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June 30, 2016

Mr. Don Crowe
Arena Sports Mill Creek, LLC
9040 Willows Road
Redmond, WA 98052

Geotechnical Engineering Report
13518 Bothell-Everett Highway
RN File No. 3125-001A

Dear Mr. Crowe:

This letter serves as a transmittal for our report update for the project located at 13518 Bothell-Everett Highway in Mill Creek, Washington. Project plans are to develop a sports arena. The subsurface soils encountered are capable of providing support for the planned structures.

We appreciate the opportunity of working with you on this project. If you have any questions regarding this report, please contact us.

Sincerely,

Rick B. Powell, PE
Principal Engineer

RBP:am

Five Figures
Appendix A

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INTRODUCTION

This report presents the results of our geotechnical engineering investigation for your proposed sports arena project. The site is located at 13518 Bothell-Everett Highway, north of Dumas Road, as shown on the Vicinity Map in Figure 1.

We previously completed a report for the site as Cornerstone Geotechnical, dated December 1, 2004 for a previous owner. You have requested that we complete this report update to re-evaluate and update the geotechnical parameters for the site.

PROJECT DESCRIPTION

We understand you plan to construct a sports arena. We have not been provided project plans at this time but we understand that site grading will include moderate cuts and fills.

SCOPE

The scope of services to be provided by Robinson Noble, Inc., as outlined in our proposal dated June 3, 2016, is for geotechnical evaluation services, including the following:

- Review the existing geotechnical report and plans.
- The Robinson Noble staff will confirm that site conditions and grades have not changed since our exploration program.
- Update the geotechnical parameters based on the planned conditions.
- Update the seismic portions of the geotechnical report.
- Prepare the updated geotechnical report referencing the current IBC.

SITE CONDITIONS

Surface Conditions

The approximate seven-acre site has maximum dimensions of 500 feet parallel to Bothell-Everett Highway in the northeast-southwest direction and 650 feet in the perpendicular direction. The site is bordered by Bothell-Everett Highway to the southeast and existing residential property on the remaining three sides. A layout of the site is shown on the Site Plan in Figure 2.

The site is flat-lying with a single-family residence and detached garage located on the western corner. The site consists of grassy pasture and a wooded area with small- to- medium sized trees, with brush and evergreen trees up to 2 feet in diameter located at the eastern end of the site.

Site Reconnaissance

We walked the site on June 14, 2016 and observed surficial conditions. We did not observe any indications of unstable slopes within the site or within the surrounding area at the time of our visit. We also did not observe any new development within the area of the site or any grading that may change our design parameters.

Geology

Most of the Puget Sound Region was affected by past intrusion of continental glaciation. The last period of glaciation, the Vashon Stade of the Fraser Glaciation, ended approximately 14,000 years ago. Many of the geomorphic features seen today are a result of scouring and overriding

by glacial ice. During the Vashon Stade, areas of the Puget Sound region were overridden by over 3,000 feet of ice. Soil layers overridden by the ice sheet were compacted to a much greater extent than those that were not. Part of a typical glacial sequence within the area of the site includes the following soil deposits from newest to oldest:

Artificial Fill (af) – Fill material is often locally placed by human activities, consistency will depend on the source of the fill. The thickness and expanse of this material will be dependent on the extent of fill required to grade land to the desired elevations. Density of the fill will depend on earthwork activities and compaction efforts made during the placement of the material.

Recessional Outwash (Qvr) – These deposits were derived from the stagnating and receding Vashon glacier and consist mostly of stratified sand and gravel, but include unstratified ablation and melt-out deposits. Recessional deposits were not compacted by the glacier and are typically not as dense as those that were.

Vashon Till (Qvt) – The till is a non-sorted mixture of clay, sand, pebbles, cobbles and boulders, all in variable amounts. The till was deposited directly by the ice as it advanced over and eroded irregular surfaces of previously deposited formations and sediments. The till was well compacted by the advancing glacier and exhibits high strength and stability. Drainage is considered very poor in the till.

Advance Outwash (Qva) – The advance outwash typically is a thick section of mostly clean, pebbly sand with increasing amounts of gravel higher in the section. The advance outwash was placed by the advancing glaciers and was overridden and well compacted by the glacier.

The geologic units for this area are mapped on the Geologic Map of the Everett 7.5 - Minute Quadrangle, Snohomish County, Washington, by James P. Minard (U.S. Geological Survey, 1985). The site is mapped as being underlain by a deposit of glacial till. Our site explorations encountered glacial till and glacial drift. Glacial drift is similar to glacial till, but may exhibit more sorting of various soil grain sizes.

Explorations

Subsurface conditions were explored at the site on November 17, 2004, by excavating a total of six test pits. The test pits were excavated to depths of 7.0 to 11.5 feet below the ground surface. The explorations were located in the field by a representative from this firm who also examined the soils and geologic conditions encountered, and maintained logs of the test pits. The approximate locations of the test pits are shown on the Site Plan in Figure 2. The soils were visually classified in general accordance with the Unified Soil Classification System, a copy of which is presented as Figure 3. The logs of the test pits are presented in Figures 4 and 5.

Subsurface Conditions

A brief description of the conditions encountered in our explorations is included below. For a more detailed description of the soils encountered, review the Test Pit Logs in Figures 4 and 5.

Most of our explorations encountered a surficial layer of topsoil that averaged less than 1 foot in thickness. The topsoil consisted of loose, dark brown to black silty sand with gravel. In Test Pits 1, 4, and 6, approximately 2.0 to 3.0 feet of fill was encountered at the ground surface overlying this topsoil. This fill consisted of loose, moist, gray-brown silty sand with gravel. Underlying the topsoil, a weathered soil horizon, approximately 1 to 2 feet in thickness, was encountered in the explorations. The weathered horizon consisted of loose to medium dense, gray to red-brown, silty sand and gravel. We interpret this soil to be weathered till. In Test Pits 2 and 5, weathered till was present at the surface without the presence of topsoil or fill. Below the weathered horizon, we encountered gray silty fine to medium sand with gravel and cobbles. The soil color was predominantly gray with rust mottling. Most of the test pits were completed in dense to very dense, moist to wet, gray silty sand with gravel. This soil is interpreted as glacial till. Test Pit 6 was completed in very dense, gray fine to medium sand with silt, gravel and cobbles. This soil is interpreted as drift.

Hydrologic Conditions

Shallow groundwater seepage was encountered between 3.0 and 5.0 feet in Test Pit 3. We consider this water to be perched. We also encountered rust staining in our explorations. This mottled zone is also a sign of perched water during the wet season. The thickness of the mottled layer does not indicate full saturation of the unit. The dense to very dense till or drift below this mottled zone is considered poorly draining. During the wetter times of the year, we expect perched water conditions will occur as pockets of water on top of the till layer. Perched water does not represent a regional ground water "table" within the upper soil horizons. Volumes of perched groundwater vary depending upon the time of year and the upslope recharge conditions.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion that the site is compatible with the planned development. The underlying medium dense to very dense glacial deposits are capable of supporting the planned structures and pavements. We recommend that the foundations for the structures extend through any topsoil, fill, loose, or disturbed soils, and bear on the underlying medium dense to very dense, native glacial soils, or on structural fill extending to these soils. Based on our site explorations, we anticipate these soils will generally be encountered at typical footing depths.

The soils likely to be exposed during construction are highly moisture sensitive and will disturb easily when wet or during wet conditions. We recommend that construction take place during the drier summer months, if possible. If construction takes place during the wet season, additional expenses and delays should be expected due to the wet conditions. Additional expenses could include additional depth of site stripping, export of on-site soil, the import of

clean granular soil for fill, and the need to place a blanket of rock spalls in the access roads and paved areas prior to placing structural fill.

Geologic Hazards

Landslide Hazards: The subject site is underlain by dense to very dense Vashon Till or drift at shallow depths. These soils typically exhibit very high shear strength and have high resistance against slope failure. The terrain within the site is flat lying to gently sloping. Therefore, landslides are not a concern on the site.

Erosion Hazard: The erosion hazard criteria used for determination of affected areas includes soil type, slope gradient, vegetation cover, and ground water conditions. The erosion sensitivity is related to vegetative cover and the specific surface soil types (group classification), which are related to the underlying geologic soil units. The Soil Survey of Snohomish County Area Washington by the Soil Conservation Service (SCS) was reviewed to determine the erosion hazard of the on-site soils. The site surface soils were classified using the SCS classification system as Unit 1 (Alderwood gravelly sandy loam). The corresponding geologic unit for these soils is till, which is in agreement with the soils encountered in our site explorations. The erosion hazard for the soil is listed as being slight for the gently sloping conditions at the site.

Seismic Hazard: It is our opinion based on our subsurface explorations that the Soil Profile in accordance with the 2012 International Building Code (IBC) is Site Class C with Seismic Design Category D. We used the US Geological Survey program "U.S. Seismic Design Maps Web Application." The design maps summary report for the 2012 IBC is included in this report as Appendix A.

Additional seismic considerations include liquefaction potential and amplification of ground motions by soft soil deposits. The liquefaction potential is highest for loose sand with a high groundwater table. The underlying dense till and drift soils are considered to have a very low potential for liquefaction and amplification of ground motion.

Site Preparation and Grading

The first step of site preparation should be to strip the vegetation, topsoil, fill or loose soils to expose medium dense to very dense native soils in pavement and building areas. This material should be removed from the site, or stockpiled for later use as landscaping fill. The resulting subgrade should be compacted to a firm, non-yielding condition. Areas observed to pump or weave should be repaired prior to placing hard surfaces.

The on-site glacial till likely to be exposed during construction is considered highly moisture sensitive, and the surface will disturb easily when wet. We expect these soils would be difficult, if not impossible, to compact to structural fill specifications in wet weather. We recommend that earthwork be conducted during the drier months. Additional expenses of wet weather or winter construction would include extra excavation and use of imported fill or rock spalls. During wet weather, alternative site preparation methods may be necessary. These methods may include utilizing a smooth-bucket trackhoe to complete site stripping and

diverting construction traffic around prepared subgrades. Disturbance to the prepared subgrade may be minimized by placing a blanket of rock spalls or imported sand and gravel in traffic and roadway areas. Cutoff drains or ditches can also be helpful in reducing grading costs during the wet season. These methods can be evaluated at the time of construction.

Structural Fill

General: All fill placed beneath buildings, pavements or other settlement sensitive features should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards, and is observed by an experienced geotechnical professional or soils technician. Field observation procedures would include the performance of a representative number of in-place density tests to document the attainment of the desired degree of relative compaction.

Materials: Imported structural fill should consist of a good quality, free-draining granular soil, free of organics and other deleterious material, and be well graded to a maximum size of about 3 inches. Imported, all-weather structural fill should contain no more than 5 percent fines (soil finer than a Standard U.S. No. 200 sieve), based on that fraction passing the U.S. 3/4-inch sieve. The use of on-site soil as structural fill will be dependent on moisture content control. Some drying of the native soils may be necessary in order to achieve compaction. During warm, sunny days this could be accomplished by spreading the material in thin lifts and compacting. Some aeration and/or addition of moisture may also be necessary. We expect that compaction of the native soils to structural fill specifications would be difficult, if not impossible, during wet weather.

Fill Placement: Following subgrade preparation, placement of the structural fill may proceed. Fill should be placed in 8- to 10-inch-thick uniform lifts, and each lift should be spread evenly and be thoroughly compacted prior to placement of subsequent lifts. All structural fill underlying building areas, and within a depth of 2 feet below pavement and sidewalk subgrade, should be compacted to at least 95 percent of its maximum dry density. Maximum dry density, in this report, refers to that density as determined by the ASTM D1557 compaction test procedure. Fill more than 2 feet beneath sidewalks and pavement subgrades should be compacted to at least 90 percent of the maximum dry density. The moisture content of the soil to be compacted should be within about 2 percent of optimum so that a readily compactable condition exists. It may be necessary to overexcavate and remove wet surficial soils in cases where drying to a compactable condition is not feasible. All compaction should be accomplished by equipment of a type and size sufficient to attain the desired degree of compaction.

Temporary and Permanent Slopes

Temporary cut slope stability is a function of many factors, such as the type and consistency of soils, depth of the cut, surcharge loads adjacent to the excavation, length of time a cut remains open, and the presence of surface or ground water. It is exceedingly difficult under these variable conditions to estimate a stable temporary cut slope geometry. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations, since the contractor is

continuously at the job site, able to observe the nature and condition of the cut slopes, and able to monitor the subsurface materials and groundwater conditions encountered.

We anticipate temporary cuts for installation of utilities. For planning purposes, we recommend that temporary cuts in the near-surface weathered soils be no greater than 1 Horizontal to 1 Vertical (1H:1V). Cuts in the dense to very dense till may stand at a 0.75H:1V inclination or possibly steeper. If groundwater seepage is encountered, we expect that flatter inclinations would be necessary.

We recommend that cut slopes be protected from erosion. Measures taken may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes. We do not recommend vertical slopes for cuts deeper than 4 feet, if worker access is necessary. We recommend that cut slope heights and inclinations conform to local and WISHA/OSHA standards.

Final slope inclinations for structural fill and the cuts in the native soils should be no steeper than 2H:1V. Lightly compacted fills or common fills should be no steeper than 3H:1V. Common fills are defined as fill material with some organics that are "trackrolled" into place. They would not meet the compaction specification of structural fill. Final slopes should be vegetated and covered with straw or jute netting. The vegetation should be maintained until it is established.

Foundations

Conventional shallow spread foundations should be founded on undisturbed, medium dense to very dense glacial soils, or be supported on structural fill extending to those soils. If the soil at the planned bottom of footing elevation is not medium dense to very dense, it should be overexcavated to expose suitable bearing soil, and the excavation should be filled with structural fill, or the footing may be overpoured with extra concrete.

Footings should extend at least 18 inches below the lowest adjacent finished ground surface for frost protection and bearing capacity considerations. Minimum foundation widths of 12 and 18 inches should be used for continuous and isolated spread footings, respectively. Standing water should not be allowed to accumulate in footing trenches. All loose or disturbed soil should be removed from the foundation excavation prior to placing concrete.

For foundations constructed as outlined above, we recommend an allowable design bearing pressure of 2,500 pounds per square foot (psf) be used for the footing design. International Building Code (IBC) guidelines should be followed when considering short-term transitory wind or seismic loads. Potential foundation settlement using the recommended allowable bearing pressure is estimated to be less than 1-inch total and ½-inch differential between footings or across a distance of about 30 feet. Higher soil bearing values may be appropriate for footings founded on the unweathered till, and with wider footings. These higher values can be determined after a review of a specific design.

Lateral Loads

The lateral earth pressure acting on retaining walls is dependent on the nature and density of the soil behind the wall, the amount of lateral wall movement, which can occur as backfill is placed, and the inclination of the backfill. Walls that are free to yield at least one-thousandth of the height of the wall are in an "active" condition. Walls restrained from movement by stiffness or bracing are in an "at-rest" condition. Active earth pressure and at-rest earth pressure can be calculated based on equivalent fluid density. Equivalent fluid densities for active and at-rest earth pressure of 35 pounds per cubic foot (pcf) and 55 pcf, respectively, may be used for design for a level backslope. These values assume that the on-site soils or imported granular fill are used for backfill, and that the wall backfill is drained. The preceding values do not include the effects of surcharges, such as due to foundation loads or other surface loads. Surcharge effects should be considered where appropriate.

Seismic lateral loads are a function of the site location, soil strength parameters and the peak horizontal ground acceleration (PGA) for a given return period. We used the US Geological Survey program "2009 PSH Deaggregation on NEHRP" to compute the PGA for the site. The 3-D histogram is included in Appendix A. The above drained active and at-rest values should be increased by a uniform pressure of 7.6H and 10.4 H psf, respectively, when considering seismic conditions. H represents the wall height.

The above lateral pressures may be resisted by friction at the base of the wall and passive resistance against the foundation. A coefficient of friction of 0.5 may be used to determine the base friction in the native glacial soils. An equivalent fluid density of 250 pcf may be used for passive resistance design. To achieve this value of passive pressure, the foundations should be poured "neat" against the native dense soils, or compacted fill should be used as backfill against the front of the footing, and the soil in front of the wall should extend a horizontal distance at least equal to three times the foundation depth. A resistance factor of 0.67 has been applied to the passive pressure to account for required movements to generate these pressures. The friction coefficient does not include a factor of safety.

All wall backfill should be well compacted. Care should be taken to prevent the buildup of excess lateral soil pressures due to overcompaction of the wall backfill.

Slabs-On-Grade

Slab-on-grade areas should be prepared as recommended in the **Site Preparation and Grading** subsection. Slabs should be supported on medium dense or firmer native soils, or on structural fill extending to these soils. Where moisture control is a concern, we recommend that slabs be underlain by 6 inches of pea gravel for use as a capillary break. A suitable vapor barrier, such as heavy plastic sheeting, should be placed over the capillary break. An additional 2-inch-thick damp sand blanket can be used to cover the vapor barrier to protect the membrane and to aid in curing the concrete. This will also help prevent cement paste bleeding down into the capillary break through joints or tears in the vapor barrier. The capillary break material should be connected to the footing drains to provide positive drainage.

Drainage

We recommend that runoff from impervious surfaces, such as roofs, driveway and access roadways, be collected and routed to an appropriate stormwater discharge system. Final site grades should allow for drainage away from any buildings. We suggest that the finished ground surface be sloped at a gradient of 3 percent minimum for a distance of at least 10 feet away from the buildings. Surface water should be collected by permanent catch basins and drain lines, and be discharged into a storm drain system.

We recommend that footing drains be used around all of the structures where moisture control is important. The underlying till/drift will pond water that accumulates in the crawl space. It is good practice to use footing drains installed at least 1 foot below the planned finished floor slab or crawl space elevation to provide drainage for the crawl space. At a minimum, the crawl space should be sloped to drain to an outlet tied to the drainage system. If drains are omitted around slab-on-grade floors where moisture control is important, the slab should be a minimum of 1 foot above surrounding grades.

Where used, footing drains should consist of 4-inch-diameter, perforated PVC pipe that is surrounded by free-draining material, such as pea gravel. Footing drains should discharge into tightlines leading to an appropriate collection and discharge point. Crawl spaces should be sloped to drain, and a positive connection should be made into the foundation drainage system. For slabs-on-grade, a drainage path should be provided from the capillary break material to the footing drain system. Roof drains should not be connected to wall or footing drains.

Detention Pond

Although vaults are being considered, we provide, herein, recommendations for ponds in the event that option is considered. The ponds should be excavated into the underlying native soils. We recommend that any fill berms be constructed of the low permeability soils. The on-site till encountered in our test pit explorations meets this criterion. Any material proposed for the berm should be evaluated for its suitability. We recommend that the berm material have a maximum permeability of 1×10^{-6} centimeters per second (cm/sec).

The cut slopes of the pond in the dense native soils should be no steeper than 2H:1V on the inside of the detention pond and no steeper than 2H:1V above the water table or on the outside portions of the pond berms. Fill slopes on the interior of the pond should be no steeper than 3.0H:1V.

Where any berms for the pond are to be constructed, the topsoil and loose soils should be removed down to the medium dense to very dense till. Areas to receive new fill should be stripped of unsuitable surface soils and compacted to a firm, non-yielding state prior to placement of the new fill. The excavation should be kept dry to allow the proper placement of structural fill. Structural fill should be placed and compacted as discussed in the **Structural Fill** subsection of this report. We recommend that the fill in any pond berms be compacted to a minimum of 92 percent of its maximum dry density as determined by the ASTM D 1557 compaction test procedure. After each lift of the fill in a berm is compacted to specification, the

surface should be scarified to a depth of 2 inches prior to placement of the next lift. The purpose of the scarification is to reduce the risk of creating preferential seepage paths through the pond or berms.

It will be important to compact the face of any pond fill embankments. This should be made explicit to the contractor performing the on-site work. Uncompacted soils on a berm face will be more susceptible to erosion and sloughing. If groundwater seepage is encountered within a cut slope face, a layer of rock spalls may be necessary to minimize erosion of the slope face. The spall layer can be placed at the time of construction, or in the future if sloughing of the slope is observed.

Detention Vault

If a concrete detention vault is to be constructed, the concrete walls of the vault may be supported on foundations bearing on the underlying gray, dense to very dense, glacial till. The allowable soil bearing pressure should not exceed 4,000 pounds per square foot (psf) for the design of the wall footings poured on undisturbed glacial till.

We recommend that footing drains be installed on the outside of perimeter footings. The footing drains should be at least 4 inches in diameter and should consist of perforated or slotted, rigid, smooth-walled PVC pipe, laid at the bottom of the footings. The drain line should be surrounded with free-draining pea gravel or coarse sand and wrapped with a layer of non-woven filter fabric. A vertical drainage blanket at least 12 inches thick, consisting of compacted pea gravel or other free-draining granular soils, should be placed against the walls. A vertical drain mat, such as Miradrain 6000 by Mirafi Inc., may be placed against the walls in lieu of the vertical drainage blanket. Structural fill is then placed behind the vertical drainage blanket or drain mat to backfill the walls. The vertical drainage blanket or drain mat should be hydraulically connected to the drain line at the base of the walls. Sufficient number of cleanouts at strategic locations should be installed for periodic cleaning of the wall drain line to prevent clogging. If sufficient fall does not exist to allow the footing drains placed at foundation elevation, the elevation of the drain may be raised. This will require buoyancy and hydrostatic forces be considered below the drain.

The perimeter walls of the concrete vault with a lid would be restrained at their top from horizontal movement and should be designed for at-rest lateral soil pressure, while the perimeter walls of a vault without a lid would be unrestrained at the top and may be designed for active lateral soil pressure. Active earth pressure and at-rest earth pressure can be calculated based on equivalent fluid density. Equivalent fluid densities for active and at rest earth pressure of 35 pcf and 55 pcf, respectively, may be used for design for level backslope. These values assume that the on-site soils are used for backfill, and that the wall backfill is drained. The preceding values do not include the effects of surcharges due to foundation loads, traffic or other surface loads. Surcharge effects should be considered where appropriate. For <u>undrained</u> soil conditions, the active and at-rest pressures should be increased to 80 pcf and 90 pcf, respectively. Undrained conditions may occur in the lower portion of the vault if there is not suitable fall to place a wall drain at the footing elevation.

All wall backfill should be well compacted. Care should be taken to prevent the buildup of excess lateral soil pressures due to overcompaction of the wall backfill. This can be accomplished by placing wall backfill in 8-inch loose lifts and compacting with small, hand-operated compactors.

We recommend that an equivalent fluid density of 250 pcf be used to calculate the allowable lateral passive resistance for the case of a level ground surface adjacent to the footing. An allowable coefficient of friction between footings and soil of 0.5 may be used, and should be applied to the vertical dead load only. A factor of safety of 2.0 has been applied to the passive pressure to account for required movements to generate these pressures. The friction coefficient does not include a factor of safety.

Utilities

Our explorations indicate that deep dewatering will not be needed to install standard depth utilities. Anticipated groundwater is expected to be handled with pumps in the trenches. We also expect that some groundwater seepage may develop during and following the wetter times of the year. We expect this seepage to mostly occur in pockets. We do not expect significant volumes of water in these excavations.

The soils likely to be exposed in utility trenches after site stripping are considered highly moisture sensitive. We recommend that they be considered for trench backfill during the drier portions of the year. Provided these soils are within 2 percent of their optimum moisture content, they should be suitable to meet compaction specifications. During the wet season, it may be difficult to achieve compaction specifications; therefore, soil amendment with kiln dust or cement may be needed to achieve proper compaction with the on-site materials.

Pavement Subgrade

The performance of roadway pavement is critically related to the conditions of the underlying subgrade. We recommend that the subgrade soils within the roadways be prepared as described in the **Site Preparation and Grading** subsection of this report. Prior to placing base material, the subgrade soils should be compacted to a non-yielding state with a vibratory roller compactor and then proof-rolled with a piece of heavy construction equipment, such as a fully-loaded dump truck. Any areas with excessive weaving or flexing should be overexcavated and recompacted or replaced with a structural fill or crushed rock placed and compacted in accordance with recommendations provided in the **Structural Fill** subsection of this report.

CONSTRUCTION OBSERVATION

We should be retained to provide observation and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, and to provide recommendations for design changes, should the conditions revealed during the work differ from those anticipated. As part of our services, we would also evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications.

USE OF THIS REPORT

We have prepared this report for Arena Sports Mill Creek, LLC and its agents, for use in planning and design of this project. The data and report should be provided to prospective contractors for their bidding and estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of subsurface conditions.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report, for consideration in design. There are possible variations in subsurface conditions. We recommend that project planning include contingencies in budget and schedule, should areas be found with conditions that vary from those described in this report.

Within the limitations of scope, schedule and budget for our services, we have strived to take care that our services have been completed in accordance with generally accepted practices followed in this area at the time this report was prepared. No other conditions, expressed or implied, should be understood.

We appreciate the opportunity to be of service to you. If there are any questions concerning this report or if we can provide additional services, please call.

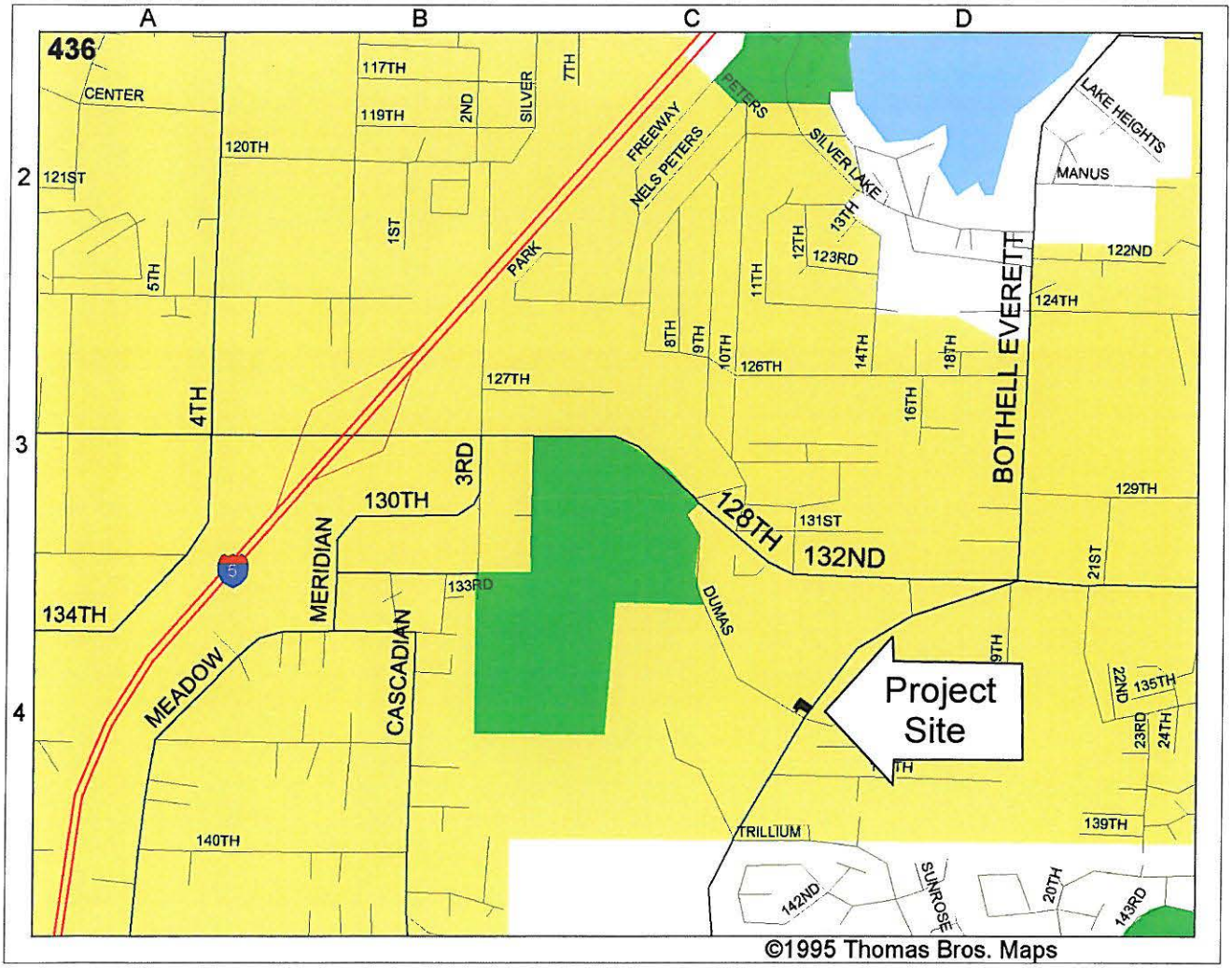
Sincerely,
Robinson Noble, Inc.

Rick B. Powell, PE
Principal Engineer

JHA:RBP:am

Five Figures
Appendix A

Vicinity Map



Cornerstone Geotechnical, Inc.

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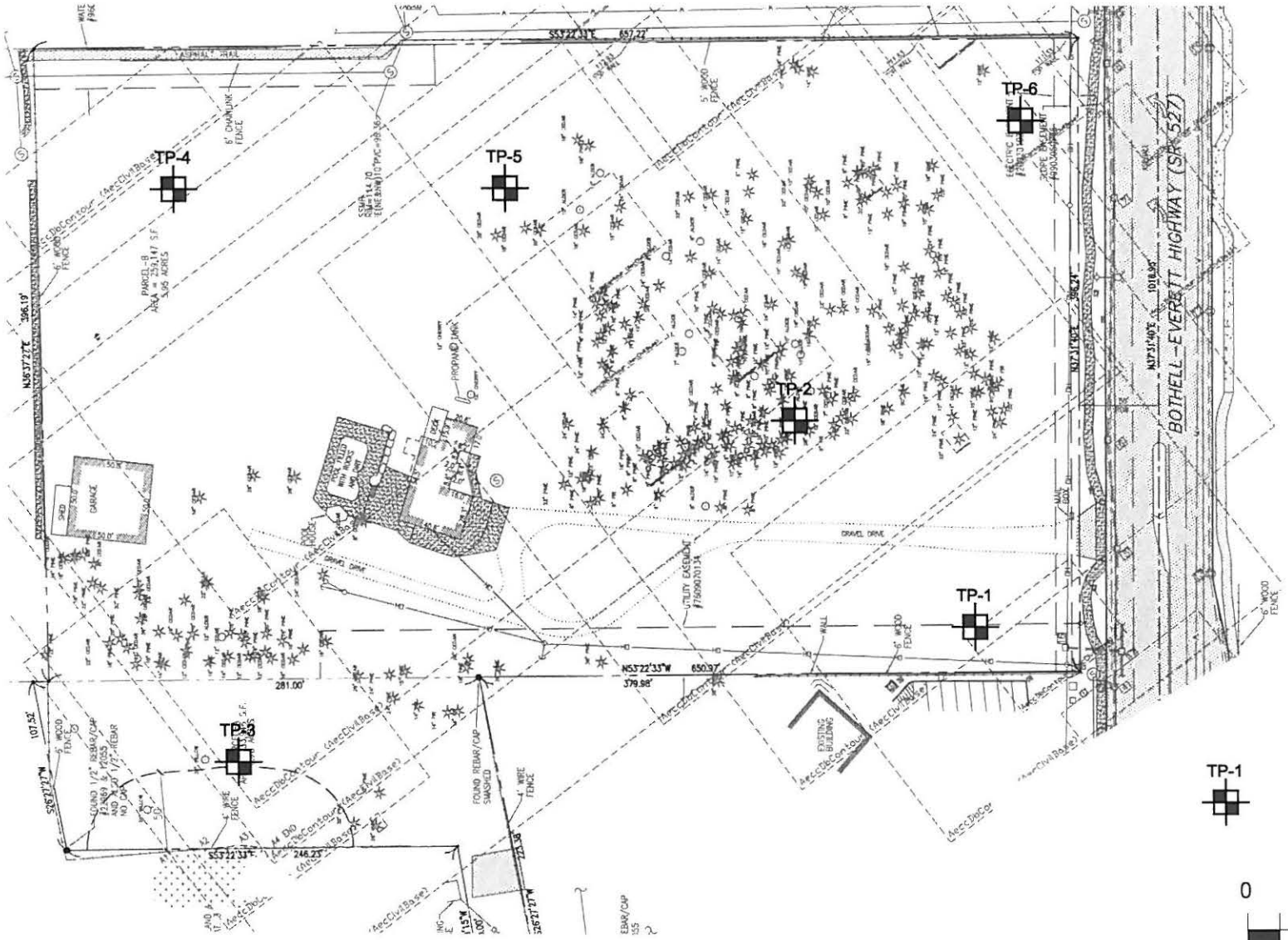
Mill Creek Church

File Number	Figure
1769	1

Site Plan



LEGEND

TP-1 Number and Approximate Location of Test Pit

0 100 200

Scale 1" = 100'

Site Plan base on an electronic version of a site plan by Allied Land Surveying dated 10/21/2004

Cornerstone Geotechnical, Inc.

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Mill Creek Church

File Number	Figure
1769	2

Unified Soil Classification System

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50% RETAINED ON number 200 SIEVE	GRAVEL MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50% OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE - GRAINED SOILS MORE THAN 50% PASSES NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50%	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50% OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1) Field classification is based on visual examination of soil in general accordance with ASTM D 2488-83.
2) Soil classification using laboratory tests is based on ASTM D 2487-83.
3) Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS

Dry- Absence of moisture, dusty, dry to the touch

Moist- Damp, but no visible water

Wet- Visible free water or saturated, usually soil is obtained from below water table

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Mill Creek Church

Figure 3

LOG OF EXPLORATION

DEPTH	USC	SOIL DESCRIPTION
TEST PIT ONE		
0.0 – 3.0	SM	GRAY-BROWN SILTY SAND WITH GRAVEL (LOOSE, MOIST) **(FILL)**
3.0 – 3.5	SM	DARK BROWN TO BLACK SILTY SAND WITH GRAVEL (LOOSE, MOIST) **(TOPSOIL)**
3.5 – 4.5	SM	BROWN SILTY FINE TO COARSE SAND WITH GRAVEL AND COBBLES (LOOSE TO MEDIUM DENSE, MOIST) **(WEATHERED TILL)**
4.5 – 11.5	SM	OCCASIONALY RUST-STAINED GRAY SILTY FINE TO COARSE SAND WITH GRAVEL AND COBBLES (DENSE TO VERY DENSE, MOIST) **(TILL)**
		SAMPLES WERE COLLECTED AT 8.0 AND 11.5 FEET GROUND WATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS ENCOUNTERED BETWEEN 0.0 AND 3.0 FEET TEST PIT WAS COMPLETED AT 11.5 FEET ON 11/17/04
TEST PIT TWO		
0.0 – 2.5	SM	BROWN SILTY FINE TO MEDIUM SAND WITH GRAVEL (LOOSE TO MEDIUM DENSE, MOIST) **(WEATHERED TILL)**
2.5 – 7.0	SM	GRAY SILTY FINE TO MEDIUM SAND WITH GRAVEL AND OCCASIONAL COBBLES (DENSE, MOIST) **(TILL)**
		SAMPLES WERE COLLECTED AT 3.0 AND 7.0 FEET GROUND WATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 7.0 FEET ON 11/17/04
TEST PIT THREE		
0.0 – 3.0	SM	DARK BROWN TO BLACK ORGANIC-LADEN SILTY SAND WITH GRAVEL (LOOSE, MOIST) **(TOPSOIL)**
3.0 – 5.0	SM	RUST-STAINED GRAY SILTY FINE TO MEDIUM SAND WITH GRAVEL AND COBBLES (LOOSE TO MEDIUM DENSE, WET) **(WEATHERED TILL)**
5.0 – 11.0	SM	GRAY SILTY FINE TO MEDIUM SAND WITH GRAVEL AND COBBLES (MEDIUM DENSE, WET) **(TILL)**
		SAMPLES WERE COLLECTED AT 7.0 AND 11.0 FEET GROUND WATER SEEPAGE WAS ENCOUNTERED BETWEEN 3.0 AND 5.0 FEET TEST PIT CAVING WAS ENCOUNTERED BETWEEN 3.0 AND 5.0 FEET TEST PIT WAS COMPLETED AT 11.0 FEET ON 11/17/04
TEST PIT FOUR		
0.0 – 2.0	SM	GRAY-BROWN SILTY SAND WITH GRAVEL (LOOSE, MOIST) **(FILL)**
2.0 – 2.5	SM	DARK BROWN TO BLACK SILTY SAND WITH GRAVEL (LOOSE, MOIST) **(TOPSOIL)**
2.5 – 4.0	SM	RED-BROWN SILTY FINE TO COARSE SAND WITH GRAVEL (LOOSE TO MEDIUM DENSE, MOIST) **(WEATHERED TILL)**
4.0 – 7.0	SM	GRAY SILTY FINE SAND WITH GRAVEL (DENSE, MOIST) **(TILL)**
		SAMPLE WAS COLLECTED AT 7.0 FEET GROUND WATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 7.0 FEET ON 11/17/04

LOG OF EXPLORATION

DEPTH	USC	SOIL DESCRIPTION
TEST PIT FIVE		
0.0 – 1.0	SM	DARK BROWN TO BLACK ORGANIC-LADEN SILTY SAND WITH GRAVEL (LOOSE, MOIST) **(TOPSOIL)**
1.0 – 1.5	SM	RED-BROWN SILTY FINE TO MEDIUM SAND WITH GRAVEL (LOOSE TO MEDIUM DENSE, MOIST) **(WEATHERED TILL)**
1.5 – 4.0	SM	RUST-STAINED GRAY SILTY FINE TO MEDIUM SAND WITH GRAVEL AND COBBLES (MEDIUM DENSE TO DENSE, MOIST) **(TILL)**
4.0 – 7.0	SM	GRAY SILTY FINE TO MEDIUM SAND WITH GRAVEL AND COBBLES (VERY DENSE, MOIST) **(TILL)**
		SAMPLE WAS COLLECTED AT 7.0 FEET GROUND WATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 7.0 FEET ON 11/17/04
TEST PIT SIX		
0.0 – 2.0	SM	GRAY-BROWN SILTY SAND WITH GRAVEL (LOOSE, MOIST) **(FILL)**
2.0 – 4.0	SM	DARK BROWN TO BLACK ORGANIC SILTY SAND WITH GRAVEL (LOOSE, MOIST) **(TOPSOIL)**
4.0 – 6.0	SP-SM	GRAY-BROWN FINE TO COARSE SAND WITH SILT AND GRAVEL (MEDIUM DENSE, MOIST) **(WEATHERED DRIFT)**
6.0 – 7.0	SP-SM	GRAY FINE TO MEDIUM SAND WITH SILT, GRAVEL, AND COBBLES (VERY DENSE, MOIST) **(DRIFT)**
		SAMPLES WERE COLLECTED AT 2.0, 4.5 AND 7.0 FEET GROUND WATER SEEPAGE WAS NOT ENCOUNTERED TEST PIT CAVING WAS NOT ENCOUNTERED TEST PIT WAS COMPLETED AT 7.0 FEET ON 11/17/04

Appendix A

USGS Design Maps Summary Report

User-Specified Input

Building Code Reference Document 2012 International Building Code
(which utilizes USGS hazard data available in 2008)

Site Coordinates 47.87255°N, 122.21651°W

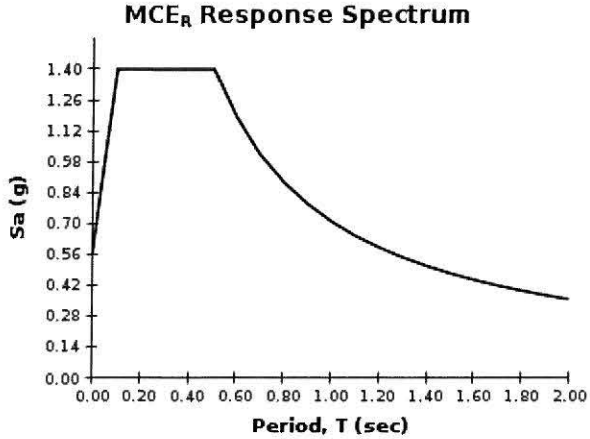
Site Soil Classification Site Class C – "Very Dense Soil and Soft Rock"

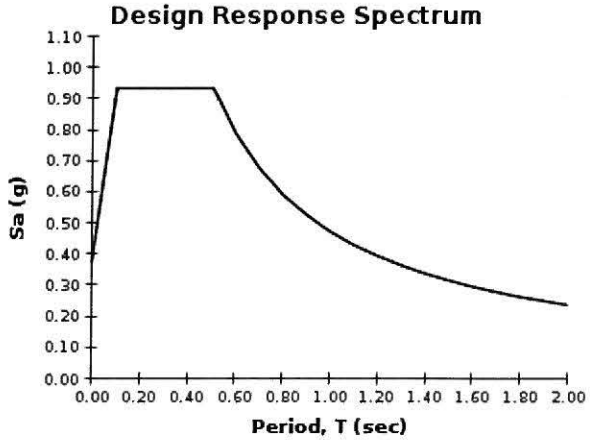
Risk Category I/II/III

USGS-Provided Output

S_S =	1.399 g	S_{MS} =	1.399 g	S_{DS} =	0.933 g
S_1 =	0.545 g	S_{M1} =	0.709 g	S_{D1} =	0.472 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

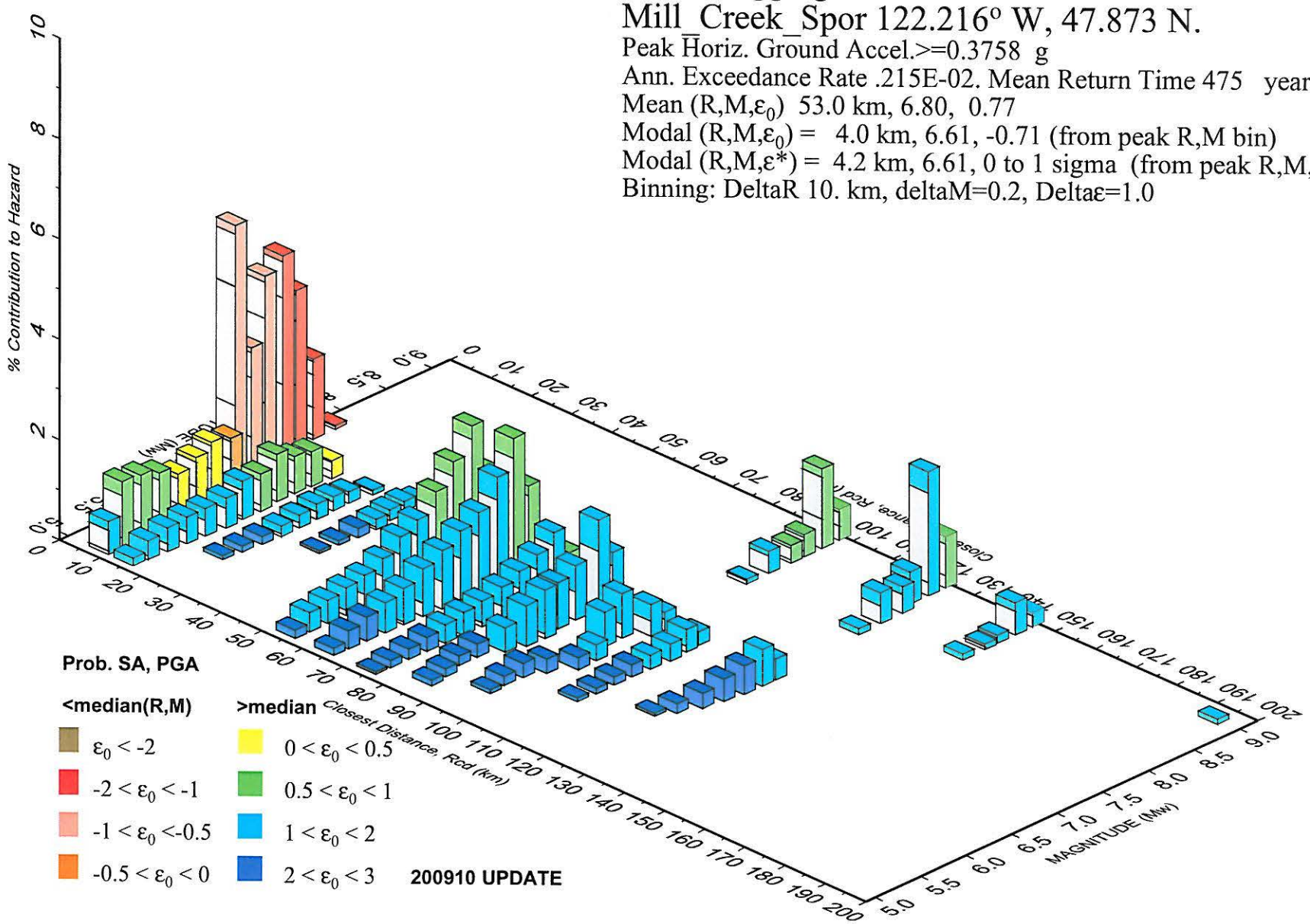




Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

PSH Deaggregation on NEHRP C soil
Mill_Creek_Spor 122.216° W, 47.873 N.

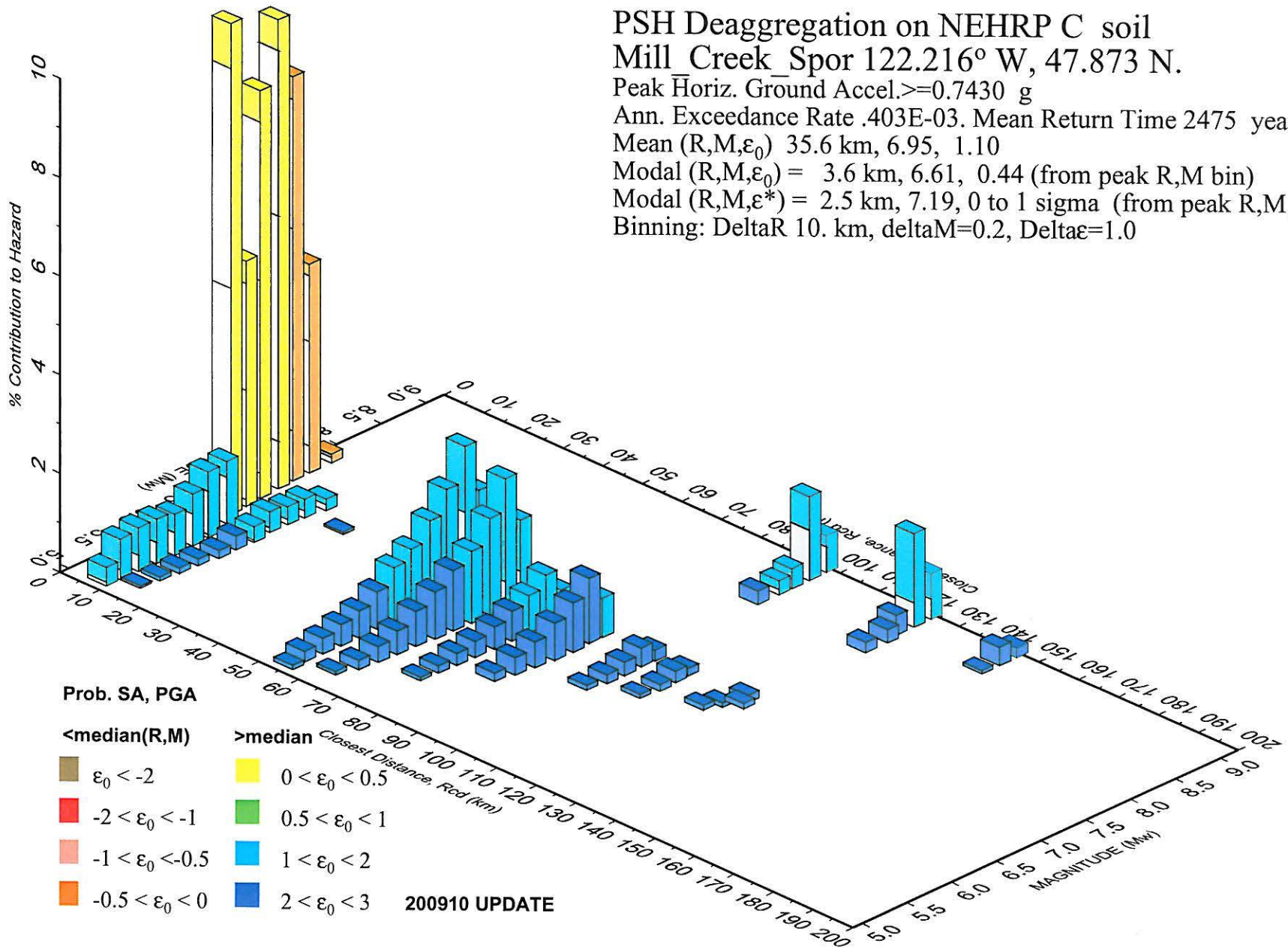
Peak Horiz. Ground Accel.>=0.3758 g
Ann. Exceedance Rate .215E-02. Mean Return Time 475 years
Mean (R,M,ε_0) 53.0 km, 6.80, 0.77
Modal (R,M,ε_0) = 4.0 km, 6.61, -0.71 (from peak R,M bin)
Modal (R,M,ε*) = 4.2 km, 6.61, 0 to 1 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0



GMT 2016 Jun 22 21:06:04 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 400. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

PSH Deaggregation on NEHRP C soil
Mill_Creek_Spor 122.216° W, 47.873 N.

Peak Horiz. Ground Accel.>=0.7430 g
Ann. Exceedance Rate .403E-03. Mean Return Time 2475 years
Mean (R,M,ε_0) 35.6 km, 6.95, 1.10
Modal (R,M,ε_0) = 3.6 km, 6.61, 0.44 (from peak R,M bin)
Modal (R,M,ε*) = 2.5 km, 7.19, 0 to 1 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0



GMT 2016 Jun 22 21:05:05 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 400. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted