GEOTECH
CONSULTANTS, INC.

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Seattle, Washington 98102
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September 1, 2022

JN 22302

Tyler Simpson
via email: *tyler_simpson@hotmail.com*

Subject: **Critical Area Study and Update to Previous Geotechnical Report**
Proposed Remodel and Expansion of Existing Residence
6454 East Mercer Way
Mercer Island, Washington

Reference: *Subsurface Exploration, Geologic Hazard, and Geotechnical Engineering Report,* subject property; Associated Earth Sciences (AESI); November 12, 2019.

Dear Mr. Simpson:

This report is intended to respond to comments by the City of Mercer Island's geotechnical third-party reviewer, who has required an updated geotechnical report to address the aspects of the planned development, as well as seismic-related issues covered under the International Building Code. In order to prepare this updated report, we have:

1. Reviewed the above-referenced report by AESI,
2. Visited the subject site in August 2022 to observe the current conditions,
3. Completed an analysis of potential ground settlement and lateral spreading in the event of the Maximum Considered Earthquake (MCE), and
4. Reviewed our previous work on nearby sites having similar topography and geologic conditions, specifically the planned redevelopment of the property at 6610 East Mercer Way, two lots south of your property.

Project Description:

We have been provided with a copy of the June 2, 2022 architectural plans prepared by Sturman Architects, and the March 21, 2022 structural plans completed by SSF Structural Engineering. Based on this information, we expect that the existing residence will undergo an extensive renovation that will include an expansion of the footprint, as well as a new second floor addition to much of the expanded structure. The northern approximately two-thirds of the existing house, north of the current garage, will be maintained. The existing foundations for this portion of the structure will be underpinned with small-diameter pipe piles to support the loads from the existing structure and the new second story. The new construction on the southern end of the residence, which will include a new garage and an outdoor living space, will also be supported on pipe piles. The notes on sheet S2.0 indicate that all piles will be 4 inches in diameter. These piles will be driven to refusal in dense, non-liquefiable soils, providing protection against unacceptable foundation settlement under static conditions and small to moderate earthquakes.

The new garage slab is indicated to be structurally supported on pipe piles.

Only shallow excavations are expected for the underpinning of the existing foundations, and the grade beams for the new foundations.

Seismic Considerations

In accordance with the 2018 International Building Code (IBC) (ASCE 7-16), the site class within 100 feet of the ground surface is best represented by Site Class Type F (Failure-Prone Site Class). However, the code allows for an exception from the F classification if the building period is less than 0.5 seconds. We anticipate the proposed residence will have a structural period of less than 0.5 seconds, and therefore a Site Class Type E (soft soil profile) can be used for the project. This will need to be confirmed by the project structural engineer. As noted in the USGS website, the mapped spectral acceleration value for a 0.2 second (S_s) and 1.0 second period (S_1) equals 1.45g and 0.50g, respectively.

The near-surface soils beneath the site consisted of saturated silty sand, sand, and silt containing various amounts of organics. These soils, designated as Lake Deposits in AESI's report have been demonstrated to have a moderate to high potential for liquefaction (soil strength loss) during a large earthquake. The IBC and ASCE 7 require that the potential for liquefaction (soil strength loss) be evaluated for the peak ground acceleration of the Maximum Considered Earthquake (MCE), which has a probability of occurring once in 2,475 years (2 percent probability of occurring in a 50-year period).

The Lake Deposits are underlain at a depth of approximately 25 feet by glacially-compressed soils, which are not susceptible to liquefaction in the event of the MCE.

Mitigation of potential hazards associated with seismic liquefaction and lateral spreading are discussed below in the Seismic Hazard Area section.

CRITICAL AREAS STUDY (MICC 19.07)

Potential Landslide Hazard Area: The entire subject site is located within a mapped Potential Landslide Hazard area. The site is essentially flat and is well set back from any steep slopes. As such, the potential for slope instability on the site is negligible. The mapping of the Potential Landslide Hazard Area is apparently due to the inference by geologists that the site lies within an ancient landslide. However, we observed no signs of landslide debris in our previous nearby borings, and none were noted in AESI's boring logs. Consistent with many lots in this area along the shore of Lake Washington, the topography is the result of alluvial deposits and sediments from Lake Washington, which covered the subject property until the Montlake Cut was constructed in the early 1900s. To our knowledge, no recent large-scale movement has been documented in this area.

The site is located over 400 feet from any significant steep slopes that would be prone to instability. This setback is more than sufficient to protect the planned development from any future instability on these distant slopes. No additional measures, such as buffers or landslide catchment walls, are needed. The proposed development will not adversely impact the stability of the steep slopes to the west.

Seismic Hazard Area: The entire subject site is located within a mapped Seismic Hazard area.

> Liquefaction: The proposed development will be supported on deep foundations embedded into glacially compressed soils that are not liquefiable, due to their dense nature. However, the loose soils between the water table and the glacially-compressed soils are susceptible to liquefaction in the event of a large earthquake. The depth and lateral extent over which

liquefaction could occur are impossible to accurately predict, due to unknowns related to the magnitude, duration, and predominant direction of shaking associated with future earthquakes, as well as variabilities in the soil composition.

From previous experience, as well as liquefaction analyses we have conducted for this project, we know that it at least partial liquefaction beneath the site and surrounding area is possible during the Maximum Considered Earthquake (MCE) with a 1-in-2,475-year probability. This liquefaction could occur between the groundwater table (5- to 7-foot depth) and the dense soils, which were found at an approximate depth of 25 feet. Considering the variability in the gradation of the alluvial soils, it is most likely that liquefaction would occur within the saturated layers of sand and silty sand, which are interbedded with silt, typically thought to have a low potential for liquefaction.

We utilized NovoLIQ to confirm that liquefaction of the soil underlying the water table is likely to occur in the MCE, which is a low probability event. The results of our liquefaction analyses are attached. NovoLIQ estimates that a total of approximately 6 inches of ground settlement is possible following widespread liquefaction extending to a depth of 25 feet. The results of this analysis are attached.

The compressive capacity of pipe piles is entirely dependent on end bearing in the dense to very dense glacially-compressed soils they are driven into. The potentially liquefiable soils encountered in the borings below the water table will provide no vertical support to the pipe piles in the event of seismic liquefaction. For a 4-inch-diameter pipe with an allowable 10-ton allowable capacity, an ultimate capacity in excess of 20 tons is achievable in static conditions. This has been verified by thousands of load tests conducted in the Seattle area over the last 20 years. Conservatively assuming a skin friction of 300 psf on the pile in the upper approximately 4 feet of non-liquefiable soils, a downdrag load of approximately 1,570 pounds could be applied to the pile. This would allow a residual ultimate compressive capacity of at least 38,430 (19.2 tons). For this short-term loading condition, that would still provide a safety factor in excess of 1.9, which is acceptable for a full-scale seismic event.

As a part of our work for the study on this property, we have reviewed recent geotechnical reports prepared for recent developments of waterfront lots to the north and south of the site. These reports similarly recommend the use of pipe piles driven into dense soils to support the homes.

<u>Lateral Spreading:</u> The potential for lateral spreading during a large earthquake, which is essentially a flow slide of the liquefied soil toward a free face (sloped bottom of Lake Washington), is even less understood than liquefaction itself. However, some methods have been developed to estimate the potential amount of lateral ground movement that could occur where liquefiable sites lie next to sloping free face conditions, such as the sloped bottom of Lake Washington. NovoLIQ utilizes several different methods to develop estimates for this lateral movement using five different methods. The results, which are attached, indicate that lateral ground movement of at least 5 to 10 feet could theoretically occur in the MCE. Having completed similar computations before by hand, we know that large values such as this are common for lakefront projects with more than a few feet of liquefiable soil beneath them.

Unfortunately, as with liquefaction, there is no accurate method for determining where, and to what extent, lateral spreading could occur. Even more involved methods, such as Finite

Element Analyses, are still approximate at best, as they rely on a multitude of assumptions about soil properties and potential characteristics of the design earthquake.

Based on the available information, significant lateral ground movement could occur during the MCE. The risk of this is no higher than on nearby waterfront properties that are underlain by similar loose soils and which have recently been developed with new homes. The theoretical lateral movements are large enough, and could extend to such a significant depth, that no pile system, drilled or driven, can prevent ground movement from occurring, or can withstand the potential lateral movements without shearing off.

Improving the ground beneath the site to prevent liquefaction and/or lateral spreading is infeasible for a waterfront residential site within a large area of potentially liquefiable soils, such as this one. Improving the resistance of the granular soils to liquefaction using stone columns, densification, or a similar method would involve strong ground vibrations, which would cause ground settlement and likely damage to neighboring properties, structures, and utilities. No localized ground improvement system on an isolated residential lot can resist the significant lateral soil loads that would result from liquefaction and lateral spreading of the upper 25 of soil that could affect a large area including both the site and adjacent properties. It would be necessary to prevent liquefaction and lateral spreading in the loose soils extending far onto neighboring properties to the north, south, and west to prevent lateral movement within the house footprint on the subject site, which is not practical.

The appropriate mitigation against foundation collapse in the event of lateral spreading is achieved by the reinforced grade beams or mat slab that interconnects the piles. In the event that the ground moves sideways a sufficient distance to bend or break the piles, the grade beams/mat slab would serve to hold the structure in one piece, even if it tilts a significant amount.

Erosion Hazard: Due to the site's very gentle topography, it is not mapped as an Erosion Hazard area. Regardless, properly installed and maintained temporary erosion control measures will be a part of the planned construction. This is necessary to avoid adverse impacts to adjoining properties and to prevent silty runoff from flowing into Lake Washington.

Buffers and Mitigation: As noted above, the entire site lies within a mapped Potential Landslide Hazard Area. However, excluding lateral spreading, the potential for either a shallow landslide affecting the subject property is negligible. As a result, a buffer or other forms of mitigation are not necessary to protect the planned development from potential landslides. The recommendations presented in AESI's geotechnical report and this addendum letter are intended to allow the project to be constructed in the proposed configuration without adverse impacts to critical areas on the site or the neighboring properties. The geotechnical recommendations associated with foundations will mitigate any potential hazards associated with the Seismic Hazard.

Statement of Risk: In order to satisfy the City of Mercer Island's requirements, a statement of risk is needed. As such, we make the following statement:

Provided the recommendations in this report are followed, it is our professional opinion that the recommendations presented in AESI's report and this addendum for the planned alterations will render the development as safe as if it were not located in a geologically hazardous area, and will not adversely impact critical areas on adjacent properties.

Please contact us if you have any questions, or if we can be of further assistance.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

09/01/2022

Marc R. McGinnis, P.E.
Principal

Attachment: NovoLiq Output

cc: **Sturman Architects**
via email: brad@sturmanarchitects.com

MRM:kg

Soil Liquefaction Analysis Report

Geotech Consultants, Inc.

Project : Simpson Residence
Project No. : 22302
Client : Tyler Simpson
Site Address : 6434 East Mercer Way, Mercer Island, Washington 98040

Borehole : BH-1
Total Depth : 29 ft
Water Level : 4 ft
Calculated By :
Reviewed By :

Table i : Input Data and Assumptions

Input Assumption	Setting
Field Test Type :	Standard Penetration Test (SPT)
Apply All Corrections to SPT?	True
Groundwater Level (ft) =	4
Earthquake Magnitude M =	7.1
Magnitude Scaling Factor (MSF) :	1.15 (Idriss, 1997 -NCEER)
Fines Content Correction :	(according to user settings)
Depth Reduction Factor (Rd) :	Idriss 1999, Golesorkhi 1989
Relative Density (Dr) Estimation :	Idriss & Boulanger, 2003
Site Topography :	Gently Sloped : 3.5 %
Ground Improvement Feature :	None
Peak Ground Acceleration PGA (g) =	0.682

Table ii : CRR Calculation Methods

CRR Formula	Selected?
NCEER Workshop (1997)	True
Boulanger & Idriss (2014)	True
Vancouver Task Force (2007)	False
Cetin et al. (2004)	False
Chinese Code	False
Seed et al. (1983)	False
Japanese Highway Bridge Code	False
Tokimatsu and Yoshimi (1983)	False
Shibata (1981)	False
Kokusho et al. (1983)	False

Table iv : Field Tests

Depth (ft)	SPT Blow Counts(N)
2.5	1
4	1
7.5	6
10	14
15	5
20	11
25	53
29	50

Table iii : Subsurface Soil Layers

Layer Thickness (ft)	Soil Type	Unit Weight (lb/ft3)	Fines Content (%)	D50 (mm)	Check Liquefaction	Su (ksf)
10	Silt	105	65	0.02	True	0
15	Sand	110	25	0.3	True	0
5	Sand	135	15	1	True	0

Table v : Post-Liquefaction Displacements

Type	Method	Movement (inch)
Lateral Spreading	Youd et al., 2002	52
Lateral Spreading	Barlett & Youd, 1992	60
Lateral Spreading	Hamada et al., 1986	105
Vertical Settlement	Ishihara & Yoshimine, 1992	6

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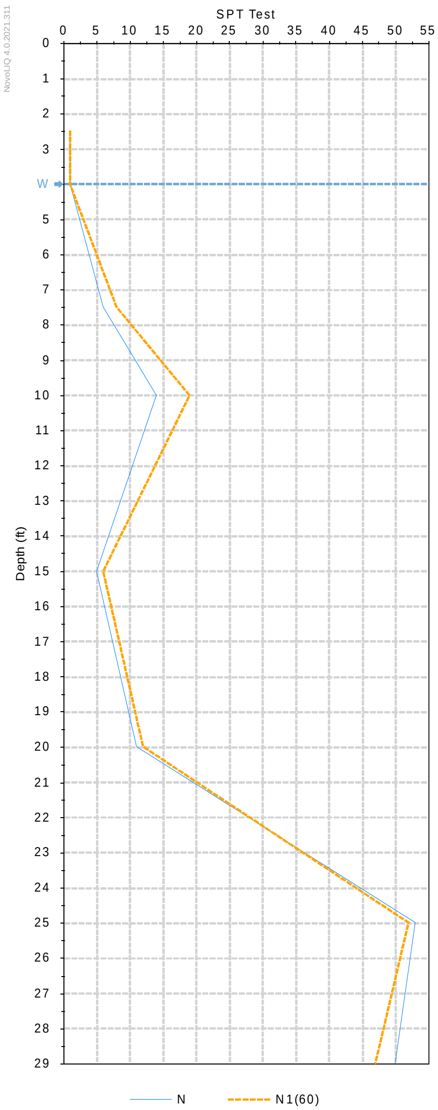

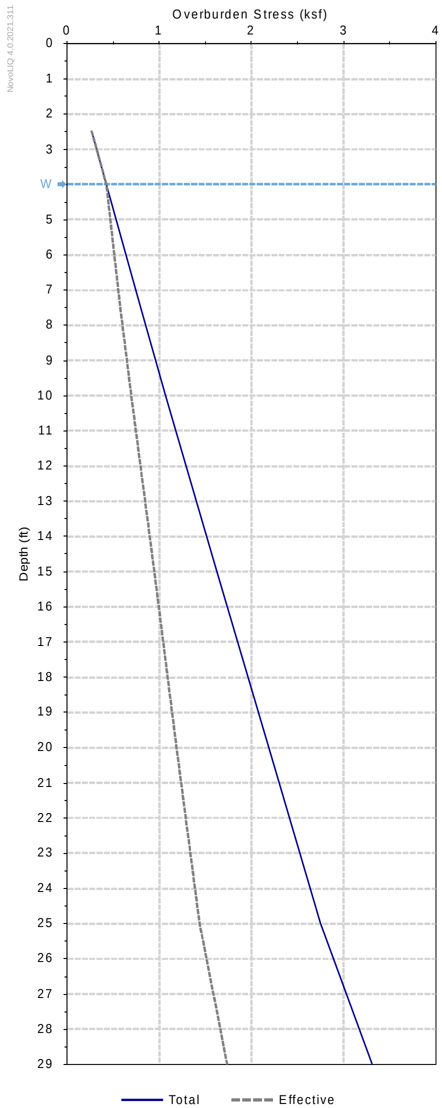

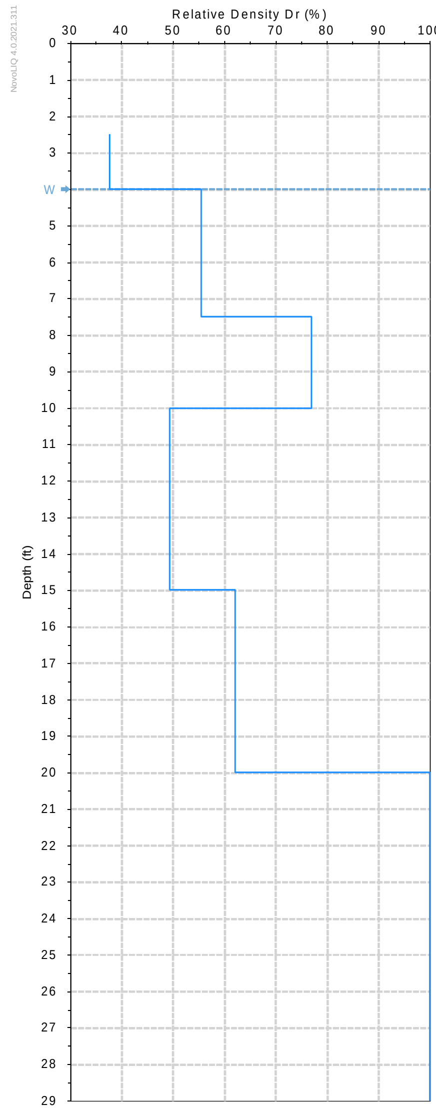

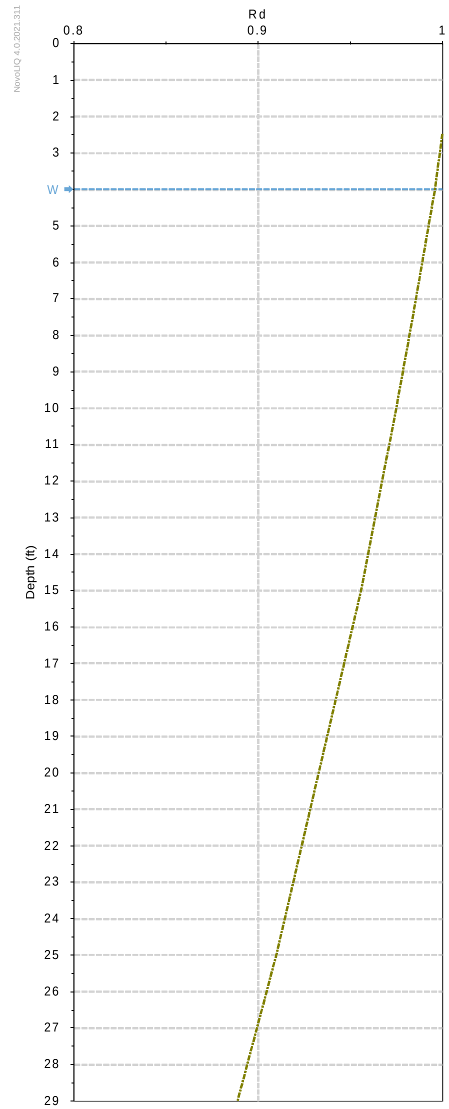

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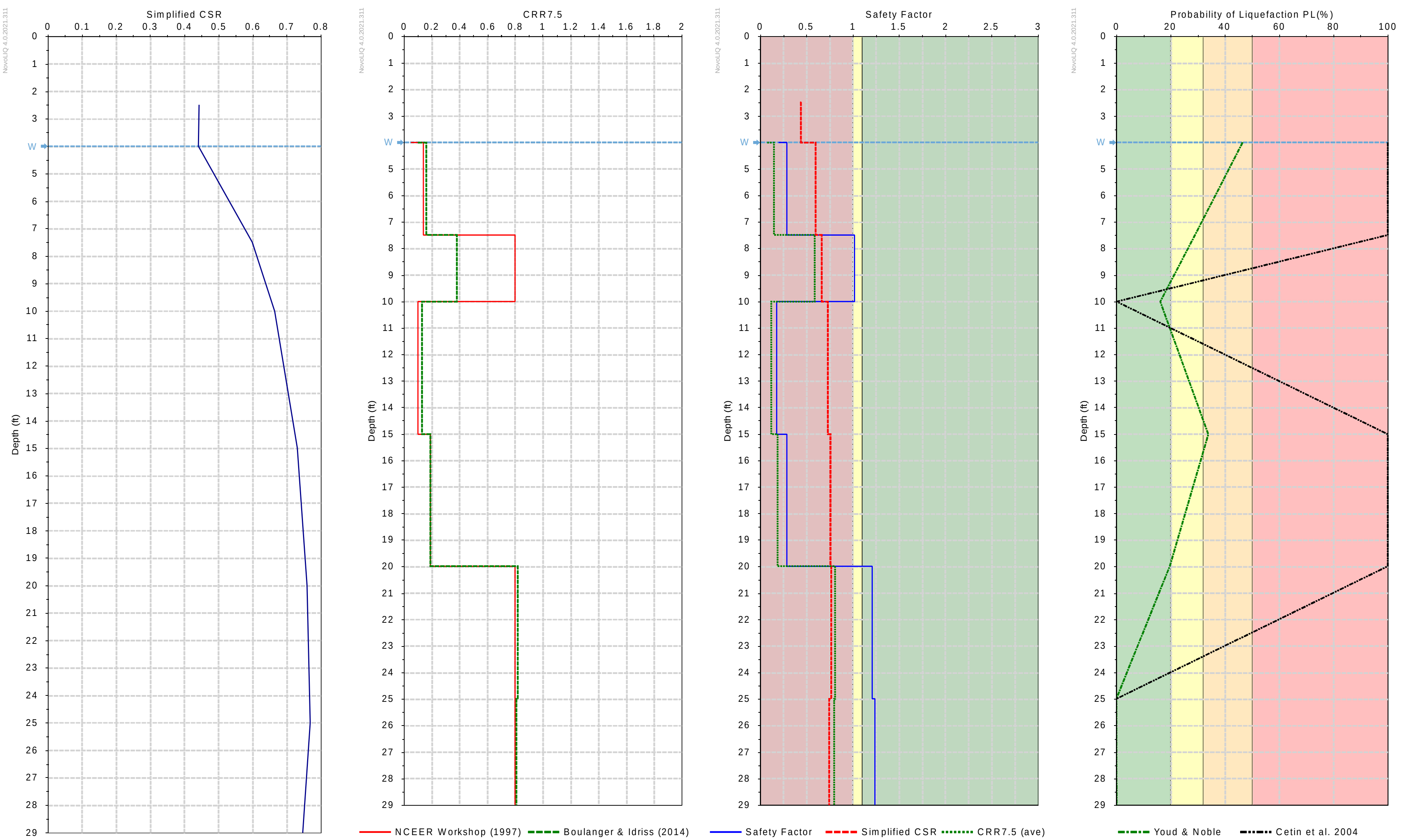

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