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MEMORANDUM

DATE: March 15, 2024

TO: Maile Intrachat and Hoa Hoang

C/O: Tanya Nachia Somanna – Warm Modern Living

FROM: Khal Shawish, PE
Alex Rinaldi, LG, EIT

KHALED M. SHAWISH
STATE OF WASHINGTON
35215
REGISTERED
PROFESSIONAL ENGINEER

3/15/24

RE: Intrachat Hoang Residence
7929 East Mercer Way
Mercer Island, Washington
NGA File No. 1276521

This memorandum presents our responses to the geotechnical peer review performed by the City of Mercer Island regarding the proposed residence development located at **7929 East Mercer Way on Mercer Island, Washington.**

We previously prepared a geotechnical report for the proposed development dated January 14, 2022, and a first-round comment response memorandum dated January 11, 2024. For use in preparing this memorandum, we were provided with a plan set titled "Intrachat Hoang Residence," dated November 27, 2023. Additional geotechnical peer review comments were provided based on the updated plans and our initial comment response.

On the following pages, we summarize the geotechnical comments followed by our responses.

Geotechnical Peer Review Comment Response

Comment 1: *On page 2 of NGA response provided in 01/11/24 memo, the following was noted: "An acceleration of 0.3g was utilized in the modeling and is reflective of a 50 percent reduction in the peak ground shaking, due to excess pore water pressure generation and subsequent soil stiffness softening."*

IBC and ASCE 7-22 indicate the use of MCE ground motion to evaluate liquefaction potential.

"The potential for liquefaction, seismically-induced permanent ground displacement, and soil strength loss evaluated for site peak ground acceleration, earthquake magnitude, and source characteristics consistent with the MCEG peak ground acceleration. Peak ground acceleration shall be determined based on either (1) a site-specific study taking into account soil amplification effects as specified in Section 11.4.7 or (2) the value of the MCEG peak ground acceleration parameter PGAM from the USGS Seismic Design Geodatabase for the applicable site class."

Please revise the liquefaction potential calculations and post-liquefaction settlement estimates to use MCE loading instead of the current 50% reduced value.

Response 1: The liquefaction analysis has been revised to include the maximum considered earthquake geometric mean value of 0.687g, as a site-specific seismic evaluation has not been performed for this site. The results indicated up to 11.36-inches of liquefaction induced settlement. The results of the analysis are attached to this document as Plate A-1.

Comment 2: *IBC requires an assessment of potential consequences of liquefaction. Lateral soil movement is one of the potential consequences.*

With residual strength values assigned to liquefied soils, even gently sloping terrain could be subject to flow failures or lateral spreading. The potential impact to the proposed residence as well as adjacent properties is required in order to support the statement of risk (MICC 19.07.160.B.3) required for approval of the building permit.

Please provide basis for the residual strengths used in slope stability analyses to identify potential post-liquefaction lateral spreading or flow failures. Please provide stability cross section showing results and supporting calculations for the estimated soil deformations in a report addendum.

Response 2: Guidance on selection of residual strength values for soils expected to be liquefaction susceptible were provided by the USGS Technical Report titled "Probabilistic Residual Shear Strength Criteria for Post-Liquefaction Evaluation of Cohesionless Soil Deposits." The research and discussion in this report indicates the available liquefied shear strength values for cohesionless soil deposits can be correlated to the minimum standard penetration test N-value. Utilizing the equation provided for average liquefied shear strength and a minimum corrected N-value obtained from the previous drilling, we determined shear strength value to be approximately 185 psf. As such, an undrained shear strength model was defined for the granular deposits below the groundwater table and the above value input. The water table was elevated by approximately 4-feet from the observed elevation in the modeling to reflect potential rise during seismic loading. The relatively loose granular deposits encountered above the groundwater table were assigned a 30-degree friction angle with no cohesion based on SPT correlations. The lateral spreading or flow failure potential for this site was modeled in the limit equilibrium slope stability software Slide2 by Rocscience, as shown in Figure 2. Because of the relatively high peak ground acceleration for this site, a Newmark Displacement Analysis was utilized to assess the expected downslope deformation in the event of such a seismic scenario for better risk management. A seismic record scenario was input into the program to run this analysis and consisted of the Nahanni, Canada 1985 earthquake which defines a peak ground acceleration of 0.978g. As such, a scaling factor was applied to the analysis to adjust for site conditions. The analysis computes the cumulative earth displacement in the downslope direction for all seismic accelerations during the seismic record that cause the slope stability to fall below a factor of safety of 1. The results indicated approximately 6.815 inches of total displacement, which is considered tolerable given the deep foundation support and unlikely nature of such an event. The probability of this magnitude of failure was assessed by the above mentioned USGS technical report by using equation 4-15 which considers the N-value correlated liquefied shear strength of the soils and the flow failure demand, which is equivalent to the cyclic stress ratio (CSR). The average CSR along the liquefiable soil profile shown on the revised liquefaction analysis (Plate A-1) was determined by the software as 0.52. With these values we computed a probability of flow failure of less than 1 percent.

Comment 3: *The NGA response gives the basis of the pseudostatic coefficient, however, please review the calculations since the results do not agree with general discussion presented in NHI 11-032. For example, on NHI page 6-8, "Values of β less than 1.0 in Equation 6-3 and in Figure 6-3 would be typically associated with seismic conditions in the eastern United States, firm ground conditions, and lower acceleration levels, while values of β greater than 1.0 would be associated with the Western U.S., higher accelerations, and Site Class C or D site conditions." Beta value used by NGA is 0.734.*

NHI Figure 6-3 indicates that the scaling factor, alpha, should be at least 0.5 for a beta value of 1 and slope height of 100 feet. NGA indicates an alpha value of 0.367.

Please review and revise submitted calculations and revise stability analyses to reflect revised pseudostatic coefficient. Provide results in report addendum.

Response 3: The calculations were revised and attached to this document as Figure 3. Accordingly, the pseudostatic coefficient of horizontal ground motion selected for the slope stability analyses is 0.19g. A revised slope stability analysis was provided utilizing Slide2 by Rocscience and resulted in very similar results with respect to size and shape of slide and with a factor of safety for sliding of 0.995. The revised slope stability analysis is provided in Figure 1.

CLOSURE

We recommend that NGA be retained to provide construction monitoring and consultation during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities comply with project plans and specifications.

Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering practices in effect in this area at the time this letter was prepared. No other warranty, expressed or implied, is made. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner.

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REFERENCES

- *Heim, A. 1932. Bergsturz und Menschenleben. Fretz and Wasmuth Verlag, Zurich. 218 pp. (English translation by Skermer, N.A. 1989. Landslides and Human Lives. BiTech Publishers,Vancouver. p. 195).*

- *Van Gassen, W. and Cruden, D.M., (1989) Momentum Transfer and Friction in the Debris of Rock Avalanches. Canadian Geotechnical Journal, 26, 623-628.*

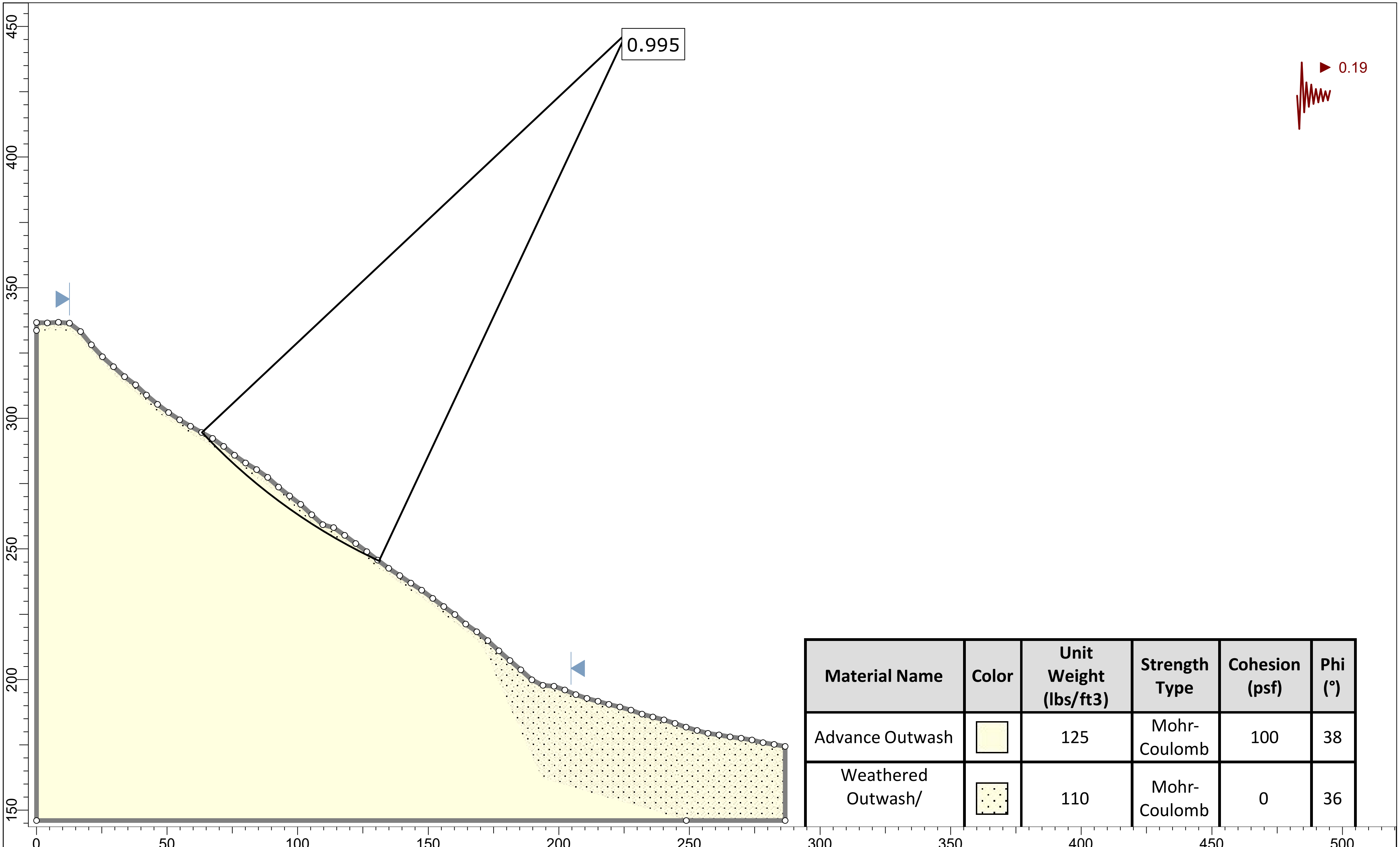
- *Washington State Department of Natural Resources Geologic Information Portal.*

- *National Cooperative Highway Research Program, Report 611, 2008. Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments.*

- *U.S. Department of Transportation Federal Highway Administration Publication FHWA-NHI-11-032, 2011. LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations Reference Manual.*

- *Final Technical Report – Probabilistic Shear Strength Criteria for Post-Liquefaction Evaluation of Cohesionless Soil Deposits – M. Guitierrez, M. Eddy, and P.M.H. Lumbantoruan, 2006.*

ATTACHMENTS: Three Figures
Plate A-1: Liquefaction Analysis



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Advance Outwash		125	Mohr-Coulomb	100	38
Weathered Outwash/		110	Mohr-Coulomb	0	36

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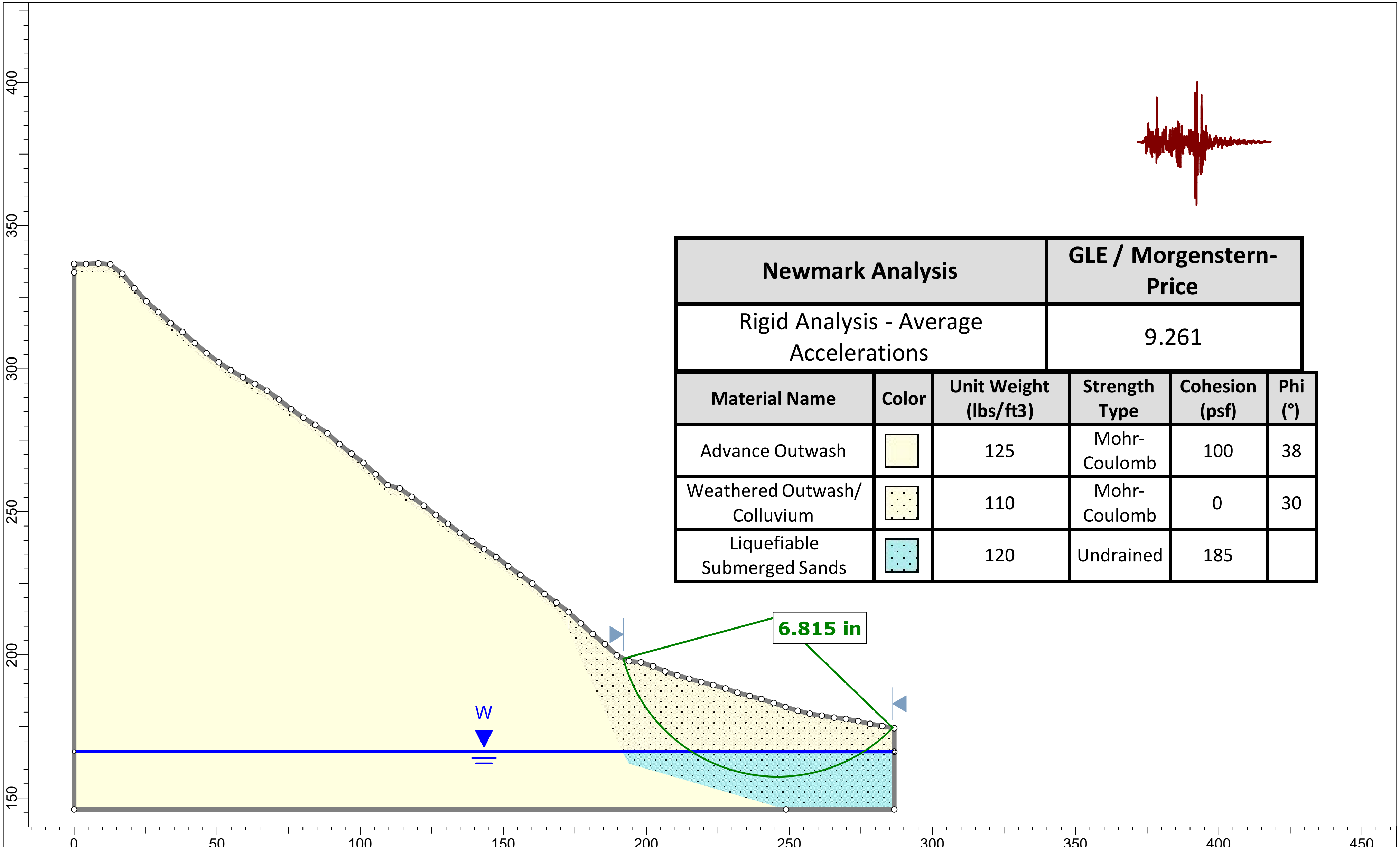
SLIDEINTERPRET 9.031

Project	Intrachat-Hoang Residence Development
Group	Slope Stability Analysis
Scenario	Seismic Analysis
Drawn By	
Figure 1	
Date	3/13/2024, 4:30:49 PM
File Name	Intrachat Res Revised Stability.slmd



Newmark Analysis	GLE / Morgenstern-Price
Rigid Analysis - Average Accelerations	9.261

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Advance Outwash		125	Mohr-Coulomb	100	38
Weathered Outwash/ Colluvium		110	Mohr-Coulomb	0	30
Liquefiable Submerged Sands		120	Undrained	185	

NGA
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SLIDEINTERPRET 9.031

Project	Intrachat-Hoang Residence Development		
Group	Slope Stability Analysis	*Scenario*	Newmark Displacement Analysis
Drawn By		*Figure 2*	
Date	3/13/2024, 4:30:49 PM	*File Name*	Intrachat Res Revised Stability.slmd

Reference: Publication No. FHWA-NHI-11-032, Section 6.2.2
Site: 7929 East Mercer Way, Mercer Island, WA
Pseudostatic Coefficient of Horizontal Acceleration Determination

H	100'	
PGA:	0.624	(USGS Mapped Acceleration Site Class B Conditions per NHI-11-032)
F_{PGA}	1.0	
F_v	1.5	
S1	0.504	
β	1.21	
α	0.605	
K_{av}	0.378	
K_s	0.19g	

Eqn 6-1 $k_{m_{ax}} = F_{PGA} \times PGA$

Site Adjusted PGA and Maximum Value of Seismic Coefficient

Eqn 6-2 $k_{av} = \alpha \times k_{\max}$

Average Peak Acceleration

Eqn 6-3 $\alpha = 1 + 0.01 \times H \times (0.5 \times \beta - 1)$

Where: α Slope Height Reduction Factor
β Acceleration Response Spectrum Shape Function

Eqn 6-4 $\beta = F_v \times S_1 / k_{m_{ax}}$

Where: F_v AASHTO site factor for 1 sec. spectral acceleration
S_1 1 Second Spectral Acceleration for Site Class B

Eqn 6-5 $k_s = 0.5 \times \alpha \times F_{PGA} \times PGA$

K_s 50% reduction of average seismic coefficient
*(assuming a C/D ratio of 1.1 and 1-2 inches of permanent displacement)

Figure 3

LIQUEFACTION ANALYSIS

Intrachat Hoang Residence

Hole No.=B-2 Water Depth=18 ft Surface Elev.=182

Magnitude=6.5
Acceleration=0.687g