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CITY OF MONROE
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ENGINEERING DEPARTMENT

July 19, 2012
Project No. KE120241A

H3O Development
25877 SE 22nd Place
Sammamish, Washington 98075

Attention: Mr. Brad Smith

City of Monroe
806 West Main Street
Monroe, Washington 98272

Attention: Mr. Brad Fielberg, Public Works Director

Subject: Limited Subsurface Exploration and
Geotechnical Engineering Recommendations
Lake Tye Cable Wakeboard Park Building and Tower Foundations
Fryelands Boulevard at Lake Tye Park
Monroe, Washington

Dear Mr. Smith and Mr. Fielberg:

This letter-report presents the results of our limited subsurface exploration program, and geotechnical engineering recommendations for the construction of a cable wakeboarding facility in Lake Tye in Monroe, Washington. Our work was performed in general accordance with our scope of work and cost proposal letter dated June 12, 2012. Authorization to proceed with this study was granted by means of signed proposal from the City of Monroe and H3O Development. Our work has been performed in accordance with local standards of practice in the field of geotechnical engineering at the time it was completed. No other warranty, express or implied, is made. This letter-report has been prepared for the exclusive use of the City of Monroe, H3O Development, and their agents for specific application to this project. The site location is shown on the "Vicinity Map," Figure 1; the approximate locations of the exploration borings completed for this study are shown on the "Site and Exploration Plan," Figure 2.

EXHIBIT# M5

BACKGROUND

The proposed project would include the construction of a shore building and a cable wakeboarding facility in Lake Tye in Monroe, Washington. The shore building would be located adjacent to the existing boat launch, skateboard park, and parking lot in Lake Tye Park at the south end of Lake Tye. The building would provide administrative offices, ticketing, and storage. The building would be located at the shoreline and would be connected to viewing platforms and a start/finish dock in the lake. The moving pull cable will be supported by a series of lattice towers founded on prefabricated bases sitting on the lake bottom. The towers will be supported by guy wires anchored on the shore.

SUBSURFACE EXPLORATION

We completed two subsurface exploration borings at the shore building location. The boring locations were limited to accessible areas on shore. We did not attempt to explore locations on the slope down to the lake shore. During the drilling process, samples were obtained at generally 2.5- to 5-foot-depth intervals. The exploration borings were continuously observed and logged by a representative from our firm. The exploration logs attached to this letter-report are based on field notes, drilling action, and inspection of the samples collected from the borings.

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

The samples obtained from the split-barrel sampler were classified in the field and representative portions placed in watertight containers. The samples were then transported to our laboratory for further visual classification. Upon completion of our exploration borings, we abandoned the borings in accordance with Washington State Department of Ecology (Ecology) guidelines using bentonite.

SUBSURFACE CONDITIONS

Fill

Both explorations encountered existing fill soils. Fill depths were generally observed to be approximately 7 feet at both EB-1 and EB-2. Fill consisted of brown silty sand and brown sand with gravel, few silt. Fill soils were generally loose to medium dense and very moist.

Recessional Outwash

Below the fill in all borings, the soils generally consisted of dense to very dense, wet brown sand with varying amounts of gravel. This material is interpreted to represent recessional outwash deposits. Meltwater streams emanating from the retreating Vashon age glacial ice sheet that occupied the Puget Sound region deposited this material. Recessional outwash is typically loose to medium dense. The high recorded blow counts on the attached exploration boring logs for the outwash are due to the coarse nature of the deposit and may not be representative of the actual density of these soils. With proper preparation, these outwash deposits are suitable for support of the proposed structure. Excavated recessional outwash sediments are suitable for use as structural fill if they are at a moisture content that allows compaction to a firm and unyielding condition and the specified minimum compaction level.

Published Geologic Map

Our interpretations of subsurface conditions on-site are generally consistent with a published geologic map of the area, as represented by the *Geologic Map of the Maltby Quadrangle, Snohomish and King Counties, Washington,* by J.P. Minard (1985). The referenced map indicates that the project area is expected to be underlain at shallow depth by recent alluvium. The mapped alluvium overlies older outwash deposits, into which our borings penetrated.

Hydrology

Free ground water was encountered in both borings near the contact between the existing fill and underlying recessional outwash deposits, approximately 7 feet in depth. The depth of ground water is likely related to the regional water table which is also reflected by the water level of the adjacent Lake Tye. Ground water levels should be expected to change in response to lake level, weather, season, changes in local land use, and other factors.

LAKE BOTTOM SEDIMENTS

To estimate the consistency of the lake bottom sediments, we probed the bottom of the lake. The probe consisted of three sections of 10-foot-long, 1.5-inch-diameter, Schedule 80 polyvinyl chloride (PVC) pipe screwed together into one 30-foot-long pipe. This probe was advanced from a boat at the approximate tower locations. The towers were located by determining the distance of each of the six towers from two survey nails located in the asphalt path east of Lake Tye as measured on the project plans. While on the lake, distance from the two survey nails was determined by GPS.

The water depth ranged from 26 to 28 feet at the probed locations. The shallow lake bottom sediments were soft and, based on probing action, appeared to be mostly fine-grained. No sand and gravel was encountered. The top 1 to 2 feet of sediments were generally very soft and then gradually firmed up for total penetrations of 2 to 4 feet. After penetrating to depths of 2 to 4 feet, probe resistance typically moved the boat away from the probe locations, limiting the ability to probe deeper. Probe penetration was typically not limited by high resistance from the pond bottom sediments to further penetration of the probe. The table below details the probe penetration depths and water depths at each tower location (as numbered on the attached plan, Figure 2).

	Penetration (feet)	**Water Depth (feet)**
Tower 1	2	26
Tower 2	3-4	27
Tower 3	3-4	27
Tower 4	2	28
Tower 5	4	28
Tower 6	4	27

RECOMMENDATIONS

Site Preparation

Existing buried utilities, vegetation, topsoil, existing fill, and any other deleterious materials should be removed where they are located below planned onshore construction areas. All disturbed soils resulting from demolition activities should be removed to expose underlying undisturbed native sediments and replaced with structural fill, as needed.

The observed fill depth in our borings was up to approximately 7 feet below existing grade just northeast of the existing parking lot and north of the existing skate park. We recommend that existing fill be removed from below areas of planned foundations to expose underlying undisturbed native sediments, followed by restoration of the planned foundation grade with structural fill. Removal of existing fill should extend laterally beyond the building footprint or accessory structure foundation element by a distance equal to the depth of overexcavation. For example, if existing fill is removed to a depth of 2 feet below a planned footing area, the excavation should also extend laterally 2 feet beyond the building footprint in that area. Where existing fill is removed and replaced with structural fill, conventional shallow foundations may be used for foundation support. Other foundation support alternatives are discussed later in this letter-report, which may alleviate the need to remove all existing fill below the buildings.

In our opinion, stable, temporary construction slopes should be the responsibility of the contractor and should be determined during construction. For planning purposes, we anticipate that temporary, unsupported cut slopes in unsaturated existing fill or natural sediments can be made at a maximum slope of 1.5H:1V (Horizontal:Vertical). Flatter temporary cut slopes are recommended in areas of ground water seepage. As is typical with earthwork operations, some sloughing and raveling may occur, and cut slopes may have to be adjusted in the field. In addition, WISHA/OSHA regulations should be followed at all times. Permanent, unsupported cut or structural fill slopes should not exceed a gradient of 2H:1V. Substantial excavations below static ground water levels should not be attempted.

Structural Fill

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

After stripping, planned excavation, and any required overexcavation have been performed to the satisfaction of the geotechnical engineer, the upper 12 inches of exposed ground in areas to receive fill should be recompacted to 95 percent of the modified Proctor maximum density using ASTM:D 1557 as the standard. If the subgrade contains silty soils and too much moisture, adequate recompaction may be difficult or impossible to obtain and should probably not be attempted. In lieu of recompaction, the area to receive fill should be blanketed with washed rock or quarry spalls to act as a capillary break between the new fill and the wet subgrade. Where the exposed ground remains soft and further overexcavation is impractical, placement of an engineering stabilization fabric may be necessary to prevent contamination of the free-draining layer by silt migration from below.

After recompaction of the exposed ground is tested and approved, or a free-draining rock course is laid, structural fill may be placed to attain desired grades. Structural fill is defined as non-organic soil, acceptable to the geotechnical engineer, placed in maximum 8-inch loose lifts, with each lift being compacted to 95 percent of the modified Proctor maximum density using ASTM:D 1557 as the standard. In the case of roadway and utility trench filling, the backfill should be placed and compacted in accordance with current City of Monroe codes and standards. The top of the compacted fill should extend horizontally outward a minimum distance of 3 feet beyond the locations of the roadway edges before sloping down at an angle of 2H:1V.

The contractor should note that any proposed fill soils must be evaluated by Associated Earth Sciences, Inc. (AESI) prior to their use in fills. This would require that we have a sample of the material 72 hours in advance to perform a Proctor test and determine its field compaction standard. Soils in which the amount of fine-grained material (smaller than the No. 200 sieve) is greater than approximately 5 percent (measured on the minus No. 4 sieve size) should be considered moisture-sensitive. Use of moisture-sensitive soil in structural fills should be limited to favorable dry weather conditions. The existing fill soils present on-site contained significant amounts of silt and are considered highly moisture-sensitive. The native soils present on-site were typically sand and gravel, containing free ground water, and while less moisture-sensitive than the existing fill soils, are considered moderately moisture-sensitive. If fill is placed during wet weather or if proper compaction cannot be obtained, a select import material consisting of a clean, free-draining gravel and/or sand should be used. Free-draining fill consists of non-organic soil with the amount of fine-grained material limited to 5 percent by weight when measured on the minus No. 4 sieve fraction with at least 25 percent retained on the No. 4 sieve.

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

Foundations

The subsurface explorations completed for this study encountered fill depths up to 7 feet thick. Depending on foundation locations and elevations, special foundation support alternatives may be needed. Possible approaches include removal of all existing fill below buildings and replacement with structural fill and installation of rock-filled trenches. If design foundation elevation is deep enough to penetrate most of the existing fill, the exposed base of the excavation can be compacted to 95 percent relative compaction and the new footings constructed as planned. The following

sections provide preliminary design values for these two approaches. A third foundation support alternative would be small-diameter driven pipe piles (pin piles). We are available to provide pin pile design and installation recommendations if this type of building and dock support alternative is selected. The final design approach should be based on situation-specific recommendations that we can provide once a preliminary design has been developed.

Shallow Foundations

Spread footings may be used for building support when founded directly on undisturbed native sediments, a limited thickness (less than 2 feet) of recompacted existing fill, or on structural fill placed above such suitable native deposits. Foundations supported on undisturbed native recessional outwash deposits or structural fill above suitable native sediments should be designed with an allowable foundation soil bearing pressure of 2,500 pounds per square foot (psf). Perimeter footings should be buried at least 18 inches into the surrounding soil for frost protection. However, all footings must penetrate to the prescribed bearing stratum, and no footing should be founded in or above organic or loose soils. All footings should have a minimum width of 18 inches.

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

Anticipated settlement of footings founded as described above should be on the order of ¾ inch or less. However, disturbed soil not removed from footing excavations prior to footing placement could result in increased settlements. All footing areas should be inspected by AESI prior to placing concrete to verify that the design bearing capacity of the soils has been attained and that construction conforms to the recommendations contained in this letter-report. Such inspections may be required by the City of Monroe.

Rock-Filled Trenches

In areas of existing fill, an alternative to removal and replacement of all existing fill would be the use of rock-filled trenches. Rock-filled trenches are trenches excavated below planned foundation areas, backfilled with compacted crushed rock. If needed, rock-filled trenches below foundations should extend laterally beyond all foundation elements by at least 1 foot, and should have a minimum width of 4 feet (or as designated by the field engineer/engineering geologist). All rock-filled trenches should be excavated down to expose underlying, native sediments. Because of the potential for caving, the actual trench width may be greater than specified. It would be

appropriate to backfill the trenches as the excavation proceeds to reduce caving. The use of a larger, track-mounted backhoe will greatly speed trench excavation over the use of a conventional rubber-tired backhoe. In order to reduce disturbance of the bearing soils exposed in the trench, we recommend that the teeth of the backhoe bucket be covered with a digging plate.

To determine when suitable bearing has been achieved and to verify proper rock placement, the geotechnical engineer/engineering geologist must be present on a full-time basis during footing trench excavation and backfill. After the bearing stratum has been reached, the trench should be immediately backfilled. We recommend the use of "railroad ballast" or 2- to 4-inch-size crushed rock for backfill. The crushed rock must be tamped into place to achieve a tightly packed mass; this may be done with either a "Hoepac"-type compactor mounted on the backhoe, or more typically, with the bucket of the backhoe itself. Staging areas should be maintained so that the rock is not contaminated by mud prior to placement in the trench. Equipment access to trench locations should also be maintained. Upon successful completion of rock trenches, conventional shallow foundations may be constructed as described above, with allowable foundation soil bearing pressures of 2,500 psf assuming ½ inch of allowable settlement.

Drainage Considerations

Perimeter footings should be provided with foundation drains. Drains should consist of rigid, perforated PVC pipe surrounded by washed pea gravel. The drains should be constructed with sufficient gradient to allow gravity discharge away from the proposed building. Roof and surface runoff should not discharge into the footing drain system, but should be handled by a separate, rigid, tightline drain.

Floor Support

Floor slabs can be supported on suitable native sediments or on structural fill placed above suitable native sediments. Floor slabs should be cast atop a minimum of 4 inches of clean, washed, crushed rock or pea gravel to act as a capillary break. Areas of subgrade that are disturbed (loosened) during construction should be compacted to a firm and unyielding condition prior to placement of capillary break material. Floor slabs should also be protected from dampness by an impervious moisture barrier at least 10 mils thick. The moisture barrier should be placed between the capillary break material and the concrete slab.

Pavement Recommendations

Pavement and sidewalk areas should be prepared in accordance with the "Site Preparation" section of this letter-report. If the stripped native soil or existing fill pavement subgrade can be compacted to 95 percent of ASTM:D 1557 and is firm and unyielding, no additional overexcavation is

required. Soft or yielding areas should be overexcavated to provide a suitable subgrade and backfilled with structural fill.

Foundation Walls

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

As required by the 2009 *International Building Code* (IBC), retaining wall design should include a seismic surcharge pressure in addition to the equivalent fluid pressures presented above. Considering the site soils and the recommended wall backfill materials, we recommend a seismic surcharge pressure of 5H and 10H psf, where H is the wall height in feet for the active and at-rest loading conditions, respectively. The seismic surcharge should be modeled as a rectangular distribution with the resultant applied at the midpoint of the walls.

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

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

Lateral loads can be resisted by friction between the foundation and the natural soils or supporting structural fill soils, and by passive earth pressure acting on the buried portions of the foundations. The foundations must be backfilled with structural fill and compacted to at least 95 percent of the

maximum dry density to achieve the passive resistance provided below. We recommend the following allowable design parameters:

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

Geologic Hazards

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

We understand the project will be designed under the 2009 IBC. Under the 2009 IBC, sites underlain by liquefaction-prone soils default to a Site Class "F", which requires the development of a site-specific seismic response spectrum. The 2006 IBC contained a procedure for bypassing the development of a site-specific response spectrum provided the fundamental building period of the building was less than ½ second (typically associated with conventional buildings less than about five stories in height). This procedure has been removed from the 2009 IBC. To comply with the 2009 IBC code with respect to site classification "F" requires collaboration with the project structural engineer. Through the collaborative process, the site classification can be changed from "F" to "C", "D", or "E", depending upon building type, height, fundamental period of vibration, and occupancy classification. At this point in the project, the information required to work through the 2009 IBC Site Class "F" reclassification procedure was not available. Once the project structural engineer has established the required parameters, the reclassification procedure can be completed.

CLOSURE

We appreciate the opportunity to be of service on this project. We are available to answer any questions you may have regarding this letter-report. Construction observation and testing services

are not part of our currently approved scope of work. We would be pleased to provide a scope of work and cost proposal for construction phase services on request.

Sincerely,
ASSOCIATED EARTH SCIENCES, INC.
Kirkland, Washington

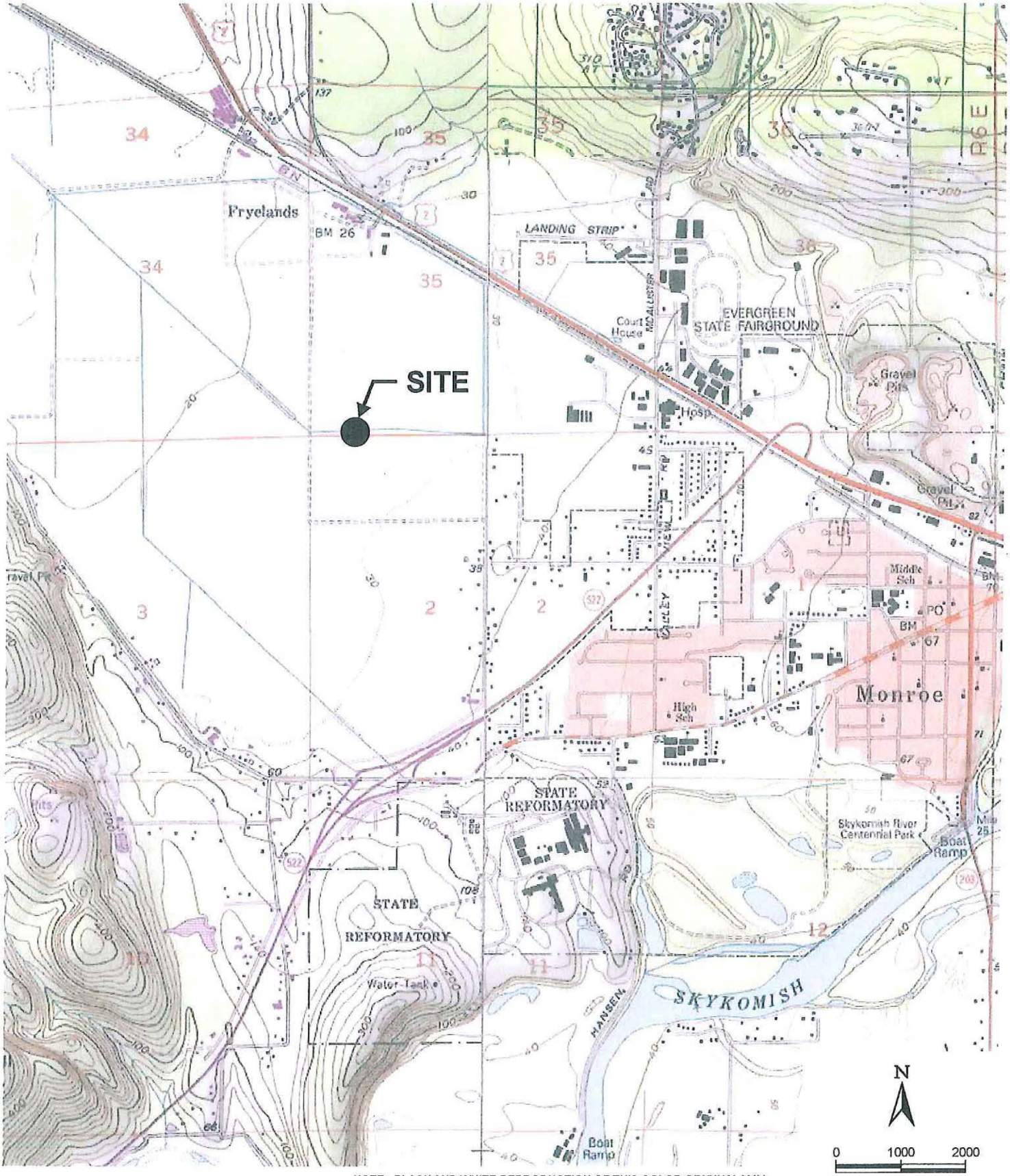
KURT D. MERRIMAN
STATE OF WASHINGTON
23580
REGISTERED
PROFESSIONAL ENGINEER
7/19/12

7/19/12

Luke Mioduszewski
Senior Staff Geologist

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

Attachments: Figure 1: Vicinity Map
Figure 2: Site and Exploration Plan
Subsurface Exploration Logs EB-1 and EB-2



REFERENCE: USGS TOPO!

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

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VICINITY MAP

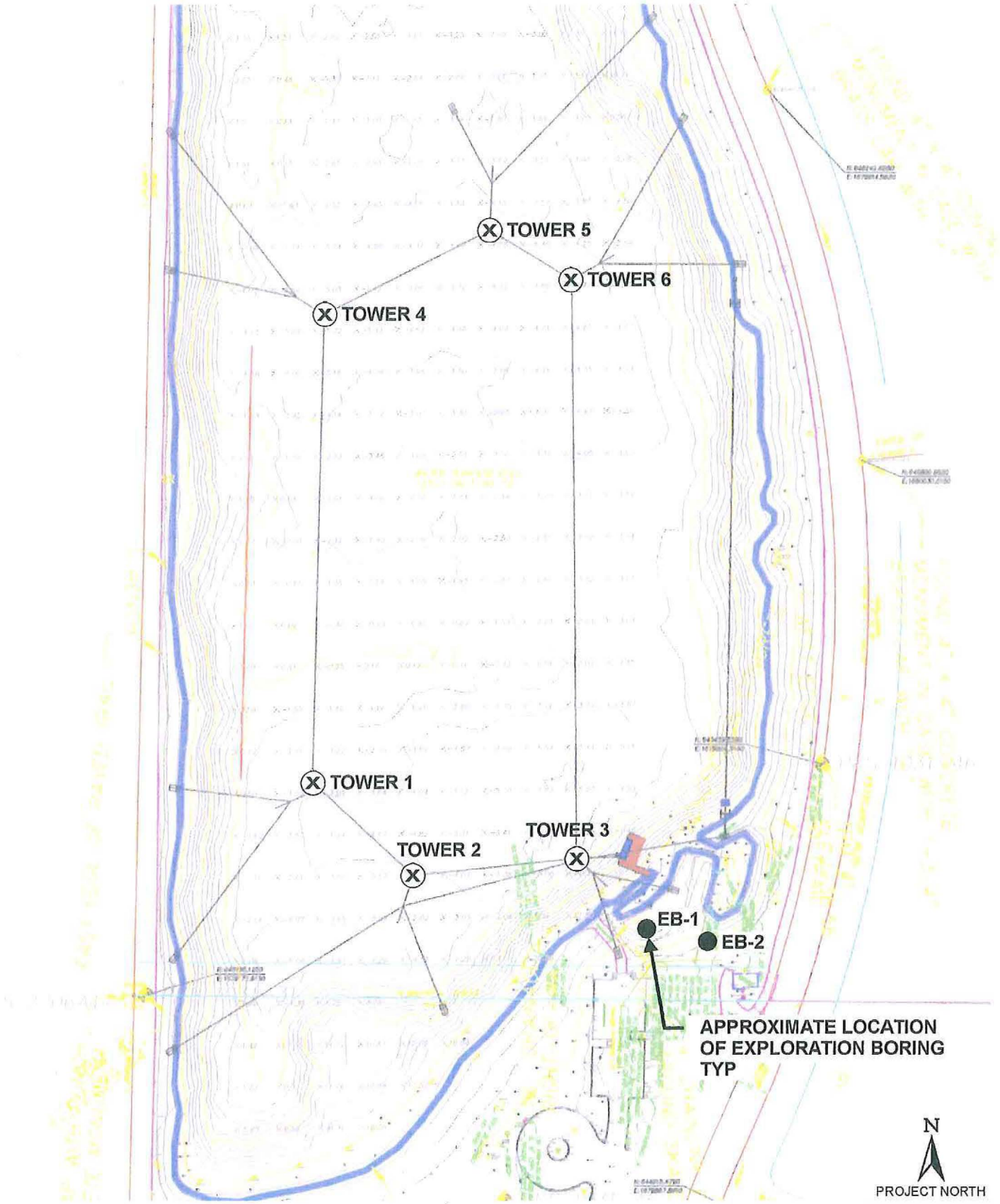
MONROE CABLE WAKEBOARD PARK
MONROE, WASHINGTON

FIGURE 1

DATE 7/12

PROJ. NO. KE120241A

120241 Monroe Wakeboard Park \ 120241 Vicinity.cdr



REFERENCE: SESITEC LIVING WATER

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

Associated Earth Sciences, Inc.

PARTIAL SITE AND EXPLORATION PLAN

MONROE CABLE WAKEBOARD PARK
MONROE, WASHINGTON

FIGURE 2
DATE 7/12
PROJ. NO. KE120241A

120241 Monroe Wakeboard Park \ 120241 Site and Explr.cdr

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

Coarse-Grained Soils

Density	SPT(2) blows/foot
Very Loose	0 to 4
Loose	4 to 10
Medium Dense	10 to 30
Dense	30 to 50
Very Dense	>50

Fine-Grained Soils

Consistency	SPT(2) blows/foot
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

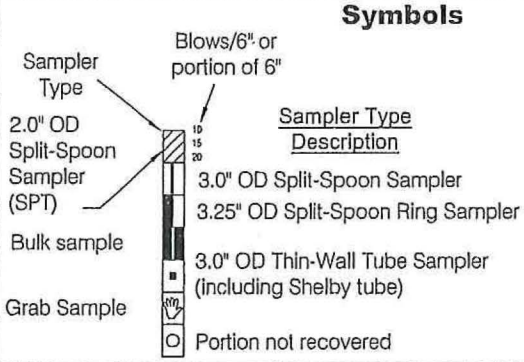
Slightly Moist - Perceptible moisture

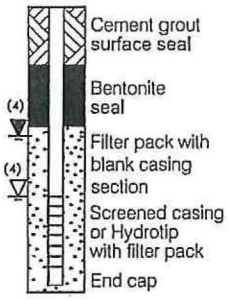
Moist - Damp but no visible water

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.

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EXPLORATION LOG KEY

FIGURE A1

blockslog_key.dwg LAYOUT: Layout2

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Exploration Log

Project Number	Exploration Number	Sheet
KE120241A	EB-1	1 of 1

Project Name	Lake Tye Cable Wakeboard Park	Ground Surface Elevation (ft)	~30
Location	Monroe, WA	Datum	N/A
Driller/Equipment	Geologic Drill / XL	Date Start/Finish	7/2/12,7/2/12
Hammer Weight/Drop	140# / 30"	Hole Diameter (in)	6 inches

Depth (ft)	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
				Topsoil					
				Fill					
		S-1		Loose, very moist, dark brown, silty fine to medium SAND, trace to few gravel, organics (SM). Recovery 8 inches.			5 4 4	▲8	
5		S-2		Medium dense, very moist, brown to blue gray, silty fine to coarse SAND, few gravel, organics (SM). Recovery 18 inches. Driller reports gravel at 7 feet.		▼	3 5 6	▲11	
				Vashon Recessional Outwash					
10		S-3		Very dense, wet, brown, fine to coarse SAND, with gravel, trace silt; gradational stratification, blowcounts overstated due to gravel (SW).			16 30 25	▲55	
15		S-4		Dense, wet, brown, fine to coarse SAND, with gravel; gradational stratification, blowcounts overstated due to gravel (SW).			15 17 16	▲33	
				Bottom of exploration boring at 16.5 feet					
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: LDM

Approved by:

Associated Earth Sciences, Inc.

Exploration Log

Project Number	Exploration Number	Sheet
KE120241A	EB-2	1 of 1

Project Name: Lake Tye Cable Wakeboard Park
Location: Monroe, WA
Driller/Equipment: Geologic Drill / XL
Hammer Weight/Drop: 140# / 30"

Ground Surface Elevation (ft): ~30
Datum: N/A
Date Start/Finish: 7/2/12,7/2/12
Hole Diameter (in): 6 inches

Depth (ft)	ST	Samples	Graphic Symbol	DESCRIPTION	Well Completion	Water Level	Blows/6"	Blows/Foot (10 20 30 40)	Other Tests
				Topsoil					
				Fill					
				Driller reports gravel at 2.5 feet.					
		S-1		Medium dense, very moist to wet, dark brown, silty fine to medium SAND, few gravel, organics (SM). Recovery 12 inches.			5 15 12	▲27	
5				Very moist to wet, brown, SAND, with gravel grading to little silt. Driller reports smoother drilling at approximately 5 feet.					
		S-2		Loose, very moist to wet, silty fine to medium SAND, few gravel (SM). Recovery 8 inches.		▼	4 5 4	▲9	
				Vashon Recessional Outwash					
10		S-3		Very dense, wet, brown, fine to coarse SAND, with gravel, trace silt; gradational stratification, blowcounts overstated due to gravel (SW). Driller reports less gravel at 12 feet.			50/5"	▲50/5"	
15		S-4		Dense, wet, brown, fine to coarse SAND, few gravel grading to gravelly SAND; gradational stratification, blowcounts overstated due to gravel (SW).			15 16 25	▲44	
				Bottom of exploration boring at 16.5 feet					
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: LDM
Approved by: