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March 10, 2016

Mr. Steve Willie
Jerome W. Morrissette & Associates Inc., P.S.
1700 Cooper Point Road SW, Suite B-2
Olympia, Washington 98502-1110

Geotechnical Engineering Report
Main Street Roadway
Bucoda, Washington
RN File No. 3101-001A

Dear Mr. Willie:

This letter serves as a transmittal for our report for the Main Street Roadway project in Bucoda, Washington. Currently during overbank flooding on the Skookumchuck River portions of South Main Street and East 11th Street become flooded resulting in approximately 50 residences being isolated from emergency services. We understand that the plan is to provide access to the residences by raising the elevation above the flood stage and installing box culverts. The road elevation will be a minimum of 246 feet for approximately 1,200 feet of South Main Street and 300 feet of East 11th Street.

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

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INTRODUCTION

This report presents the results of our geotechnical engineering investigation for the Town of Bucoda's Main Street Roadway project. You have requested that we complete this report to evaluate subsurface conditions and provide recommendations for site development.

PROJECT DESCRIPTION

Currently during overbank flooding on the Skookumchuck River portions of South Main Street and East 11th Street become flooded resulting in approximately 50 residences being isolated from emergency services. We understand that the plan is to provide access to the residences by raising the elevation above the flood stage and installing box culverts or bridge structure. The road elevation will be a minimum of 246 feet for approximately 1,200 feet of South Main Street and 300 feet of East 11th Street.

SCOPE

The purpose of this study is to explore and characterize the subsurface conditions and present recommendations for site development through the design phase of the project. Specifically, our scope of services as outlined in Phase I of our Services Agreement, dated January 25, 2016, includes the following:

- Review available geologic maps for the site.
- Explore the subsurface soil and groundwater conditions in the area of the planned construction with two borings.
- Evaluate pertinent physical and engineering characteristics of the soils encountered in the borings.
- Prepare a geotechnical report containing the results of our subsurface explorations, and our conclusions and recommendations for geotechnical design elements of the project.

SITE CONDITIONS

Surface Conditions

The project includes approximately 1,200 feet of South Main Street and 300 feet of East 11th Street. The ground surface within the project area is relatively flat. Single family residences and grassy pasture border both roadway alignments. A layout of the site is shown on the Site Plan in Figure 2.

The existing roadway for South Main Street is concrete from East 11th Street extending northeast. The concrete has some shrinkage cracks but appears to be performing well. We do not know the age of the concrete but expect the roadway concrete was placed many years ago. The roadway for South Main Street from East 11th Street to the southwest consists of asphalt. The asphalt pavement is performing well with minimal signs of cracking. The roadway surface for East 11th Street between South Main Street and South Nenant Street is asphalt and is performing well with minimal signs of distress such as cracking.

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

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by glacial ice. During the Vashon Stade, areas of the Puget Sound region were overridden by 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. The roadways appear to be slightly raised, indicating that some fill exists.
>
> **Alluvium (Qal)** – Silt, sand and gravel deposited in streambeds and fans; surface relatively undissected; includes some low-level terraces and some lacustrine deposits. Alluvial deposits were not compacted by the weight of the glaciers and exhibit less strength and density.
>
> **Vashon Drift (Outwash) (Qvo)** – Outwash sand, gravel and marginal terrace deposit.

The geologic units for this area are mapped on the Geologic Map of the Centralia Quadrangle, Washington, by Henry W Schasse (U.S. Geological Survey, 1987). The site is mapped as being underlain by alluvium (Qal). Other maps reference Vashon drift (Qvo) in the vicinity of the site.

Explorations

We explored subsurface conditions within the site on February 11, 2016, by drilling two borings with a truck mounted hollow stem auger drill rig. Borings 1 and 2 were drilled to depths of 36.5 and 51.5 feet below ground surface, respectively. Samples were obtained from the borings at 5-foot intervals by driving a split spoon sampler with a 140-pound hammer dropping 30 inches. The number of blows required for penetration of three 6-inch intervals was recorded. To determine the standard penetration number at that depth the number of blows required for the lower two intervals are summed. If the number of blows reached 50 before the sampler was driven through any 6-inch interval, the sampler was not driven further and the blow count is recorded as 50 for the actual penetration distance.

The borings were located in the field by an engineer from this firm who also examined the soils and geologic conditions encountered, and maintained logs of the borings. The approximate locations of the borings 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 borings are presented in Figures 4 through 8.

We also include in Appendix A one water well report from the Washington DOE website. The well is located to the west of the site, approximately ½ mile.

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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 Boring Logs in Figures 4 through 8.

Our explorations generally encountered fine to coarse sand with varying amounts of silt and gravel. The material extended to the depth explored with our borings.

Laboratory Testing

We completed moisture contents on selected samples from our explorations. The moisture contents are shown on the boring logs.

Hydrologic Conditions

Shallow groundwater seepage was encountered at 6 and 7 feet in Boring 1 and 2, respectively. The water gradient appears to have a slight slope downward to the elevation of Skookumchuck River. Therefore, the elevation of the river is controlling the groundwater levels in the near vicinity. We expect the water level represents the regional water table in the area. We expect during high flows of the river, the water table is near the ground surface.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion that the site is compatible with the planned development. The underlying medium dense or better alluvial deposits are capable of supporting the planned box culverts and pavements. We recommend that the foundations for the structures extend through any fill, topsoil, loose, or disturbed soils, and bear on the underlying medium dense or firmer, native 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 planned improvements will be created by raising the road grade by placing structural fill to the desired elevation. The roadway embankments will consist of slopes with an angle of 2 Horizontal to 1 Vertical (2H: 1V) or less or use of retaining walls. The retaining walls could consist of short rockeries, concrete walls or Mechanically Stabilized Earth (MSE) walls with block facing.

Geologic Hazards

Erosion Hazard: The erosion hazard criteria used for determination of affected areas includes soil type, slope gradient, vegetation cover, and groundwater 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. We reviewed the Web Soil Survey by the Natural Resources Conservation Service (NRCS) to determine the erosion hazard of the on-site soils. The site surface soils were classified using the SCS classification system as Spanaway gravelly sandy loam, 0 to 3 percent slopes (Unit 110) and Spanaway gravelly sandy loam, 3 to 15 percent slopes (Unit 111). The corresponding geologic unit for these soils is volcanic ash over gravelly outwash, which is in general agreement with the soils encountered in our site

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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 D 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 B.

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. We used a computer program titled "Liquefy Pro" to estimate the amount of liquefaction based on the soil conditions, groundwater level and different site peak ground accelerations (PGA). We have provided the following table to show the liquefaction potential and anticipated settlement.

Probability	Return Period	PGA %**	Settlement (in)
50% in 50 years	1 in 73 years	14.8	not liquefaction
10% in 50 years	1 in 475 years	39.5	1½
5% in 50 years*	1 in 1,000 years	52.5	1¾
2% in 50 years	1 in 2,475 years	72.6	2

WSDOT typically uses this as a design standard for most State roadway projects.
****Determined from USGS website with PSHA 2008 update.***

The detailed graphs of the analysis are shown in Appendix B. The results of the analysis indicate that slight amount of liquefaction may occur. The amount of settlement during a design shaking event is estimated to range from 1½ to 2 inches, for the 475 to 2,475 year event. Because the subsurface soils in this area are reasonably consistent, we expect a reasonable estimate of the differential settlement is approximately half of the total settlement (i.e., about ¾ to 1 inch across a distance of about 50 feet). The amount of potential settlement is very small considering the magnitude of the earthquake to generate the accelerations.

Site Preparation and Grading

Roadway Area:

Existing Roadway

It is our opinion that fills could be placed on the existing asphalt or concrete pavement to build the roadway up to the existing grade. From a geotechnical perspective the existing road surface does not need to be removed. Therefore, no subgrade preparation needs to occur. The fill can be placed on the existing road surface and should be placed in accordance with the **Structural Fill** subsection below.

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Existing Shoulder

The first step of site preparation in the existing shoulders should be to strip the vegetation, topsoil, or loose soils to expose medium dense or firmer native soils in the area planned for the roadway embankment, retaining walls or road surface. The excavated 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 yield should be repaired prior to placing hard surfaces. The geotechnical engineer should evaluate the exposed subgrade prior to placement of footings or any additional structural fill.

Existing Utility Trenches

The existing utility trenches should be evaluated by the geotechnical engineer to confirm the density of the existing fill if the trench backfill will support the roadway, walls or the embankment. Random amount of density tests could be completed to determine the density and suitability of the fill. The determination whether existing fill can be left in place will be completed by the geotechnical engineer in the field during construction and if the utilities are under the planned improvements.

Foundation Area

The first step of site preparation should be to strip the vegetation, topsoil, or loose soils to expose medium dense or firmer native soils in the area of the box culvert foundations. The excavated 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 yield should be repaired prior to placing hard surfaces. The excavation cuts should follow the recommendation in the **Temporary and Permanent Slopes** subsection below. The geotechnical engineer should evaluate the footing subgrade prior to placement of footings or any additional structural fill.

The interaction between the footings and the existing utilities will be critical. The design plans should have specific details on how the footings will be installed around or above the existing utilities.

Structural Fill

General: All fill placed beneath bridge foundations, 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. We recommend that the imported material should satisfy WSDOT Standard Specification 9-03.14(3) for common borrow.

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The use of on-site soil as structural fill will be dependent on moisture content control and if acceptable by the local jurisdictions. 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.

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 culvert footings, 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 groundwater. 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.

For planning purposes, we recommend that temporary cuts in the native soils be no steeper than 1.5 Horizontal to 1 Vertical (1.5H: 1V). If groundwater seepage is encountered, we expect that flatter inclinations may be necessary. Dewatering may be required depending on the depth of the footings. Dewatering will help the stability of the excavation slopes.

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 granular structural fill and the native soils should be no steeper than 2H:1V. Lightly compacted fills, common fills, or structural fill predominately consisting of fine grained soils 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.

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Culvert Foundations

Shallow spread foundations could be used to support the culvert walls. The footings should be founded on undisturbed, medium dense or firmer soil. If the soil at the planned bottom of footing elevation is not suitable, it should be overexcavated to expose suitable bearing soil. Footings should extend at least 18 inches below the lowest adjacent finished ground surface for frost protection. 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 a design bearing pressure as shown in Figure 9 be used for the footing design. The design chart was created using LRFD standards from the 2012 AASHTO manual.

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. We recommend horizontal surcharge load for traffic of 110 psf should be used for the culvert walls (nonyielding walls). The horizontal load for traffic surcharge can be reduced to 60 psf for active walls (yielding walls).

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 "2008 PSH Deaggregation on NEHRP" to compute the PGAs for the site. The 3-D histograms are included in Appendix B. We used 50% of the PGA calculated from the 475 year event, which correlates to 0.2g. The above drained active and at-rest values should be increased by a uniform pressure of 8.2H and 14.1H 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 0.6 may be used to determine the base friction in the native glacial soils. An equivalent fluid density of 275 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.

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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.

Drainage

We recommend that runoff from impervious surfaces be collected and routed to an appropriate storm water discharge system.

Dewatering may be required depending on the depth of the culvert footings. We expect the water can be handled with pumps inside the excavation. The dewatering method will be up to the contractor during construction.

Pavement

Design: We used the results of our subsurface explorations to design the pavement section for the road in general accordance with the 2015 WSDOT pavement guide. We have conservatively estimated the traffic count by multiplying the number of residences (50) by 4 trips per day to equal approximately 200 trips per day. We have also assumed that 5% of those trips will be trucks. Using a typical pavement design life of 20 years, we have calculated the equivalent single-axle loads (EASLs) to be about 40,000 ESALs. The above values can be reevaluated with more accurate traffic data.

We have estimated the subgrade soils to have a California Bearing Ratio (CBR) of approximately 7. This correlates to a subgrade modulus value of 10,500 pounds per square inch for the on-site soils compacted to structural fill specifications. This CBR value was not based on specific field or laboratory testing of the subgrade material but is estimated from the subsurface soil conditions described in our report. It is our opinion this is a conservative value considering the subsurface soils encountered and the acceptable performance of the existing road surface.

We used the procedures outlined in the 1993 AASHTO Guide for Design of Pavement Structures to design a pavement section for the project.

	Strength Coefficient	Thickness (in)**
Hot-Mix Asphalt (HMA) wearing course	0.44	3
Crushed Surface Base Course (CSBC)*	0.14	6
Compacted Subgrade – Structural Fill*	0.06	12

** 95% compaction based on ASTM Test Method D1557*

***Minimum pavement thickness should satisfy the local government pavement code requirements for these types of roadways.*

CSBC for the pavement section should confirm to WSDOT Standard Specifications 2012 Section 9-03.9(3), Crushed Surfacing. HMA aggregate should conform to WSDOT Standard Specifications 2012 Section 9-03.8(2) for <0.3 ESALs. HMA should be prepared, placed and tested in accordance with WSDOT Standard Specification Section 5-04.

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Preparation: 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 Jerome W. Morrissette & Associates Inc., P.S., 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.

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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.

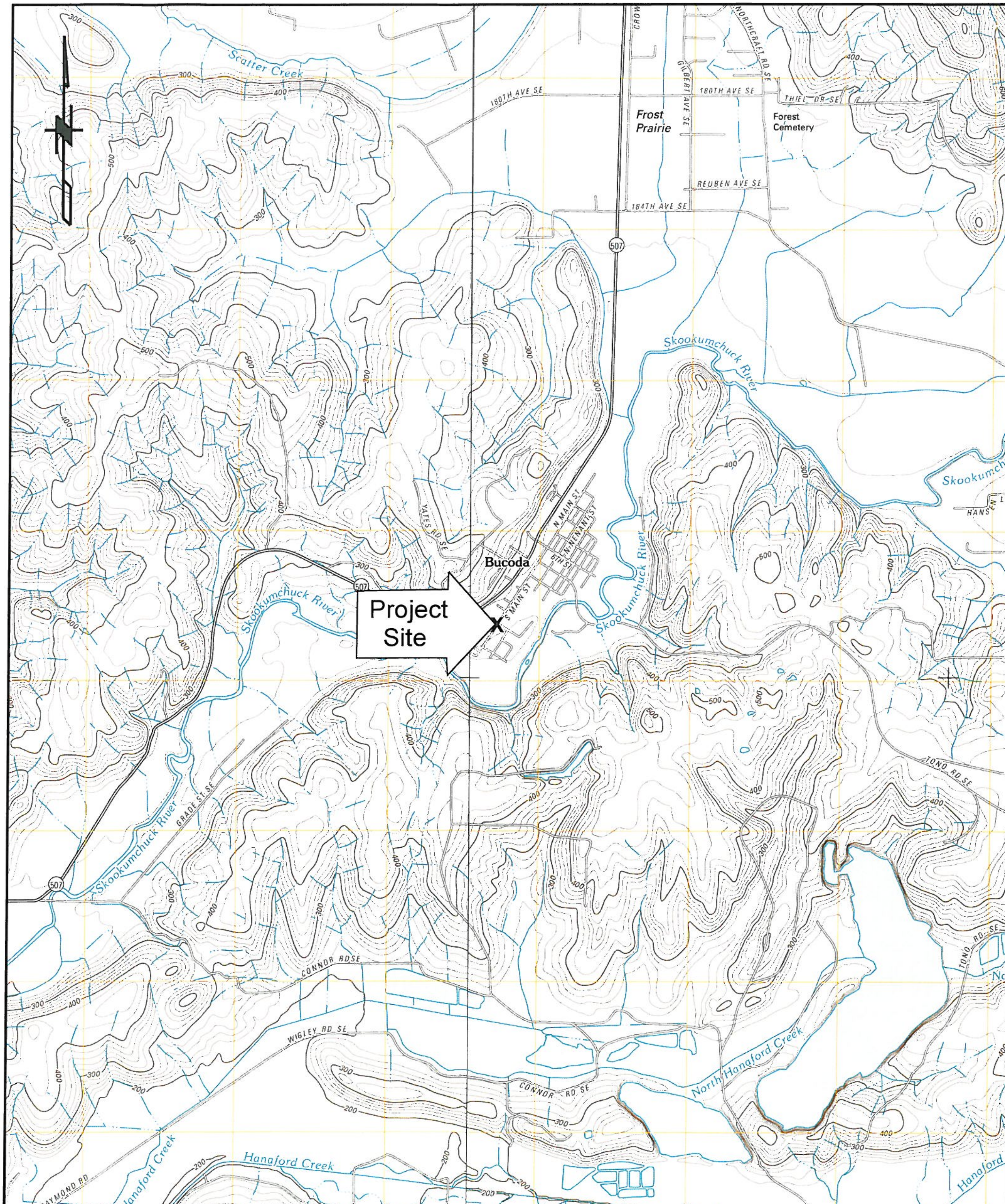
Sincerely,
Robinson Noble, Inc.

Kevin H. Biersner, EIT
Staff Engineer

Rick B. Powell, PE
Principal Engineer

KHB:RBP:am

Nine Figures
Appendix A and B



Note: Basemap taken from Bucoda 7.5-minute series, and Violet Prairie 7.5-minute series. USGS 2011.

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Figure 1
Vicinity Map
Town of Bucoda: Main Street Roadway

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Note: Basemap taken from Thurston County parcel viewer (geodata.org) 2/24/16.

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Figure 2
Site Plan
Town of Bucoda: Main Street Roadway

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50% RETAINED ON NO. 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 2488-93.

* 2) Soil classification using laboratory tests is based on ASTM D 2487-93.

3) Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance, of soils, and/or test data.

* Modifications have been applied to ASTM methods to describe sit and clay content.

$N_{60} = N_M * C_E * C_B * C_R * C_S$

N_M = blows/foot, measured in field

$C_E = ER_m/60$, convert measured hammer energy to 60% for comparison with design charts.

C_B = adjusts borehole diameter

C_R = rod length, adjusts for energy loss in rods

C_S = Sample liner = 1.0

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

KEY TO BORING LOG SYMBOLS

▽ Ground water level

● Blows required to drive sample 12 in. using SPT (converted to N_{60})

MC (■) = % Moisture = $\frac{\text{(Weight of water)}}{\text{(Weight of dry soil)}}$

DD = Dry Density

SM — Letter symbol for soil type

— Contact between soil strata (Dashed line indicates approximate contact between soils)

ML — Letter symbol for soil type

NOTE: The stratification lines represent the approximate boundaries between soil types and the transition may be gradual

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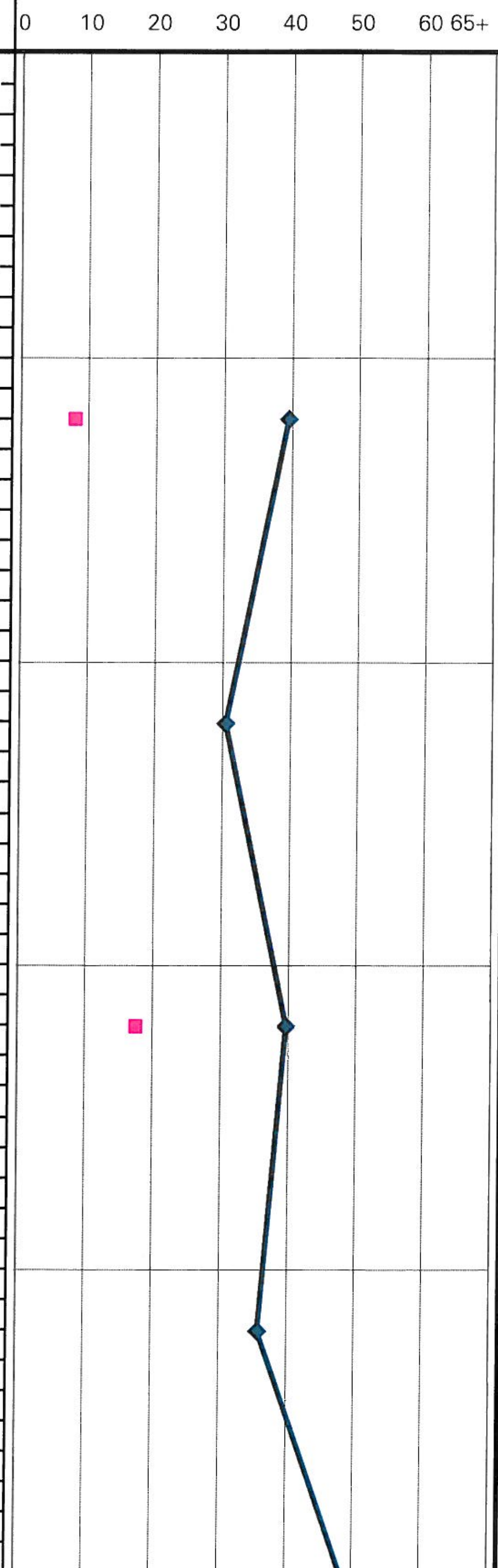
Figure 3

Town of Bucoda: Main Street Roadway

B-1	Date	2/11/2016	Hole dia. (in)	6
	Logged by	KHB	Hole depth ft	36.5
	Driller	Holocene	Well dia. (in)	N/A
Page 1 of 2	Elevation (ft)	-	Well depth	N/A
	Sample Liner	No	Hammer Eff.	86%

Standard Penetration Resistance
(140 lb. weight, 30" drop)
◆ SPT N_{60} (blows/ft)
■ Moisture Content (%)

LITHOLOGY / DESCRIPTION	U.S.C.	Sample Recovery/ Driven Interval (in)	N- Blow Counts (blows/6")	Static Water Level	Depth (feet)
					1
					2
					3
					4
Brown fine to coarse sand with gravel and silt (dense, moist)	SP-SM	6/18	17 20 15	▽	5 6
---?---------?---					7 8 9
Brown silty fine to coarse sand with gravel (medium dense, wet)	SM	4/18	15 13 14		10 11
					12 13 14
Brown silty fine to coarse sand with gravel (dense, moist to wet)	SM	9/18	12 17 14		15 16
---?---------?---					17 18 19
Brown fine to medium sand with gravel (medium dense, moist to wet)	SP	6/18	11 12 14		20 21
---?---------?---					22 23 24 25



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Town of Bucoda - Main Street Roadway

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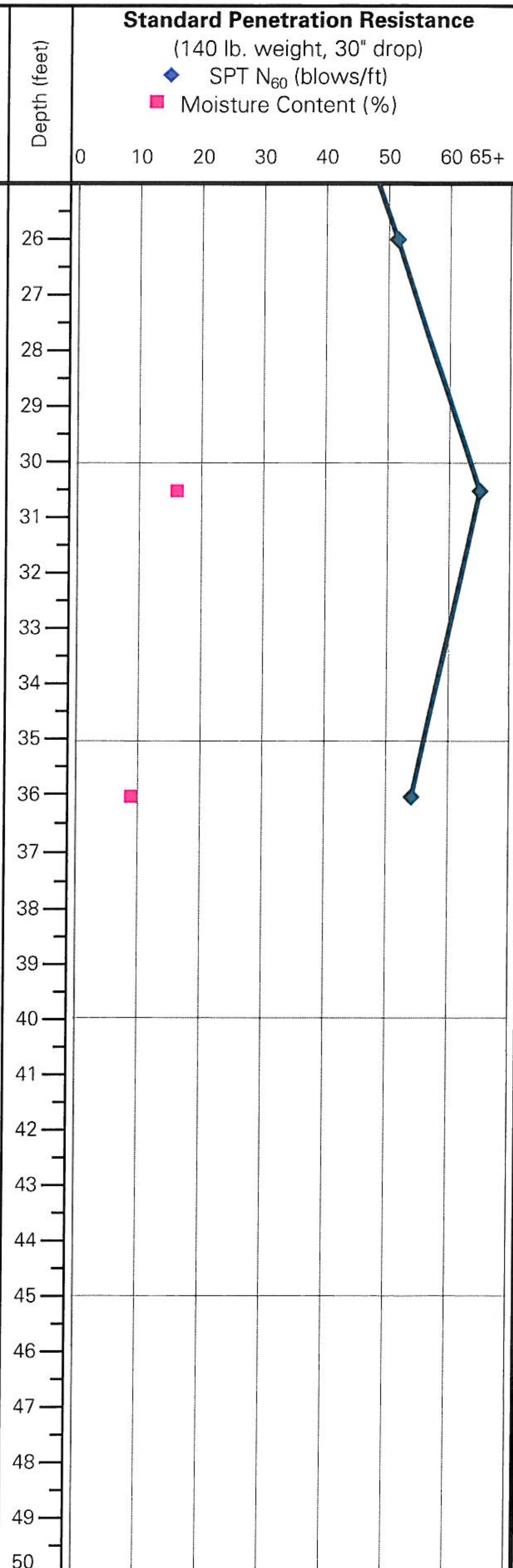
Figure 4

B-1				
	Date	2/11/2016	Hole diameter	6
	Logged by	KHB	Hole depth	36.5
	Driller	Holocene	Well diameter	N/A
Page 2 of 2	Elevation (ft)	-	Well depth	N/A
	Sample Liner	No	Hammer Eff.	86%

LITHOLOGY / DESCRIPTION	U.S.C.	Sample Recovery/ Driven Interval (in)	N- Blow Counts (blows/6")	Static Water Level
Brown gravel with fine to coarse sand (dense, moist)	GW	7/18	12 16 20	
?-----------?				
Gray fine to coarse sand with gravel and silt (very dense, moist to wet)	SP_SM	9/12	18 50/6"	
Brown fine to coarse sand with gravel and silt (dense, moist to wet)	SP-SM	9/18	9 16 20	

Boring completed at 36.5 feet on 2/11/2016
Groundwater observed at 6 feet
Caving observed at 8 feet



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Town of Bucoda - Main Street Roadway

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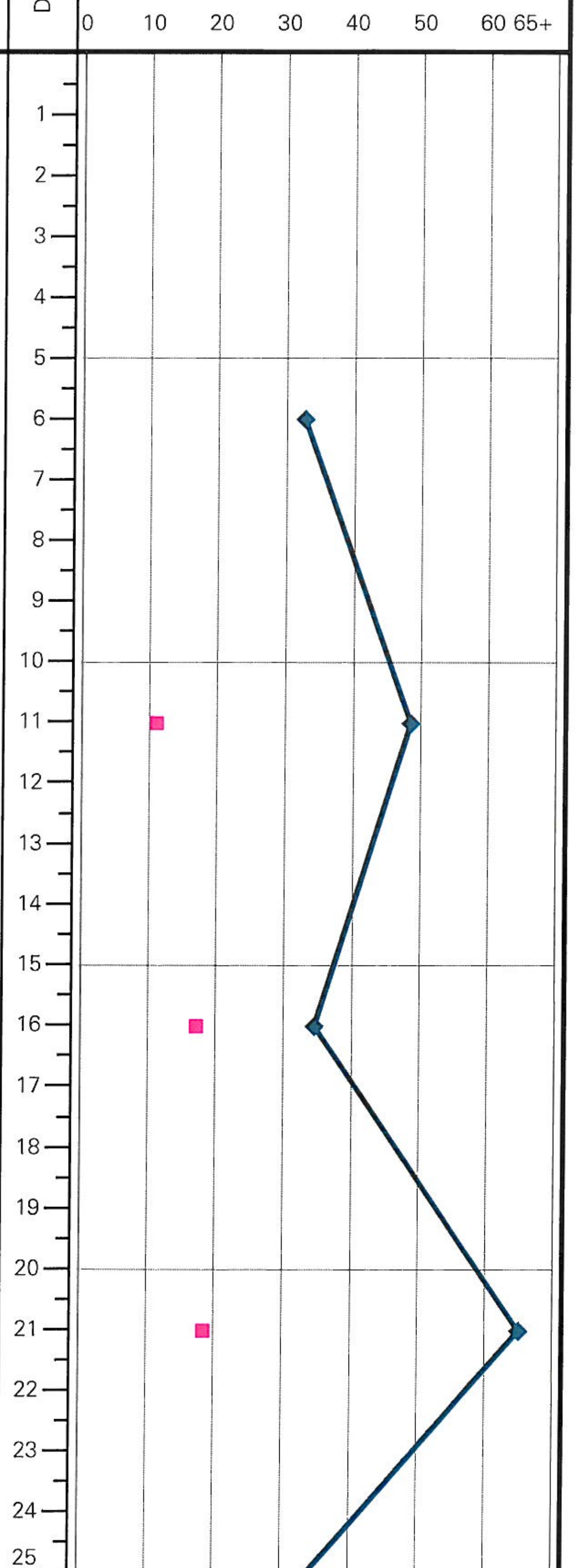
B-2				
	Date	2/11/2016	Hole dia. (in)	6
	Logged by	KHB	Hole depth ft	36.5
	Driller	Holocene	Well dia. (in)	N/A
Page 1 of 3	Elevation (ft)	-	Well depth	N/A
	Sample Liner	No	Hammer Eff.	86%

LITHOLOGY / DESCRIPTION	U.S.C.	Sample Recovery/ Driven Interval (in)	N- Blow Counts (blows/6")	Static Water Level	Depth (feet)
Brown fine to medium sand with gravel and silt (medium dense, moist)	SP-SM	4/18	4 12 17	▽	5–6
Brown fine to coarse sand with gravel and silt (dense, moist)	SP-SM	6/18	20 18 25		10–11
Brown fine to coarse sand with gravel and silt (medium dense, wet)	SP-SM	9/18	15 14 13		15–16
Brown fine to coarse sand with gravel and silt with layer of reddish-brown sand and gravel (very dense, wet)	SP-SM	8/18	22 37 43		20–21

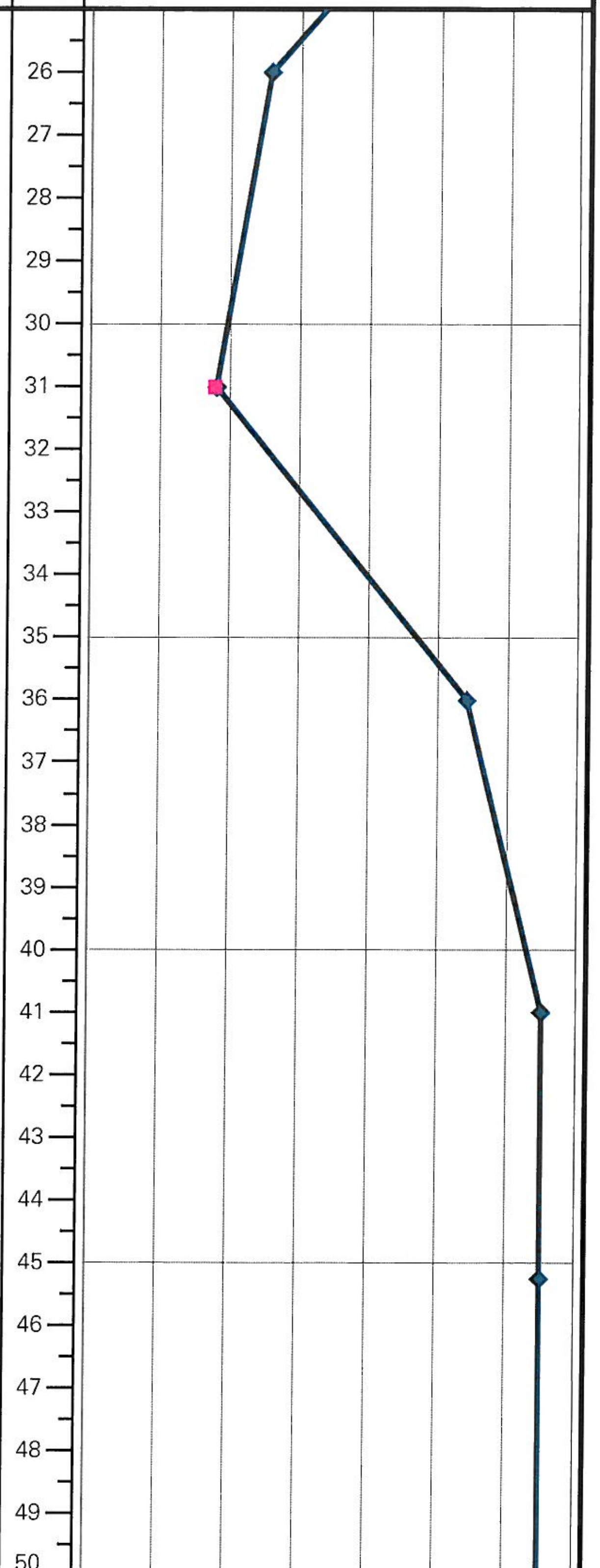
Standard Penetration Resistance
(140 lb. weight, 30" drop)
◆ SPT N_{60} (blows/ft)
■ Moisture Content (%)



B-2	Date	2/11/2016	Hole diameter	6
	Logged by	KHB	Hole depth	36.5
	Driller	Holocene	Well diameter	N/A
Page 2 of 3	Elevation (ft)	-	Well depth	N/A
	Sample Liner	No	Hammer Eff.	86%

LITHOLOGY / DESCRIPTION	U.S.C.	Sample Recovery/ Driven Interval (in)	N- Blow Counts (blows/6")	Static Water Level
no recovery		0/18	4 9 9	
- - - ? - - - - - ? - - -				
Gray fine to coarse sand with gravel (medium dense, moist)	SP	18/18	4 6 6	
- - - ? - - - - - ? - - -				
Gray fine to coarse sand with silt and gravel (dense, moist)	SP-SM	18/18	21 20 16	
- - - ? - - - - - ? - - -				
Gray fine to coarse sand with fine to coarse gravel (dense, moist)	SP	6/18	31 17 30	
no recovery		0/5	50/5"	

Standard Penetration Resistance
(140 lb. weight, 30" drop)
◆ SPT N_{60} (blows/ft)
■ Moisture Content (%)



ROBINSON NOBLE
Phone: 425-488-0599
Fax: 425-488-2330
17625 - 130th Avenue Northeast, Suite 102
Woodinville, Washington 98072

Town of Bucoda - Main Street Roadway

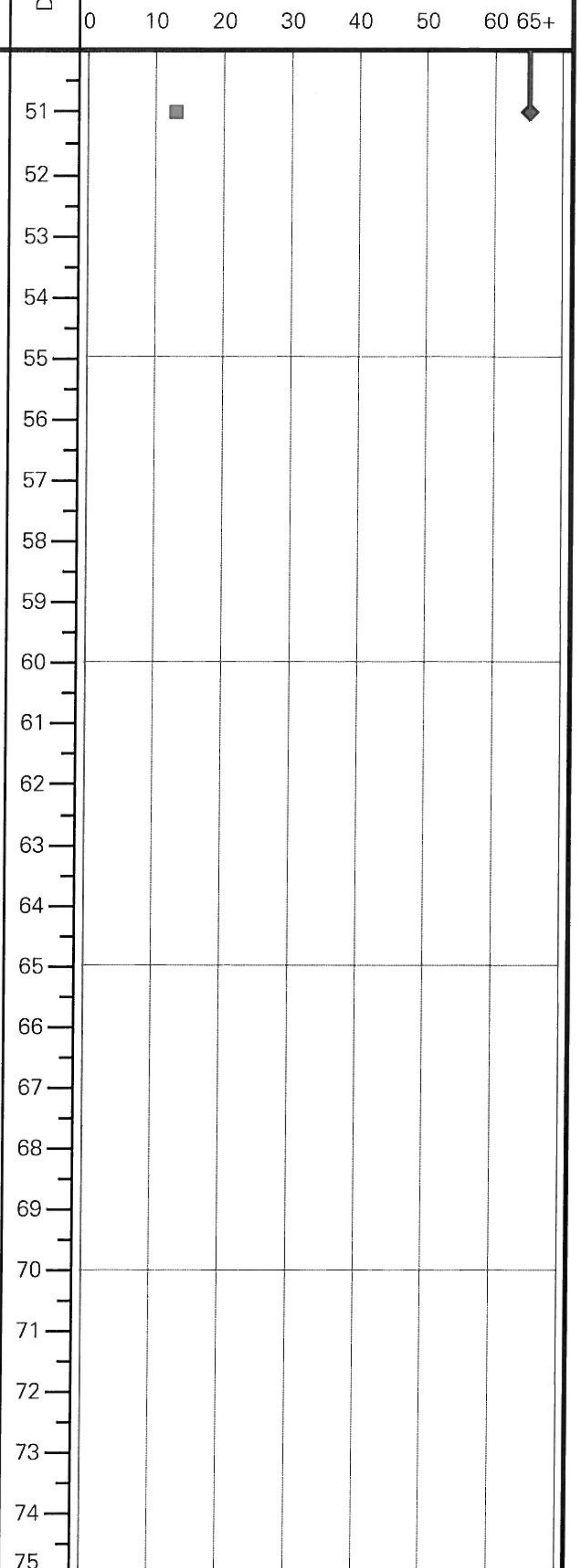
3101-001A | Figure 7

B-2						U.S.C.	Sample Recovery/ Driven Interval (in)	N- Blow Counts (blows/6")	Static Water Level	Depth (feet)	Standard Penetration Resistance (140 lb. weight, 30" drop) ◆ SPT N_{60} (blows/ft) ■ Moisture Content (%)
	Date	2/11/2016	Hole diameter	6							
	Logged by	KHB	Hole depth	36.5							
	Driller	Holocene	Well diameter	N/A							
Page 3 of 3	Elevation (ft)	-	Well depth	N/A							
	Sample Liner	No	Hammer Eff.	86%							

LITHOLOGY / DESCRIPTION

LITHOLOGY / DESCRIPTION	U.S.C.	Sample Recovery/ Driven Interval (in)	N- Blow Counts (blows/6")	Static Water Level
Gray fine to coarse sand with gravel (dense, moist)	SP	9/18	25 20 20	

Boring completed at 51.5 feet on 2/11/2016
Groundwater observed at 7 feet
Caving observed at 14 feet



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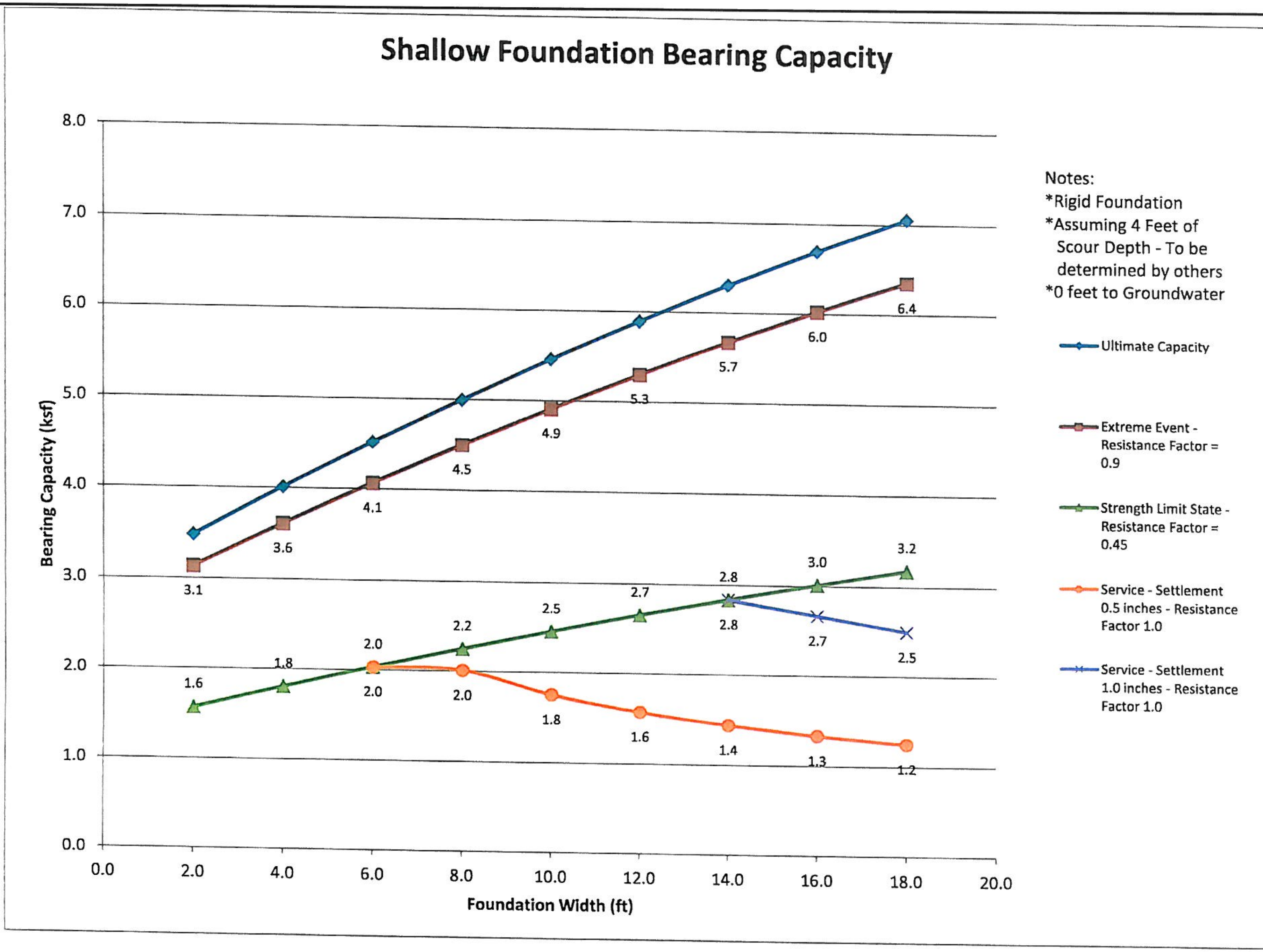


ROBINSON NOBLE	PM: RBP Feb 2016 3101-001A		Figure 9 Bearing Capacity VS Foundation Width Jerome W Morrissette: Town of Bucoda

Appendix A

The Department of Ecology does NOT Warranty the Data and/or the Information on this Well Report.

File Original and First Copy with Department of Ecology
Second Copy — Owner's Copy
Third Copy — Driller's Copy

WATER WELL REPORT

STATE OF WASHINGTON

Start Card No. 066395
UNIQUE WELL I.D. # ____
Water Right Permit No. ____

(1) OWNER: Name Robert Gordon Address PO Box 220 Bucoda, WA.

(2) LOCATION OF WELL: County Thurston SE 1/4 SW 1/4 Sec 12 T. 15 N., R. 2W W.M.

(2a) STREET ADDRESS OF WELL (or nearest address) @ 20104 Bucoda Hwy

(3) PROPOSED USE: [x] Domestic [ ] Irrigation [ ] DeWater Industrial [ ] Test Well [ ] Municipal [ ] Other [ ]

(4) TYPE OF WORK: Owner's number of well (If more than one) 1
Abandoned [ ] New well [x] Deepened [ ] Reconditioned [ ] Method: Dug [ ] Cable [ ] Rotary [x] Bored [ ] Driven [ ] Jetted [ ]

(5) DIMENSIONS: Diameter of well 6 inches.
Drilled 79 feet. Depth of completed well 79 ft.

(6) CONSTRUCTION DETAILS:
Casing installed: 6 " Diam. from +1 ft. to 74 ft.
Welded [x] ____ " Diam. from ____ ft. to ____ ft.
Liner installed [ ] ____ " Diam. from ____ ft. to ____ ft.
Threaded [ ]

Perforations: Yes [ ] No [x]
Type of perforator used ____
SIZE of perforations ____ in. by ____ in.
____ perforations from ____ ft. to ____ ft.
____ perforations from ____ ft. to ____ ft.
____ perforations from ____ ft. to ____ ft.

Screens: Yes [x] No [ ]
Manufacturer's Name WESCO
Type Wire Model No. ____
Diam. 6 Slot size 14 from 74 ft. to 79 ft.
Diam. ____ Slot size ____ from ____ ft. to ____ ft.

Gravel packed: Yes [ ] No [x] Size of gravel ____
Gravel placed from ____ ft. to ____ ft.

Surface seal: Yes [x] No [ ] To what depth? 18 ft.
Material used in seal Bentonite
Did any strata contain unusable water? Yes [ ] No [x]
Type of water? ____ Depth of strata ____
Method of sealing strata off ____

(7) PUMP: Manufacturer's Name F&W
Type: Sub H.P. 1 1/2

(8) WATER LEVELS: Land-surface elevation above mean sea level 200 ft.
Static level 13 ft. below top of well Date 4-9-96
Artesian pressure ____ lbs. per square inch Date ____
Artesian water is controlled by ____ (Cap, valve, etc.)

(9) WELL TESTS: Drawdown is amount water level is lowered below static level
Was a pump test made? Yes [ ] No [x] If yes, by whom? ____
Yield: ____ gal./min. with ____ ft. drawdown after ____ hrs.
" " " "
" " " "

Recovery data (time taken as zero when pump turned off) (water level measured from well top to water level)

Time	Water Level	Time	Water Level	Time	Water Level

Date of test 4-9-96
Bailer test 30 gal./min. with 2 ft. drawdown after 1 hrs.
Airtest ____ gal./min. with stem set at ____ ft. for ____ hrs.
Artesian flow ____ g.p.m. Date ____
Temperature of water ____ Was a chemical analysis made? Yes [x] No [ ]

(10) WELL LOG or ABANDONMENT PROCEDURE DESCRIPTION

Formation: Describe by color, character, size of material and structure, and show thickness of aquifers and the kind and nature of the material in each stratum penetrated, with at least one entry for each change of information.

MATERIAL	FROM	TO
Sandy loam w/ sand (CUT)	0	7
Clay (tan) w/ sand	7	14
Sand-clay-gravel	14	31
Compact sand-gravel-clay	31	59
Soupy sand-(w/ water) (reddish)	59	64
Sand-clay	64	70
WB Sand w/gravel	70	79

Work Started 4-8, 1996 Completed 4-9, 1996

WELL CONSTRUCTOR CERTIFICATION:

I constructed and/or accept responsibility for construction of this well, and its compliance with all Washington well construction standards. Materials used and the information reported above are true to my best knowledge and belief.

NAME TIMS WELL DRILLING (PERSON, FIRM, OR CORPORATION) (TYPE OR PRINT)
Address 6246 Libby Rd NE Olympia
(Signed) ____ (WELL DRILLER) License No. 832
Contractor's Registration No. TIMSWD1131PE Date 6-1, 1996

(USE ADDITIONAL SHEETS IF NECESSARY)

Ecology is an Equal Opportunity and Affirmative Action employer. For special accommodation needs, contact the Water Resources Program at (206) 407-6600. The TDD number is (206) 407-6006.

ECY 050-1-20 (9/93)

Appendix B

USGS Design Maps Summary Report

User–Specified Input

Report Title Town of Bucoda
Thu February 18, 2016 20:13:48 UTC

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

Site Coordinates 46.79493°N, 122.87257°W

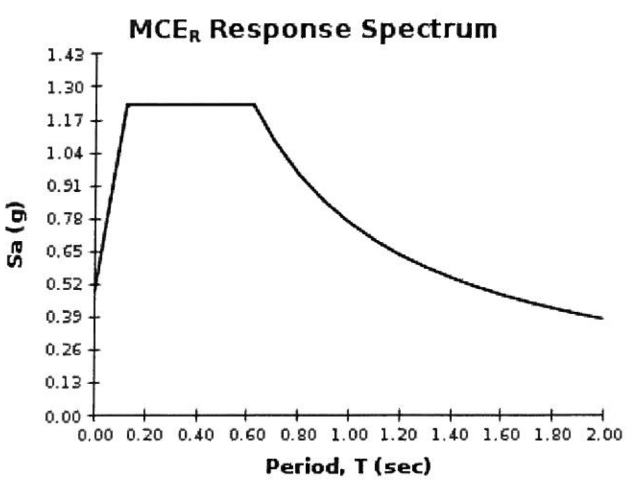
Site Soil Classification Site Class D – "Stiff Soil"

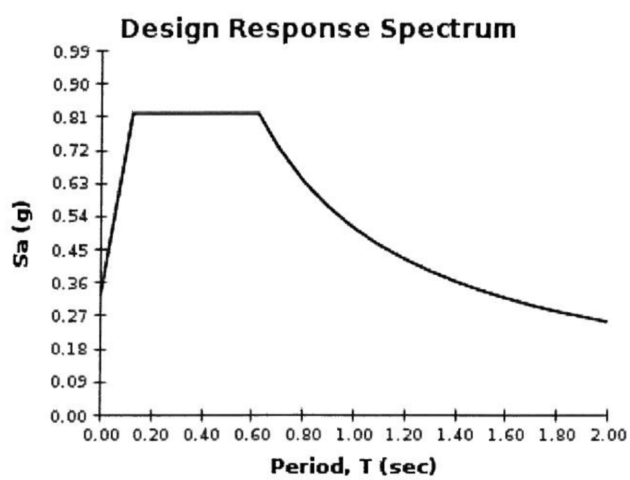
Risk Category I/II/III

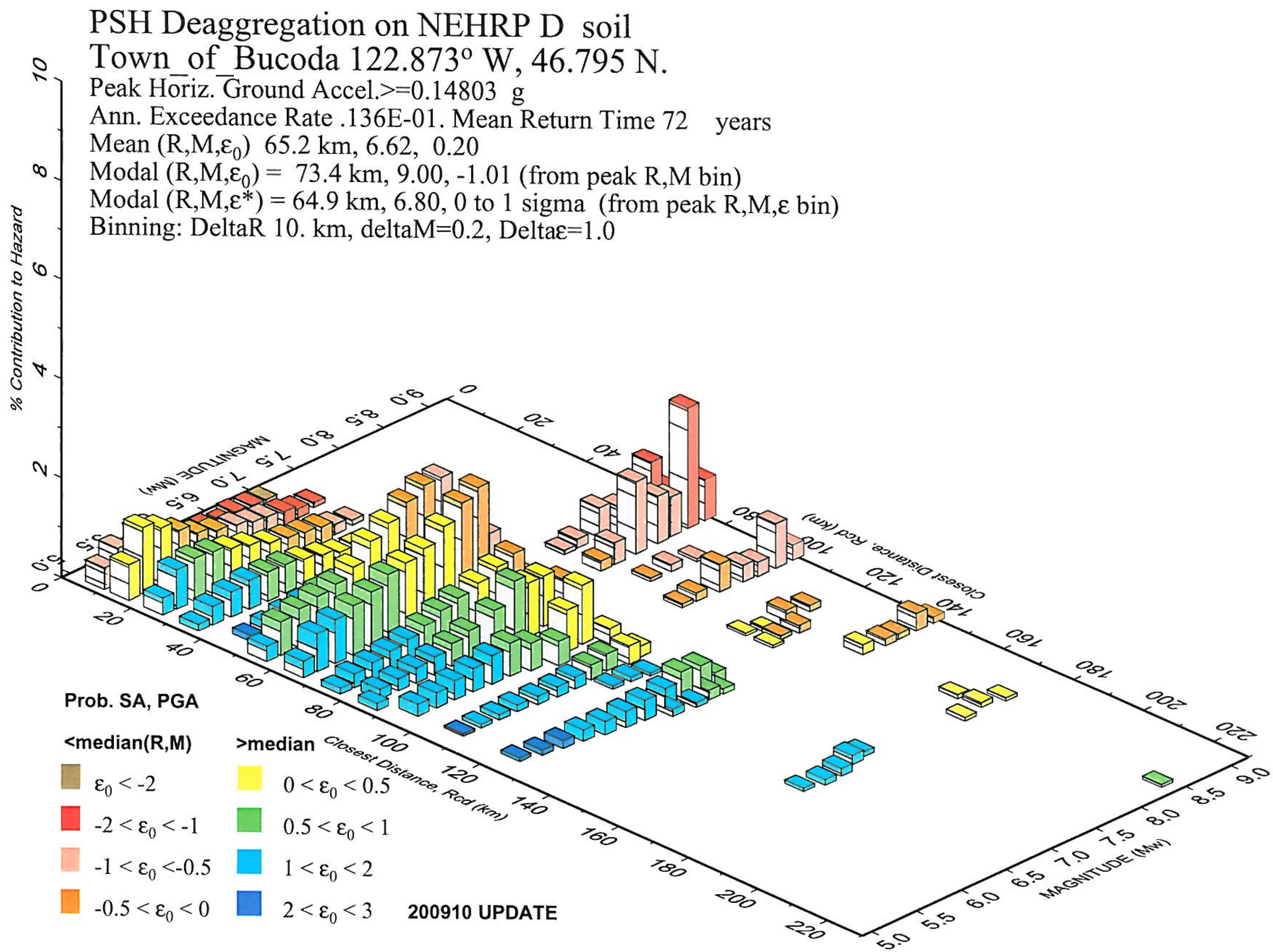
USGS–Provided Output

S_S =	1.211 g	S_{MS} =	1.230 g	S_{DS} =	0.820 g
S_1 =	0.508 g	S_{M1} =	0.763 g	S_{D1} =	0.508 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.







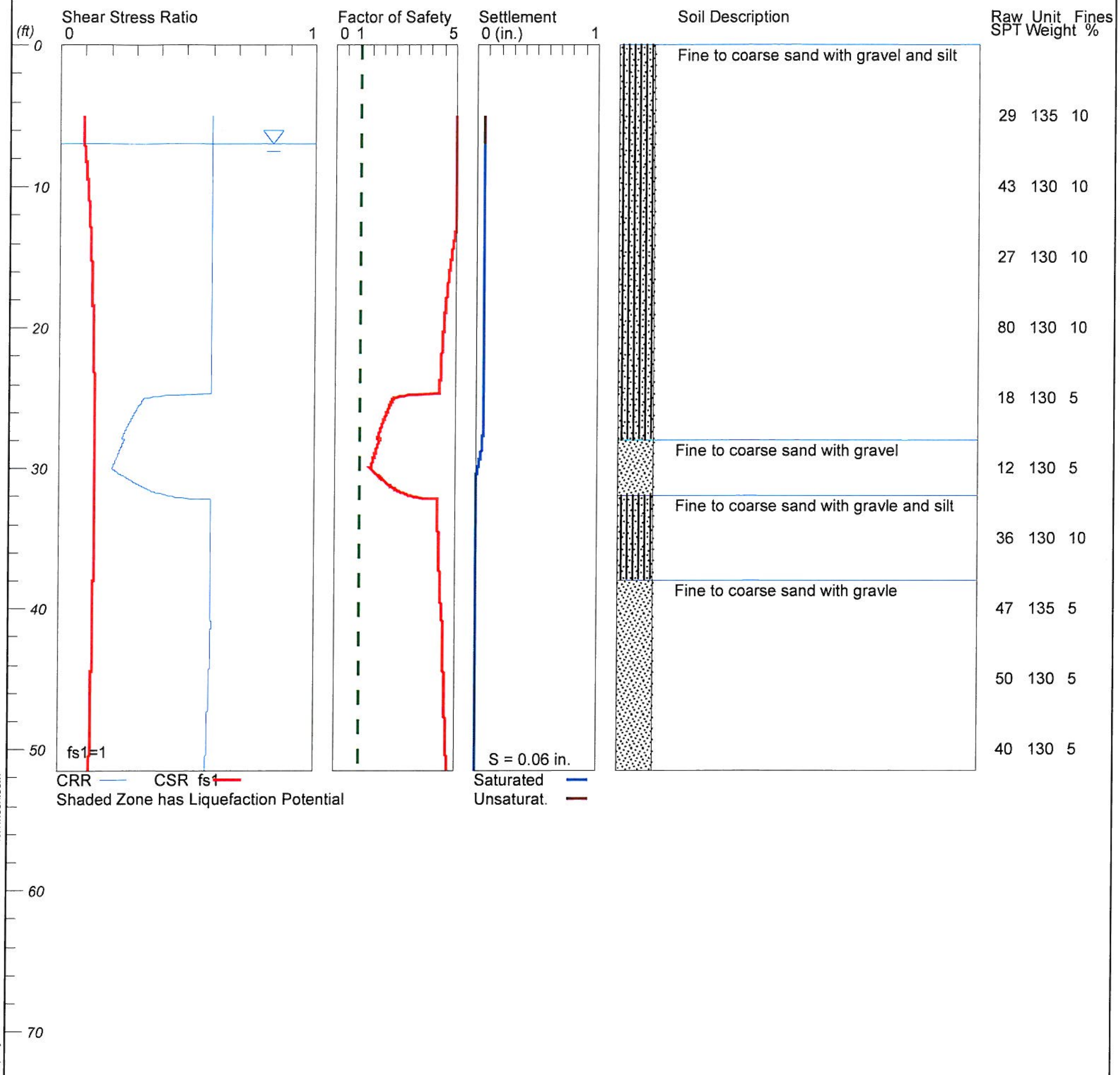
GMT 2016 Feb 24 02:01:12 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 190. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

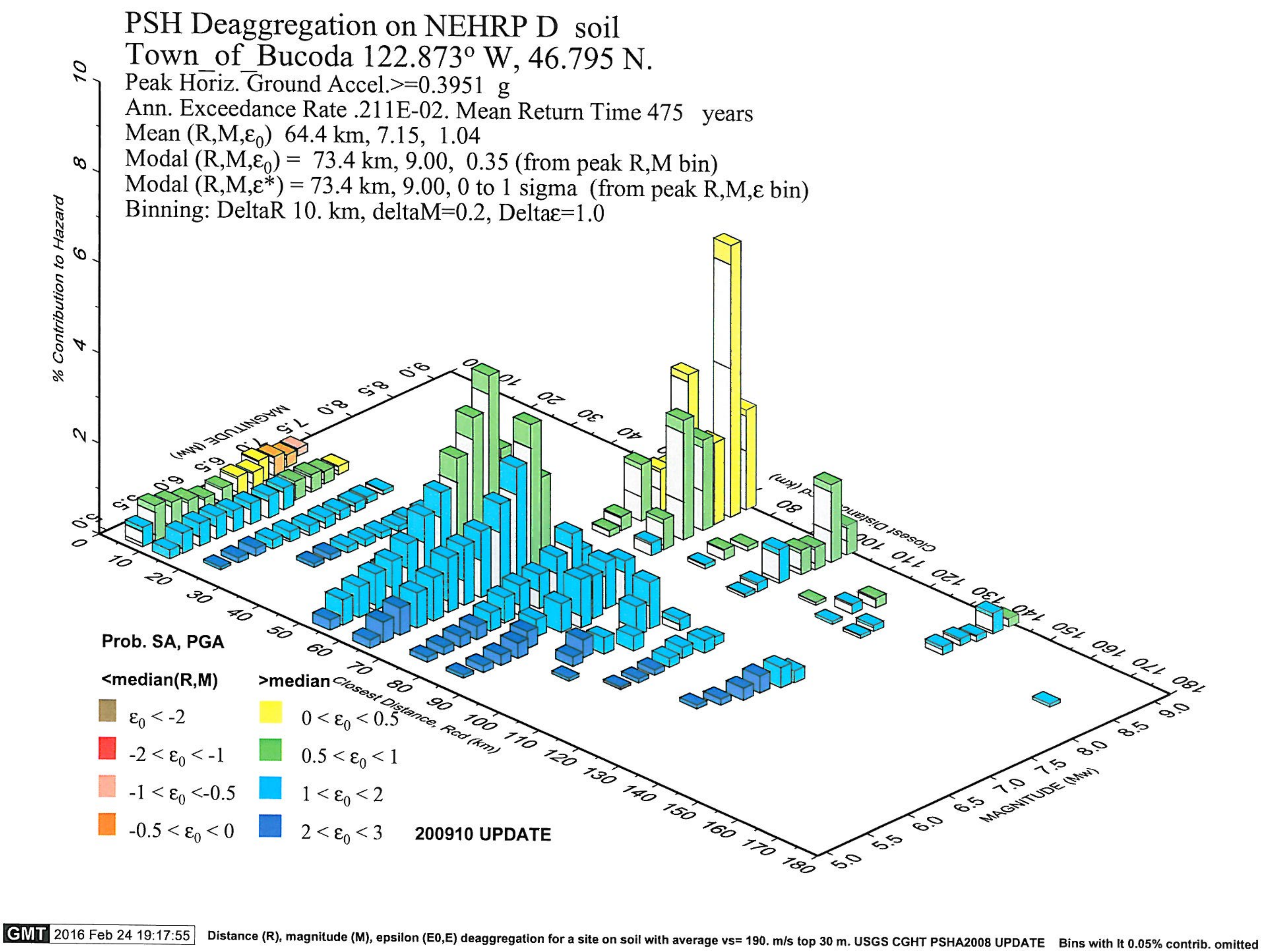
LIQUEFACTION ANALYSIS

Town of Bucoda

Hole No.= Water Depth=7 ft

Magnitude=7
Acceleration=.1480g



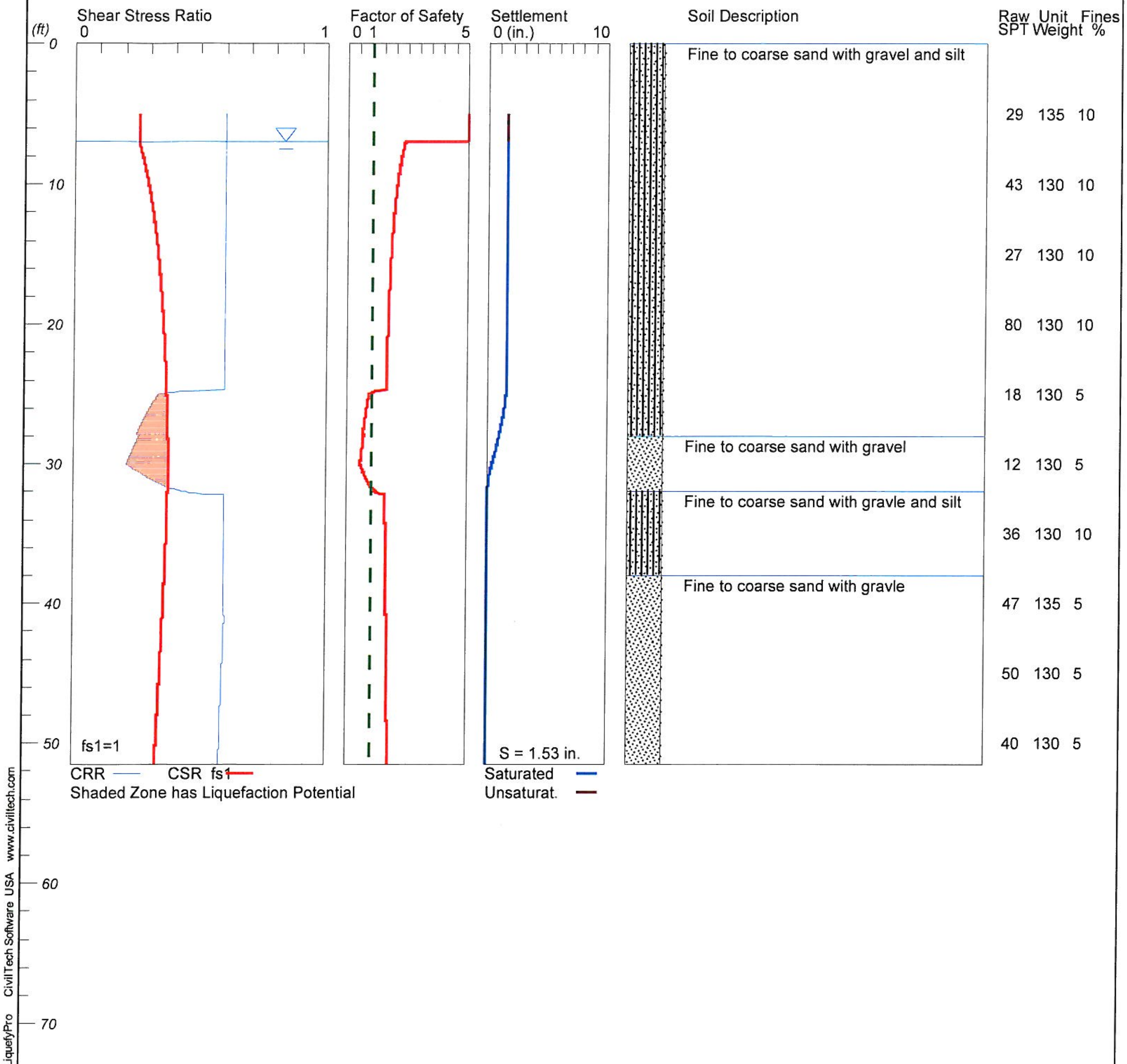

PSH Deaggregation on NEHRP D soil
Town_of_Bucoda 122.873° W, 46.795 N.
Peak Horiz. Ground Accel.>=0.3951 g
Ann. Exceedance Rate .211E-02. Mean Return Time 475 years
Mean (R,M,ε0) 64.4 km, 7.15, 1.04
Modal (R,M,ε0) = 73.4 km, 9.00, 0.35 (from peak R,M bin)
Modal (R,M,ε*) = 73.4 km, 9.00, 0 to 1 sigma (from peak R,M,ε bin)
Binning: DeltaR 10. km, deltaM=0.2, Deltaε=1.0
% Contribution to Hazard
MAGNITUDE (Mw)
Closest Distance, Rcd (km)
Prob. SA, PGA
<median(R,M)
>median
ε0 < -2
-2 < ε0 < -1
-1 < ε0 <-0.5
-0.5 < ε0 < 0
0 < ε0 < 0.5
0.5 < ε0 < 1
1 < ε0 < 2
2 < ε0 < 3
200910 UPDATE
GMT 2016 Feb 24 19:17:55
Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 190. m/s top 30 m. USGS CGHT PSHA2008 UPDATE
Bins with lt 0.05% contrib. omitted

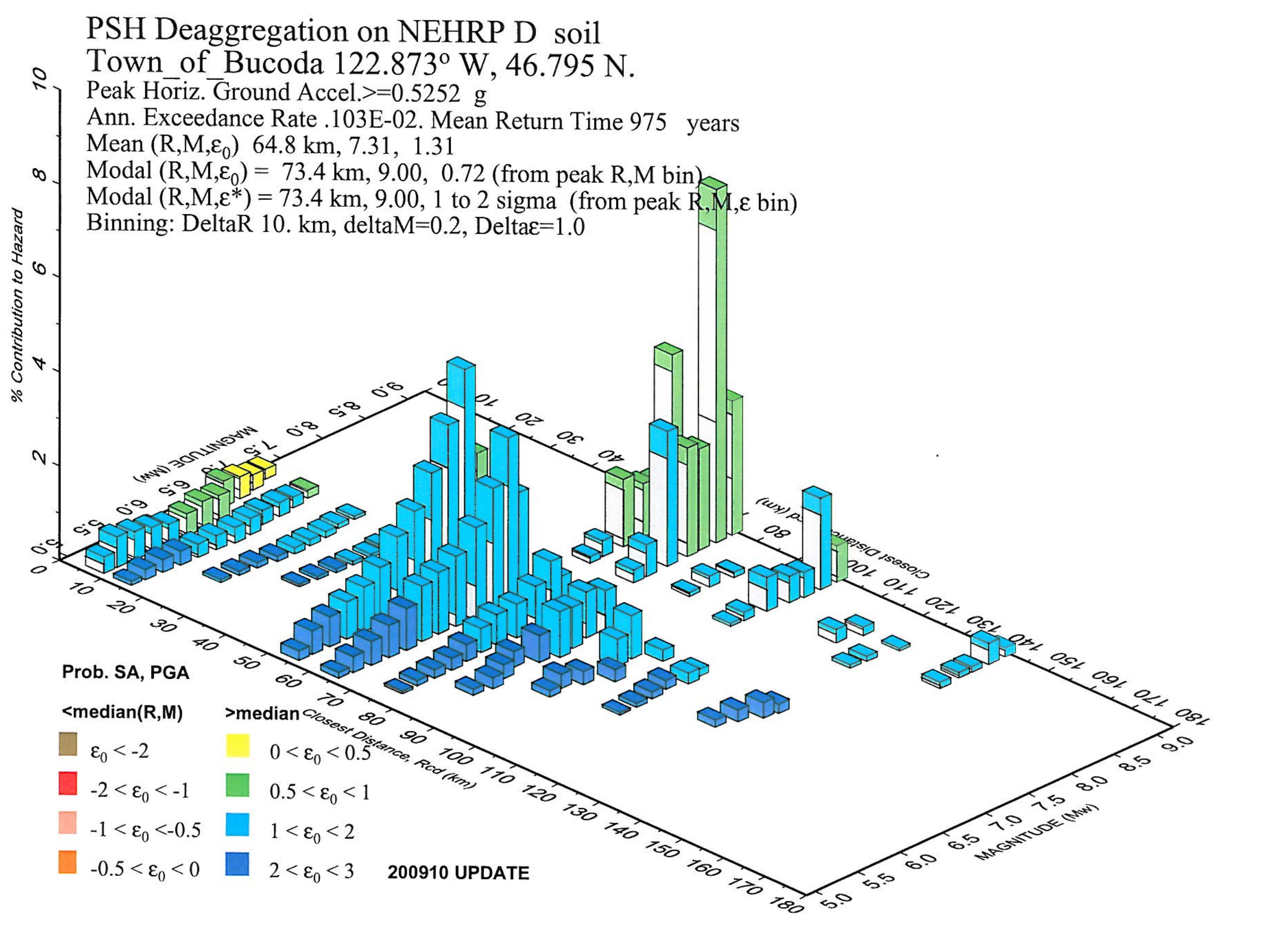
LIQUEFACTION ANALYSIS

Town of Bucoda

Hole No.= Water Depth=7 ft

Magnitude=7
Acceleration=.3951g





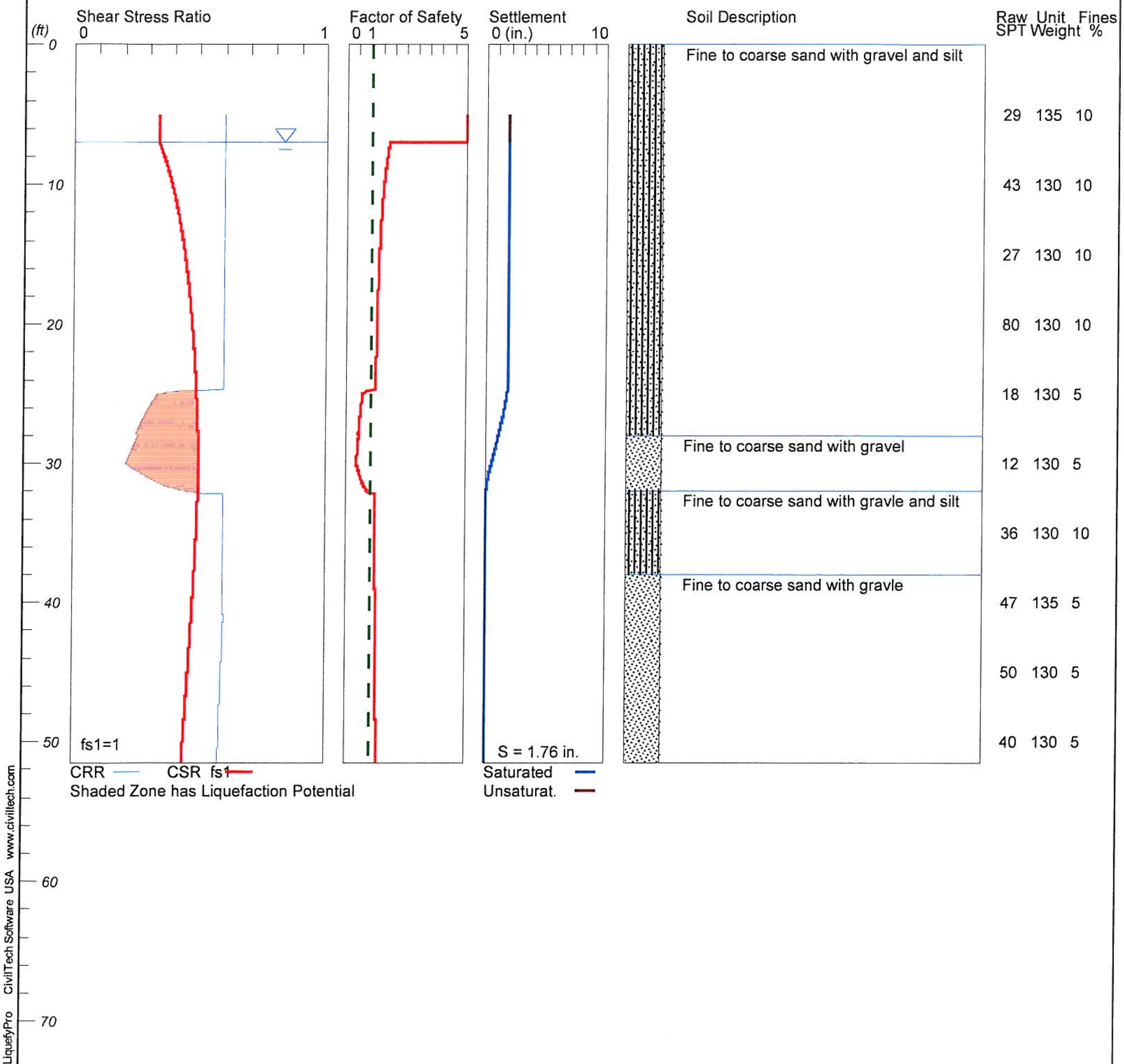
GMT 2016 Feb 24 18:22:23 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 190. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

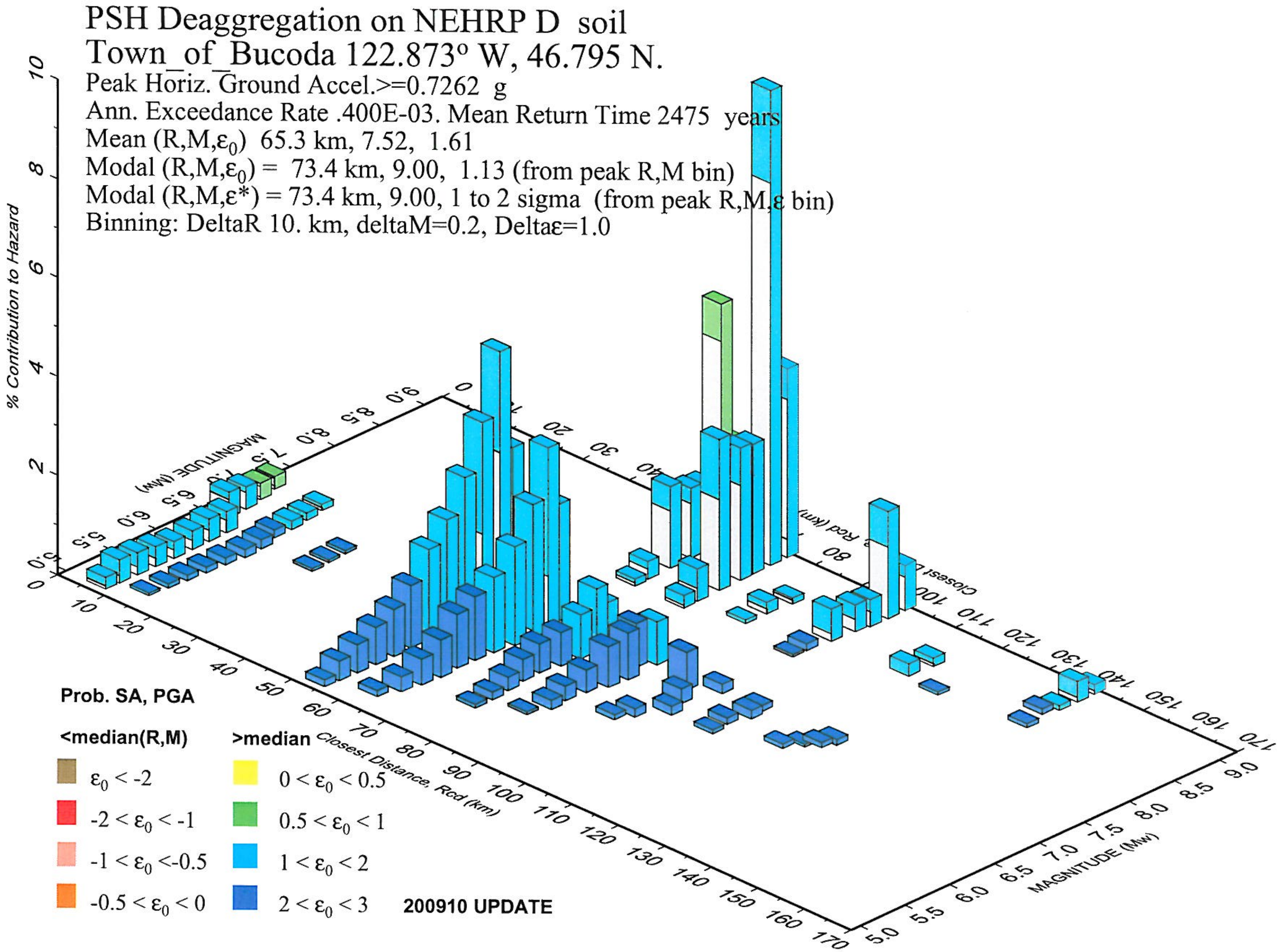
LIQUEFACTION ANALYSIS

Town of Bucoda

Hole No.=   Water Depth=7 ft

Magnitude=7
Acceleration=0.525g





GMT 2016 Feb 24 19:17:24 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on soil with average vs= 190. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

LIQUEFACTION ANALYSIS

Town of Bucoda

Hole No.= Water Depth=7 ft

Magnitude=7
Acceleration=.7262g