. In those areas where the existing fill soils extend below the planned footing elevation of 519 feet, the existing fill soils should either be overexcavated and replaced with structural fill, or the footings should be deepened to bear directly on the competent, native sediments. Structural fill placed below any footing should extend horizontally outward from the footing edges a distance equal to or greater than the thickness of the fill below the footing. The depths at which sediments suitable for foundation support were encountered in our explorations are summarized below in Table 1.

Table 1
Approximate Depth to Foundation Bearing Soils

Exploration Boring	Location*	Apx. Depth to Bearing Soils (feet)
EB-1	NW building corner	12
EB-2	NE building corner	0.5
EB-3	SW building corner	13
EB-4	SE building corner	5.5

*See Figure 2 for approximate boring locations.

We recommend that an allowable foundation soil bearing pressure of 2,500 pounds per square foot (psf) be utilized for design purposes, including both dead and live loads. Alternatively, a soil bearing pressure of 4,000 psf may be used provided all of the footings bear directly on the undisturbed, dense to very dense lodgement till. An increase of one-third may be used for short-term wind or seismic loading.

11.2 Footing Depths

Perimeter footings for the proposed building 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.

11.3 Footings Adjacent to Cuts

It should be noted that the area bounded by lines extending downward at 1H:1V from any footing must not intersect another footing or intersect a filled area that has not been compacted to at least 95 percent of ASTM:D 1557. In addition, a 1.5H:1V line extending down 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.

11.4 Footing Settlement

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.

11.5 Footing Subgrade Bearing Verification

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. ST

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 that 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 that cannot yield should be designed for an equivalent fluid of 50 pcf. If roadways, 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.

12.1 Wall Backfill

The lateral pressures presented above are based on the conditions of a uniform horizontal backfill consisting of either the on-site, natural glacial sediments, or imported sand and gravel compacted to 90 to 95 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 unacceptable settlement behind the walls. Thus, the compaction level is critical and must be tested by our firm during placement. The recommended compaction of 90 to 95 percent of ASTM:D 1557 applies to any structural fill placed behind the wall within a distance equal to the wall height and up to the elevation of the top of the wall. ST

12.2 Wall Drainage

Footing drains must 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 the ground surface using imported washed gravel against the walls placed to be continuous with the footing drain.

12.3 Passive Resistance and Friction Factors

Lateral loads can be resisted by friction between the foundation and the competent natural sediments or supporting structural fill soils and/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 allowable design parameters.

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

13.0 FLOOR SUPPORT

Slab-on-grade floors may be constructed either directly on the medium dense to very dense natural sediments or on structural fill placed over these materials. Areas of the slab subgrade that are disturbed (loosened) during construction should be recompacted to an unyielding condition prior to placing the pea gravel, as described below.

If moisture intrusion through slab-on-grade floors is to be limitec
constructed atop a capillary break consisting of a minimum thickness o
gravel. The pea gravel should be overlain by a 10-mil (minimum
retarder.

SOG floors
10 mil
vapor
barr.

14.0 DRAINAGE CONSIDERATIONS

14.1 Footing Drains

Permanent foundation walls should be provided with a drain at the base of 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 and the drain should be constructed with sufficient gradient to allow discharge away from the building.

14.2 Wall Drainage

All retaining walls should be lined with a minimum 12-inch-thick, washed gravel blanket or backfilled with free-draining fill to within 2 feet of the ground surface and be continuous with the footing drain.

If permanent foundation walls are cast directly against tem nage
should be provided to control moisture and prevent the buildu ainst
the wall. At a minimum, we recommend that a synthetic drai lrain
or Miradrain, be installed on the face of the soldier pile wa ould
then be covered with plastic sheeting (12-mil minimum thick nent.
The drainage medium should discharge to a permanent drain le or
outside of the permanent foundation wall. The drainage s igid,
perforated, PVC pipe fully enveloped in washed pea gravel.

wall drainage

14.3 Drainage System Discharge

The foundation/wall drainage system should be tightlined to a suitable point of discharge. Given the depth of the basement floor, it is possible that gravity drainage from this elevation to an approved point of discharge may not be feasible. In lieu of gravity discharge, we recommend that the footing drains be tightlined to a sump from which the collected water can be pumped. If a sump and pump system is used, it should be equipped with a backup generator system.

14.4 Surface Water Drainage

Roof and surface runoff should not be discharged into the foundation/wall drainage system, but should be handled by a separate, rigid, tightline drain. All storm water runoff must be tightlined into an approved storm water drainage system. In planning, exterior grades adjacent to walls should be sloped downward away from the structure to achieve surface drainage.

15.0 PROJECT DESIGN AND CONSTRUCTION MONITORING

Our recommendations are preliminary in that construction details have not been finalized at the time of this report. We are available to provide additional geotechnical consultation as the project design develops and possibly changes from that upon which this report is based. If significant changes in grading are made, 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 foundations 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 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

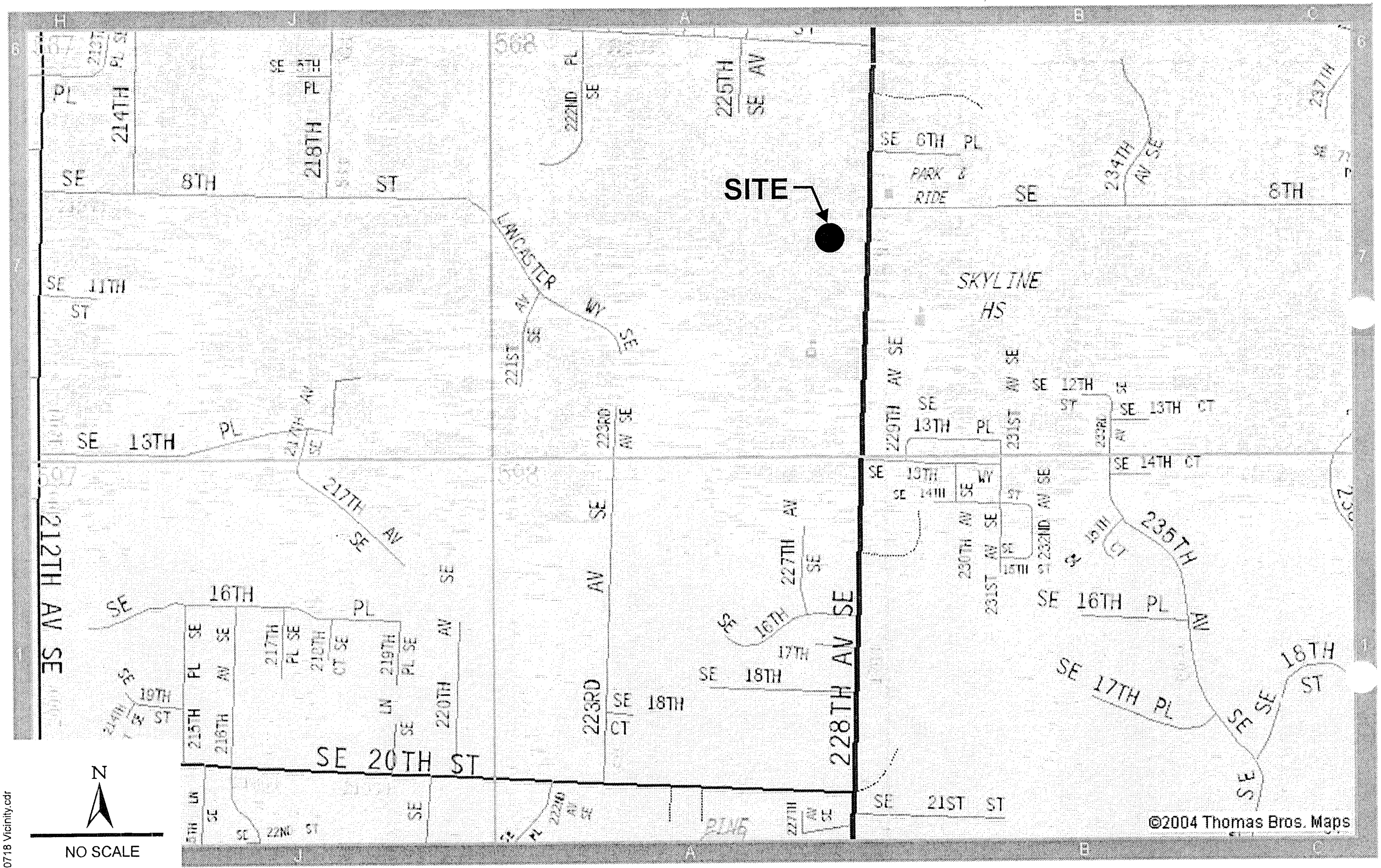
30284
EXPIRES 2/6/10
4-25-08

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

Matthew A. Miller, P.E.
Associate Engineer

Attachments: Figure 1: Vicinit
Figure lan
Appen

Pizza in
Frig,
For Thurs



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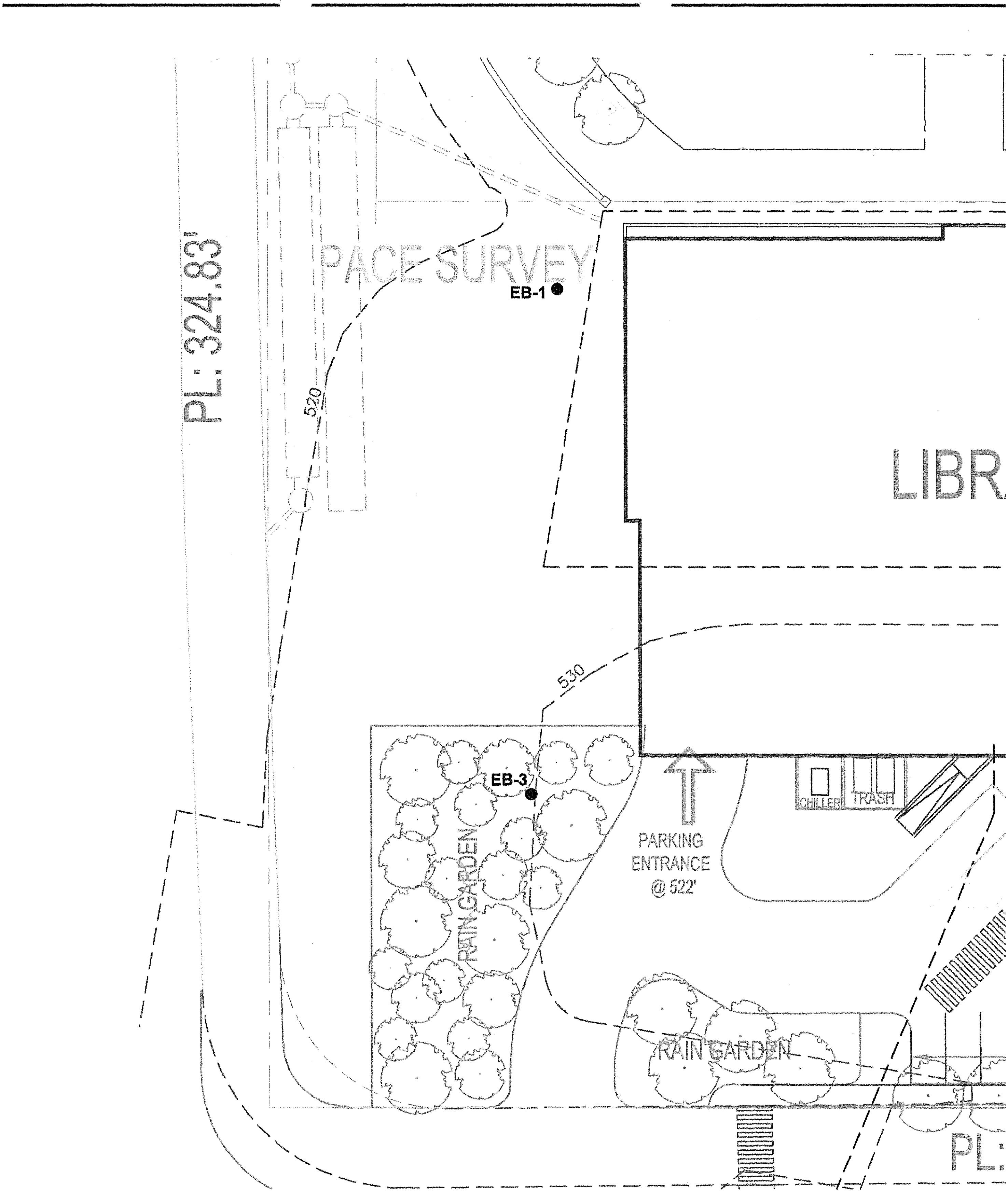
VICINITY MAP
SAMMAMISH LIBRARY
SAMMAMISH, WASHINGTON

FIGURE 1
DATE 11/07
PROJ. NO. KE070718A

070718 Sammamish Library \ 070718 Vicinity.cdr

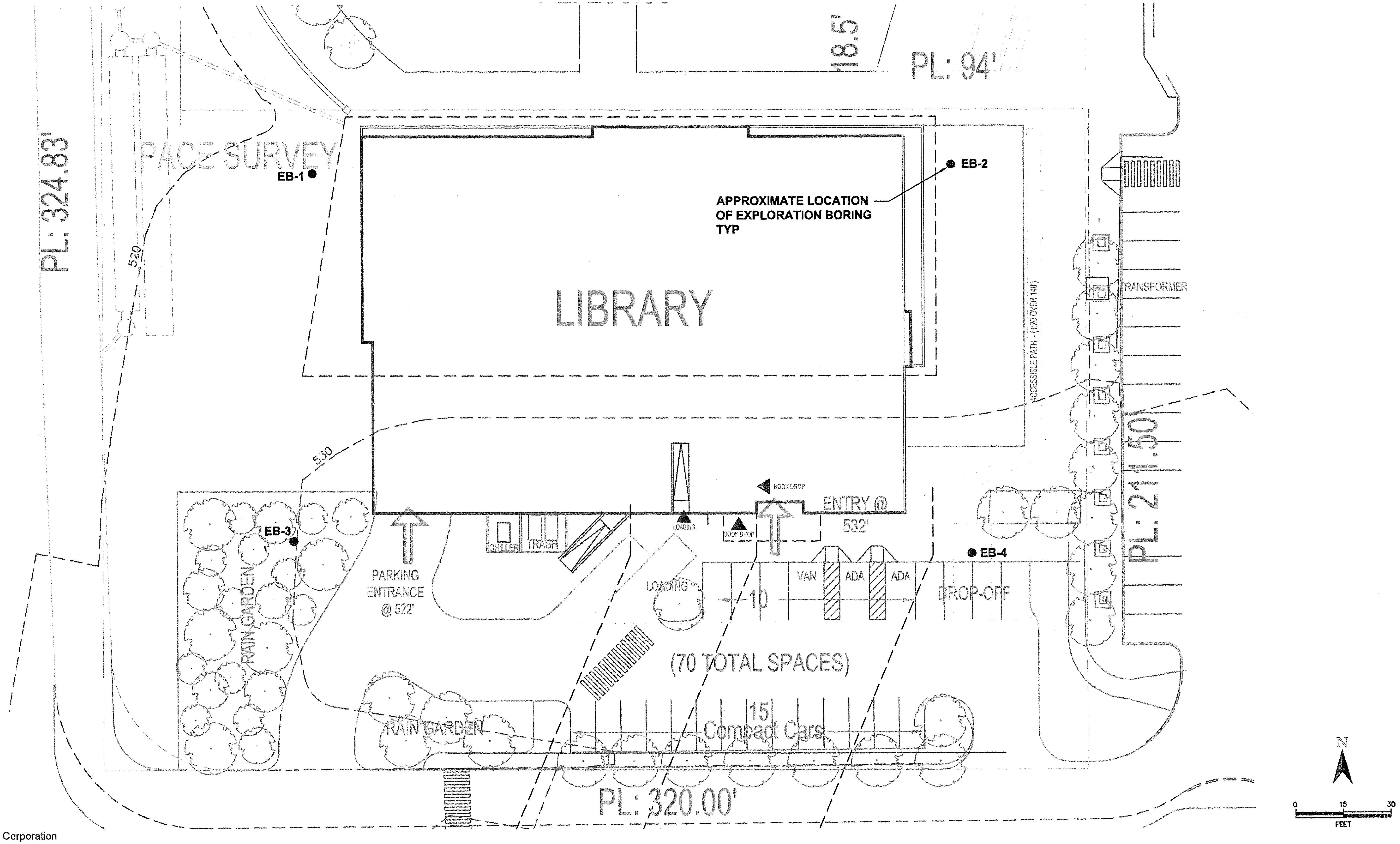


Reference: URS Corporation

Associated Earth Sciences, Inc.

SI

070718 Sammamish Library \ 070718 Site and Explr.dwg



Reference: URS Corporation

070718 Sammamish Library \ 070718 Site and Expl.dwg

Associated Earth Sciences, Inc.

SITE AND EXPLORATION PLAN
SAMMAMISH LIBRARY
SAMMAMISH, WASHINGTON

FIGURE 2

DATE 11/07

PROJECT NO. KE070718A

APPENDIX

Exploration Logs

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

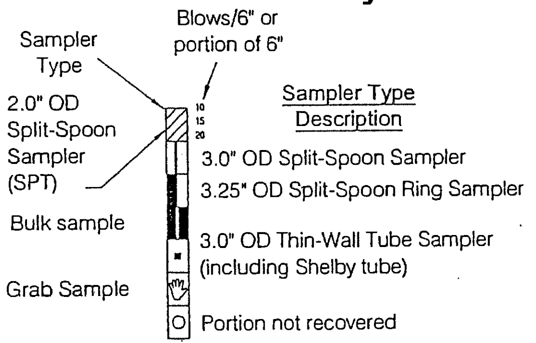
Slightly Moist - Perceptible moisture

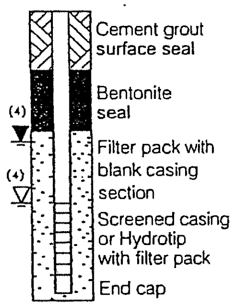
Moist - Damp but no visible water

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.

Associated Earth Sciences, Inc.

EXPLORATION LOG KEY

FIGURE A1

blockslog_key.dwg LAYOUT: Layout2

Associated Earth Sciences, Inc.

Exploration Log

Project Number	Exploration Number	Sheet
KE070718A	EB-1	1 of 1

Project Name: Sammamish Library
Location: Sammamish, WA
Driller/Equipment: Davies Drilling/Acker Rig
Hammer Weight/Drop: 140# / 30"
Ground Surface Elevation (ft): unknown
Datum: N/A
Date Start/Finish: 11/9/07,11/9/07
Hole Diameter (in): 7"

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
				Fill					
	▯	S-1		Wet, dark gray, silty SAND, with gravel, trace organics. slight oxidation.			1 1 4	▲5	
	▯	S-2		Very moist to wet, dark gray, silty SAND, with gravel, trace to few organics.			5 10 10	▲20	
5	▯	S-3		Very moist to wet, dark gray, silty SAND, with gravel, trace organics.			3 5 7	▲12	
	▯	S-4		Very moist to wet, gray, silty SAND, with gravel, trace organics.			2 3 2	▲5	
10	▯	S-5		Moist to very moist, bluish gray, with oxidation, silty SAND, with gravel. Moist, brown/gray, with oxidation, silty SAND, with gravel and driller notes harder drilling at 12 feet.			3 7 16	▲23	
				Vashon Lodgement Till					
15	▯	S-6		Moist, gray, with slight oxidation, silty SAND, with gravel.			50/6"	▲50/6"	
				Bottom of exploration boring at 15.5 feet Exploration terminated due to refusal.					
20									
25									
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: JDH
Approved by:

Associated Earth Sciences, Inc.

Exploration Log

Project Number	Exploration Number	Sheet
KE070718A	EB-2	1 of 1

Project Name Sammamish Library
Location Sammamish, WA
Driller/Equipment Davies Drilling/Acker Rig
Hammer Weight/Drop 140# / 30"

Ground Surface Elevation (ft) unknown
Datum N/A
Date Start/Finish 11/9/07,11/9/07
Hole Diameter (in) 7"

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot 10 20 30 40	Other Tests
				Topsoil with grass.					
		S-1		**Vashon Lodgement Till** Moist, brown/gray, with oxidation, silty SAND, with gravel.			10 9 10	19	
		S-2		Moist, brown, with oxidation, silty SAND, with gravel.			20 36 50/6"	50/6"	
5		S-3		As above.			50/6"	50/6"	
				Moist, bluish gray, silty SAND, with gravel and driller encounters cobbles at 6 feet bgs.					
		S-4		Rock at 7.5 feet bgs. No sample recovered. Driller encounters additional cobbles at 9 feet bgs.			100/0"	100/0"	
10		S-5		As above.			50/4.5"	50/4.5"	
				Bottom of exploration boring at 10.5 feet Exploration terminated due to refusal. Note: An initial attempt to drill this boring met with refusal on a large cobble or boulder at a depth of approximately 7 feet. The initial boring was abandoned and re-drilled approximately 3 feet south of the initial location.					
15									
20									
25									
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: JDH
Approved by:

Associated Earth Sciences, Inc.

Explorati Log

Project Number	Exploration Number	Sheet
KE070718A	EB-3	1 of 1

Project Name Sammamish Library
Location Sammamish, WA
Driller/Equipment Davies Drilling/Acker Rig
Hammer Weight/Drop 140# / 30"

Ground Surface Elevation (ft) unknown
Datum N/A
Date Start/Finish 11/9/07,11/9/07
Hole Diameter (in) 7"

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot 10 20 30 40	Other Tests
				Fill					
		S-1		Loose, very moist to wet, brown/gray, silty SAND, with gravel, trace organics, driller notes smooth/easy drilling.			1 1 2	▲3	
		S-2		Very moist, dark brown, silty SAND, little gravel, few organics.			6 5 7	▲12	
5		S-3		Very moist, dark brown/gray, silty SAND, little gravel, trace organics, slight oxidation.			3 3 3	▲6	
		S-4		As above.			3 3 2	▲5	
10		S-5		Very moist to wet, dark brown/gray, silty SAND, little to with gravel, few organics, driller notes harder drilling at 13 feet bgs.			2 6 3	▲9	
				Vashon Lodgement Till					
15		S-6		Moist, brown/gray, silty SAND, with gravel.			10 20 29	▲49	
20		S-7		Moist, gray/light brown, silty SAND, with gravel, slight oxidation, 6 inches recovery/driller notes rock.			50/4.5"	▲50/4.5"	
				Bottom of exploration boring at 20.5 feet					
25									
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: JDH
Approved by:

Associated Earth Sciences, Inc.

Exploration Log

Project Number	Exploration Number	Sheet
KE070718A	EB-4	1 of 1

Project Name: Sammamish Library
Location: Sammamish, WA
Driller/Equipment: Davies Drilling/Acker Rig
Hammer Weight/Drop: 140# / 30"
Ground Surface Elevation (ft): unknown
Datum: N/A
Date Start/Finish: 11/9/07,11/9/07
Hole Diameter (in): 7"

Depth (ft)	S T	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
		S-1		**Fill** Very moist to wet, brown and dark gray, silty SAND, with gravel (crushed rock). Moist, gray, with oxidation, silty SAND, with gravel (subrounded).			7 8 13	▲21	
		S-2		Moist, light gray/brown, with oxidation, silty SAND, with gravel (subrounded).			13 14 20	▲34	
5		S-3		As above, slightly less oxidation, driller encounters cobble at 6 feet bgs.			13 19 27	▲46	
		S-4		**Vashon Lodgement Till** Moist, gray/brown, silty SAND, with gravel.			22 50/5"	▲50/5"	
10		S-5		As above. Moist, bluish gray, fine to medium SAND, little gravel, few silt and sand.			35 50/4"	▲50/4"	
15		S-6		Very moist to wet, bluish gray, fine to medium SAND, with gravel, trace to few silt.			50/5"	▲50/5"	
20		S-7		As above, friction heat likely evaporating some moisture, driller encounters cobble at 21 feet bgs.			50/5"	▲50/5"	
25		S-8		Moist, bluish gray, silty SAND, little gravel. Drilling action smooths at 26 feet bgs.			40 50/2.5"	▲50/2.5"	
30		S-9		As above.			50/5"	▲50/5"	
35		S-10		As above, edges of sample wet from shallow water in boring, driller notes shallow water entering bore hole.			29 50/5"	▲50/5"	
40		S-11		No recovery. Bottom of exploration boring at 40.5 feet			50/6"	▲50/6"	

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: JDH
Approved by:

Associated Earth Sciences, Inc.

Celebrating Over 25 Years of Service

May 13, 2008
Project No. KE070718A

FILE COPY

King County Library System
c/o URS Corporation
1501 4th Avenue, Suite 1400
Seattle, Washington 98101

RECEIVED BY PERMIT CENTER
MAY 14 2008
CITY OF SAMMAMISH

Attention: Mr. Ross Pouley

Subject: Geotechnical Plan Review
Sammamish Library
SE 8th Street and 228th Avenue SE
Sammamish, Washington

Dear Mr. Pouley:

Associated Earth Sciences, Inc. (AESI) has performed a geotechnical review of the current design plans for the proposed new residence located at the above-referenced site. These plans include:

- Sheet S-101, prepared by Coughlin Porter Lundeen, consisting of *General Structural Notes*, dated February 4, 2008;

- Sheets S-201, S-301, S-302, and S-303 prepared by Coughlin Porter Lundeen, consisting of *Parking Level/Foundation Plan, Typical Concrete Details, Foundation Details,* and *Concrete Details*, dated May 12, 2008; and,

- Sheets C-100, C-110, C-200, C-300, C-400, C-500 prepared by Coughlin Porter Lundeen, consisting of *Demolition and TESC Plan, TESC Details, Grading and Paving Plan, Storm Drainage Plan, Water Plan,* and *Sewer Plan.*

Upon completion of our review, we offer the following comments:

- Note No. 12 on Sheet S-101 indicates an allowable soil pressure of 4,000 pounds per square foot (psf). This note also states that "Footings shall bear on firm, undisturbed earth or controlled, compacted structural fill...". For the 4,000 psf bearing pressure to apply, all footings must bear directly on the natural, dense to very dense, unweathered lodgement till sediments (not on structural fill).

Kirkland ▪ Everett ▪ Tacoma
425-827-7701 425-259-0522 253-722-2992
www.aesgeo.com

B08·00470

- Sheet C-100 shows the wheel wash adjacent to the rock construction entrance, close to the road. In its current position, it may be difficult for large trucks exiting the site to properly access the wheel wash and still make the corner out onto the street across the rock pad. The plan also shows two wood barrier rails blocking access between the wheel wash and the rock pad. We have discussed this matter with the project engineer who suggested moving the wheel wash back from the road sufficiently to correct this problem. We agree that this would correct the problem.

Provided the design plans are amended as noted above, it is our opinion that the plans reviewed by us generally conform to the recommendations provided in our November 29, 2007 geotechnical engineering report.

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

30284
5-13-08
EXPIRES 2/6/10

Timothy J. Peter

Timothy J. Peter, P.E.G.
Senior Project Geologist

Matthew A. Miller, P.E.
Associate Engineer

cc: Mr. Gavin Smith/Perkins+Will
gavin.smith@perkinswill.com

Mr. Keith Kuger, P.E./Coughlin Porter Lundeen
keithk@cplinc.com

TJP/ld
KE070718A7
Projects\20070718\KE\WP

MOVEMENT SUMMARY

Site: 228th Ave SE and SE 10th St-2030PM

228th Ave SE and SE 10th St
Year 2030 PM Peak Period
Roundabout

Movement Performance - Vehicles

Mov ID	Turn	Demand Flow veh/h	HV %	Deg. Satn v/c	Average Delay sec	Level of Service	95% Back of Queue Vehicles veh	95% Back of Queue Distance ft	Prop. Queued	Effective Stop Rate per veh	Average Speed mph
South: 228th Ave SE (NB)											
3L	L	107	1.0	0.762	22.0	LOS C	7.2	181.8	0.84	1.12	13.7
8T	T	943	1.0	0.762	22.0	LOS C	7.2	181.8	0.84	1.03	13.7
8R	R	105	1.0	0.762	22.0	LOS C	7.2	181.8	0.84	1.06	13.5
Approach		1155	1.0	0.762	22.0	LOS C	7.2	181.8	0.84	1.04	13.7
East: SE 10th St (WB)											
1L	L	182	1.0	0.589	23.3	LOS C	2.2	55.0	0.80	1.00	6.6
6T	T	1	1.0	0.589	23.3	LOS C	2.2	55.0	0.80	0.99	11.5
6R	R	62	1.0	0.589	23.3	LOS C	2.2	55.0	0.80	0.99	6.1
Approach		245	1.0	0.589	23.3	LOS C	2.2	55.0	0.80	0.99	6.5
North: 228th Ave SE (SB)											
7L	L	204	1.0	0.696	17.0	LOS C	6.0	150.8	0.73	1.00	17.7
4T	T	822	1.0	0.696	17.0	LOS C	6.0	150.8	0.73	0.86	18.9
4R	R	134	1.0	0.696	17.0	LOS C	6.0	150.8	0.73	0.90	18.7
Approach		1161	1.0	0.696	17.0	LOS C	6.0	150.8	0.73	0.89	18.6
West: SE 10th St (EB)											
5L	L	179	1.0	0.909	50.9	LOS F	6.8	170.7	0.93	1.68	4.6
2T	T	2	1.0	0.909	50.9	LOS F	6.8	170.7	0.93	1.68	3.0
2R	R	251	1.0	0.909	50.9	LOS F	6.8	170.7	0.93	1.68	3.6
Approach		432	1.0	0.909	50.9	LOS F	6.8	170.7	0.93	1.68	4.1
All Vehicles		2993	1.0	0.909	24.4	LOS C	7.2	181.8	0.81	1.07	13.0

Level of Service (LOS) Method: Delay & v/c (HCM 2010).
Roundabout LOS Method: Same as Sign Control.
Vehicle movement LOS values are based on average delay and v/c ratio (degree of saturation) per movement
LOS F will result if v/c > 1 irrespective of movement delay value (does not apply for approaches and intersection).
Intersection and Approach LOS values are based on average delay for all movements (v/c not used as specified in HCM 2010).
Roundabout Capacity Model: US HCM 2010.
HCM Delay Model used.

Processed: Monday, May 02, 2011 10:08:28 AM
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8000011, DAVID EVANS & ASSOCIATES INC, SINGLE

center. These are based on the community center being constructed at the Kellman site right here behind City Hall.

16. Which of the following three payment/membership fee models would you prefer?

17. Based on the information presented in the slides, would you be more likely to pay daily fees to use the community center or membership fees?

18. Would you be more likely to purchase a three month or annual pass? (Some explanation needed here that shorter term passes or daily passes may increase the subsidy required). Would be nice to know if that would impact their recommendation overall, although I'd still like to know their personal preference.)

19. Having now seen the influence certain spaces of the community center have on revenue, would you be likely to change your priorities from earlier in the discussion?

POTENTIAL COSTS

20. What are your initial reactions to the two preliminary options (levy or utility tax) for paying for the potential community center (i.e. are they too expensive, not expensive, about right)? (May need to explain the difference between the two funding options).

21. Which range do you prefer for the cost of the potential community center (i.e. $30 to $40 million)?

22. Having now seen the cost for each space of the community center, which spaces do you believe are less important that the potential community center could do without?

23. Has the price ranged changed now that you've seen the costs for the spaces?