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MEMORANDUM

DATE: January 11, 2024

TO: Maile Intrachat and Hoa Hoang

C/O: Tanya Nachia Somanna – Warm Modern Living

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

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

KHALED M. SHAWISH
STATE OF WASHINGTON
35215
REGISTERED
PROFESSIONAL ENGINEER
1.11.2024

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. For use in preparing this memorandum, we were provided with an updated full plan set titled "Intrachat Hoang Residence," dated November 27, 2023. Geotechnical peer review comments were presented on a previous version of the above-mentioned plans dated November 17, 2022.

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

Geotechnical Peer Review Comment Response

Comment 1: *The geotechnical engineer indicated in their report (page 5) that the onsite soils have a "moderate potential for liquefaction". Please determine the onsite soils' factor of safety against soil liquefaction. If liquefaction is anticipated under MCE earthquake ground motions, please provide an estimate of post-liquefaction settlement and analyze the potential for lateral soil movements (flow failure or lateral spreading). Please provide slope stability analyses results and supporting calculations for the estimated soil deformations in a report addendum.*

Response 1: The simplified liquefaction evaluation procedure outlined in FHWA-NHI-11-032 was utilized to determine the hazard potential based on subsurface data collected from the borings, review of nearby borings, and laboratory analyses. The procedure compares the cyclic resistance ratio to the cyclic stress ratio, induced by a design level earthquake. To aid in the analysis, we utilized the computer software Liquefy Pro by Civiltech. Inputs for the software include subsurface soil and groundwater properties, ground motion information, and drilling methods. The grain size analyses to support the design assumptions are attached to this memorandum as **Figures 1 through 4.** Along with the subsurface information collected by NGA, we also referenced a geotechnical boring performed by Cascade Geotechnical for the property located immediately southeast of the subject property (8011 East Mercer Way). Based on the above-referenced boring log, located at an approximate elevation correlating to the termination depth of B-2 performed on the subject site, the loose well-graded sand encountered beneath the site is expected to continue below the termination depth of B-2 an additional approximately 5-feet before transitioning to a cohesive silt/clay deposit. Therefore, the stratigraphy utilized in the liquefaction modeling includes a column of loose, well-graded sand extending 35-feet below the ground surface, where the groundwater surface was encountered at approximately 18-feet below the existing ground surface at the time of drilling. We did not consider the lower silt/clay unit in the analysis due to its likely high fines content and low liquefaction potential. 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. The results of the liquefaction analysis are attached on **Plate A-1.** The analysis indicates a total liquefaction-induced soil settlement of approximately 6-inches, where factors of safety for liquefaction primarily drop below one, below the observed groundwater elevation.

Our screening of the potential for flow failures or lateral spreading induced by seismicity indicates a relatively low potential to adversely impact the proposed residence. This is based on the gently sloping terrain adjacent to and below the residence, overall depth of the liquefiable soils, and the magnitude of expected liquefaction induced settlement. In our opinion, further slope stability and displacement analysis is not necessary for the site conditions and type of construction.

Comment 2: *In a report addendum, please provide static and seismic stability analyses of the slope uphill of the proposed residence. The geotechnical engineer indicated (page 7 of their report) that there is a significant potential for shallow sloughing or erosional events to occur during extreme weather conditions or seismic events. IBC also requires the use of MCE for the seismic slope stability analyses. If a pseudostatic analysis is conducted, provide an explanation (supporting documentation or references) of the seismic coefficient used in the analyses.*

Response 2: A limit equilibrium slope stability analysis was prepared along Cross-Section A-A', shown in our original report, utilizing the computer software Slope/W by GeoStudio. For seismic consideration, a psudostatic coefficient of horizontal acceleration was applied to the model. Selection of a suitable pseudostatic coefficient was determined based on the FHWA document 11-032 (LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations as well as the National Cooperative Highway Research Programs Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments, Report 611. These documents stipulate that the site-adjusted peak ground acceleration (PGA) is reduced due to the effects of wave scattering along slopes. The calculations for pseudostatic coefficient are shown in attached Figure 5 and yielded a psudostatic coefficient of 0.13g, utilized in the slope stability analysis. Our site measurements and existing topographic information indicate the site slope extending above the residence is approximately 130 to 145 feet in height; however, the slope height was limited to 100 feet for the calculation, per the above reference documents.

The stability analyses indicated generally stable conditions under static and a likely shallow slope failure under seismic conditions. The corresponding factors of safety for static and seismic conditions for the most critical slip surface are 1.2 and 0.9, respectively. The results of the stability analyses are shown in **Figures 6 and 7.**

Comment 3: *Coordinate with the structural engineer regarding the results of the post-liquefaction deformations so that the structural engineer can attest that the proposed structural design can tolerate the estimated deformations without building collapse as required by IBC.*

Response 3: NGA notified the project structural engineer of the liquefaction-induced deformation documented in response 1. The structural engineer indicated the entire structure is supported on driven pipe piles, including slabs-on-grade spanning between pile supported grade beams. The only exception is the trellis structure along the northeast corner of the house and in our opinion, damage to this structure as a result of potential liquefaction induced deformation is not considered a significant hazard to life or public safety.

Comment 4: *The geotechnical engineering explorations do not appear to locate the bearing layer for the proposed piles. Provide additional subsurface information to support proposed design and to be able to address the liquefaction concerns and potential post-liquefaction deformations discussed in a previous comment.*

Response 4: As discussed briefly in **Response 1**, we have reviewed boring logs performed by others in the near vicinity to the site. The nearest geotechnical boring log documented approximately 5-feet of undocumented fill underlain by 3-feet of loose, saturated sandy gravel, which we interpreted to correlate to the same colluvial sand layer encountered in B-2 performed at the subject site. Additionally, the surface elevation of the above-referenced boring is approximately at the termination elevation of B-2. Underlying the sandy gravel, the nearby boring encountered slightly disturbed to massive, silt; stiff to hard, which we interpreted as glacial lacustrine deposits, typical of this area. Other borings further downslope and southeast of the property also indicate silt deposits to the depths explored. Based on the review of nearby soil data, we anticipate the fine-grained strata discussed above lies within approximately 5- to 10-feet of the termination depth of lower elevation B-2 performed at the site. Furthermore, in our opinion, the consistency and relative soil density of the lower silt deposit should support the provided design foundation loads for the proposed structure and the additional overall thickness of liquefiable sandy soil that is interpreted to underly the B-2 boring has been accounted for in the liquefaction modeling as discussed in **Response 1.** That said, we expect the pin piles proposed for this site should advance 40- to 50-feet below the existing ground surface to reach non-liquefiable and cohesive soil, suitable for deep foundation support.

Comment 5: *The geotechnical engineer shall provide assumptions and supporting calculations for the 4-foot catchment capacity recommendation. Will the catchment wall be overtopped? and if so, do you anticipate slide debris reaching or impacting the proposed structure? Provide mitigation measures or recommendations as needed.*

Response 5:

General: The existing residence is overlain by a steep southeast-facing slope with an average gradient of approximately 35-40 degrees and overall height of approximately 140- to 145-feet. The slope is part of a regional break between the upper plateau of Mercer Island and the shoreline residential areas. Through field and office review, the slope appears to have been impacted by historical episodes of mass wasting, as indicated by the broad arcuate slope form. We anticipate these features were formed during the advance and retreat of continental ice sheets during the last glaciation and represent erosional features versus a dormant/indistinct deep-seated landslide complex. The observed steep slope gradient, coupled with review of nearby boring data above the slope, indicate high internal strength of the slope soils, which we anticipate consist of glacially consolidated outwash sands and possible glacial till mantling the top of slope. Therefore, we anticipate the most likely form of slide to occur above the site will consist of shallow translational skin sliding affecting the upper topsoil and weathered soil horizons. The slope geometry and soil properties of the slope were input into limit equilibrium slope stability software to determine factors of safety for sliding, as well as aid in determination of the debris runout potential and overall magnitude.

Volume Estimates: Based on the modeling discussed in **Response 2**, a shallow slide with factor of safety just below one may be expected under seismic conditions or in the event of extreme precipitation. We selected the most critical failure surface with a factor of safety of 0.9, obtained directly from Slope/W slope stability analysis. The slide mass with a factor of safety of 0.9 equated to a volume of approximately 173 cubic feet (6.4 cubic yards). Based on observation of many similar landslides in the area, we assume that only a fraction of the total volume of displaced material during such an event will be mobilized downslope and impact the debris wall. For the debris impact analysis, we estimate approximately 10 cubic feet per linear foot of slide debris will be mobilized downslope. The resting slope surface of the deposited slide debris against the lower proposed 4.0-foot catchment is estimated to be 20 degrees. With this assumed configuration, the 4-foot catchment wall can contain approximately 35-cubic feet (1.3 cubic yards) per lineal foot of wall.

Runout and Impact Force: We assumed that the velocity of the debris flow will decrease along its runout distance, due to internal friction within the debris mass and interaction with the slope surface. The landslide initial velocity was estimated utilizing the Heim Energy Line Method (Heim et al., 1932) which allows computation of a 'kinetic load' using the slopes configuration and geometric relationships. The kinetic load (k) is the vertical distance measured below a hypothetical line tangent of the slope propagation angle and the slopes surface. Analyzing the slope's geometry, we yielded an average velocity within the assumed initiation zone of 19 ft/second. From the slope stability model, we determined the runout distance from the toe of the slide to the catchment wall to be approximately 95-feet. The debris velocity just before impact was estimated using an equation from the work of Van Gassen and Cruden, 1989, where the velocity at x distance from the initiation of the slide can be determined by an empirical relationship between the initial slide velocity and the total runout distance. We determined the debris velocity just prior to impact to be approximately 13.5 feet per second. Assuming an average slide debris unit weight of 85 pcf, the debris weight was determined to be 850 pounds and an impact momentum of 11,475 foot-lbs per second. We estimate that the above-mentioned catchment volume will take approximately 35 seconds to accumulate to full condition, equating to an impact force of approximately 328 pounds. Per our initial report, the debris catchment shoring wall has been designed to resist an equivalent fluid density of 100 pounds per cubic foot, which equates to a resultant force of 800 pounds, thus the catchment wall should be structurally capable of stopping a shallow skin slide of this magnitude that may occur along the site slope. As mentioned above, as much as 173 cubic feet of material may be disturbed during an initial skin slide event and may propagate downslope in a series of slide events. After the initial event and impact discussed, debris may accumulate to a point where the wall is overtopped; however, we expect the overall momentum of the debris will be significantly reduced and will accumulate within the space between the shoring wall and the house, which is approximately 10-feet and would not cause significant damages to the residence. To maintain continued functionality of the debris catchment portion of the wall during its lifespan, we recommend periodic maintenance is performed to maintain the 4-foot exposed height above the slope surface.

Comment 6: *The excavation required for the detention tank in this area will reduce the passive resistance from the current design value. Geotechnical engineer to provide a recommended reduced passive resistance to account for the detention tank excavation located in front of a portion of the shoring wall. Provide recommendations in a report addendum. Revise shoring wall design accordingly using the reduced passive pressure in the area of the detention tank excavation.*

Response 6: The location of the detention tank and shoring wall design were revised and based on the new layout, we do not anticipate reductions in passive resistance for the shoring system.

Comment 7: *The geotechnical engineer recommended a maximum 2H:1V temporary cut slope. The geotechnical engineer should review this proposed open cut. Provide stability analyses of this temporary cut to verify that a cut of this magnitude and slope angle (currently max of 1H:1V) is stable or provide alternate temporary excavation/shoring configuration. Provide analyses in a report addendum.*

Response 7: The temporary excavation stability will be dependent on the soil conditions encountered during construction. The updated plans indicate an open excavation with overall height of approximately 7 feet within the location of the detention tank. Based on the location of the proposed detention tank it appears there is space to adjust and lay the temporary slopes back additionally, if needed. Assuming semi-competent soil conditions, free of groundwater seepage, up to a four-foot vertical cut overlain by a 2H:1V temporary slope to existing grade could safely be accommodated. The soil conditions and cuts should be evaluated and monitored by NGA at the time of excavation. We anticipate worker access will be necessary for the inlet tie-in and setting the outlet catch basin. For the catch basin (CB-5), a suitable shoring box could be utilized to limit excavation and maintain worker safety. At the inlet tie in, there is open space to the northwest of this work area and shallower temporary cut gradients could be accommodated.

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.

o-o-o

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).*

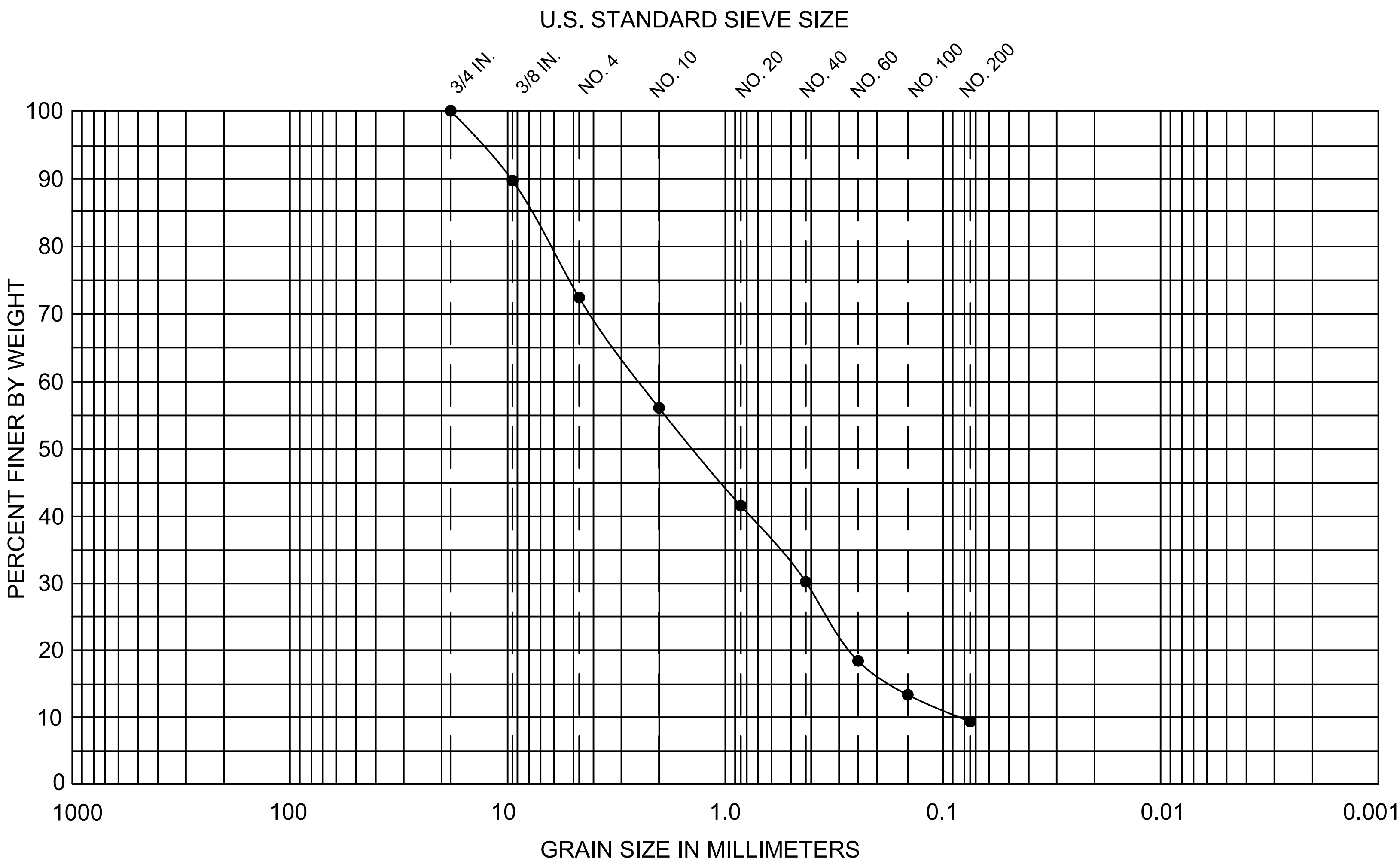
- *Van Gassen, W. and Cruden, D.M., (1989) Momentum Transfer and Friction in the Debris of Rock Avalanches. Canadian Geotechnical Journal, 26, 623-628.*

- *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.*

ATTACHMENTS: Seven Figures
Plate A-1: Liquefaction Analysis



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●SW-SM	B-2	5.0 feet	Brown, well-graded sand with gravel and silt	Gravel = 28% Sand = 63% Silt/Clay = 9%

No.	Date	Revision	By	CK
1	3/31/23	Original	FKS	ABR

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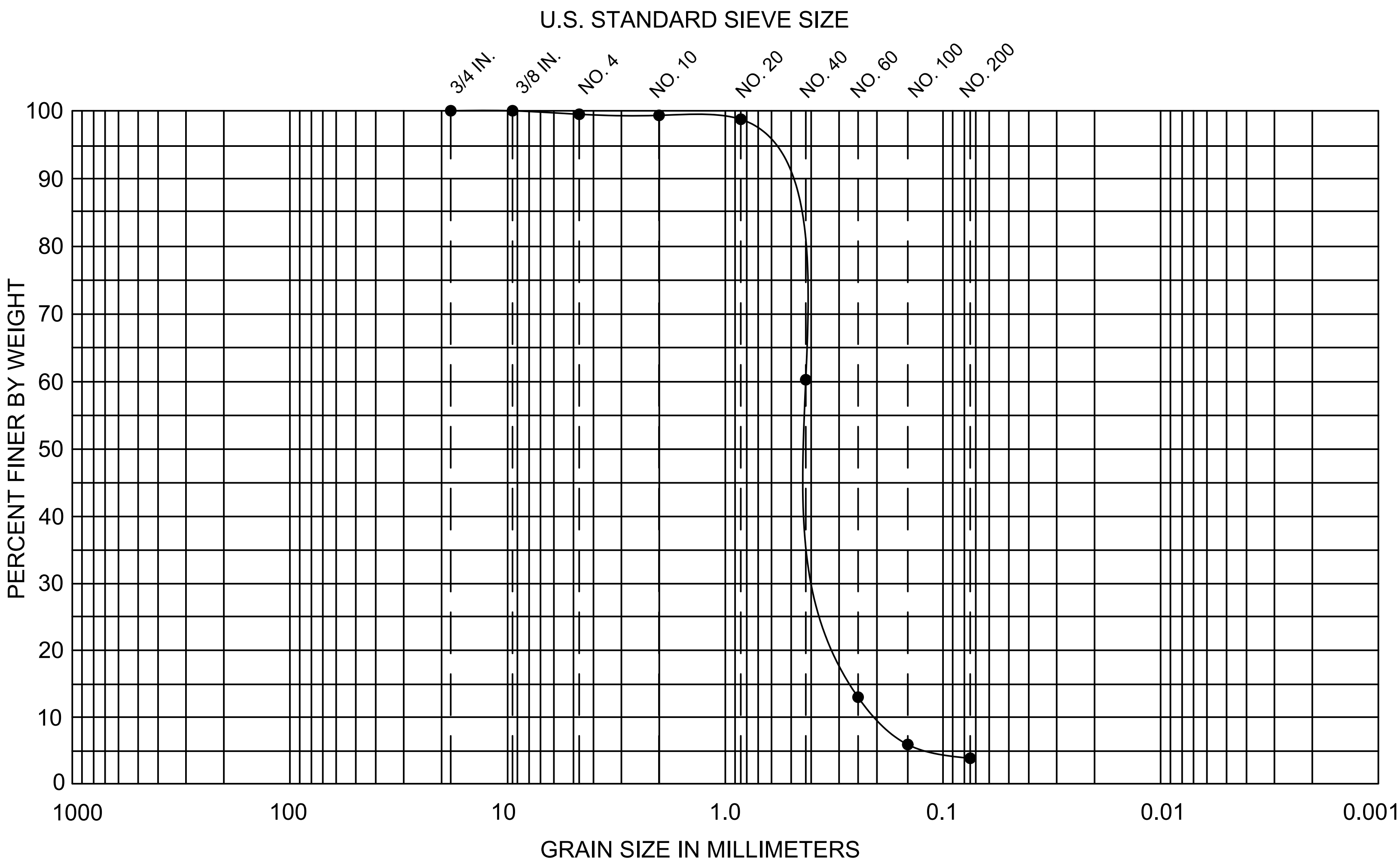
Intrachat-Hoang Residence
Development
Sieve Analysis

Project Number
1276521

Figure 1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●SW-SM	B-2	5.0 feet	Brown, poorly graded sand	Gravel = 0% Sand = 96% Silt/Clay = 4%

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Figure 2

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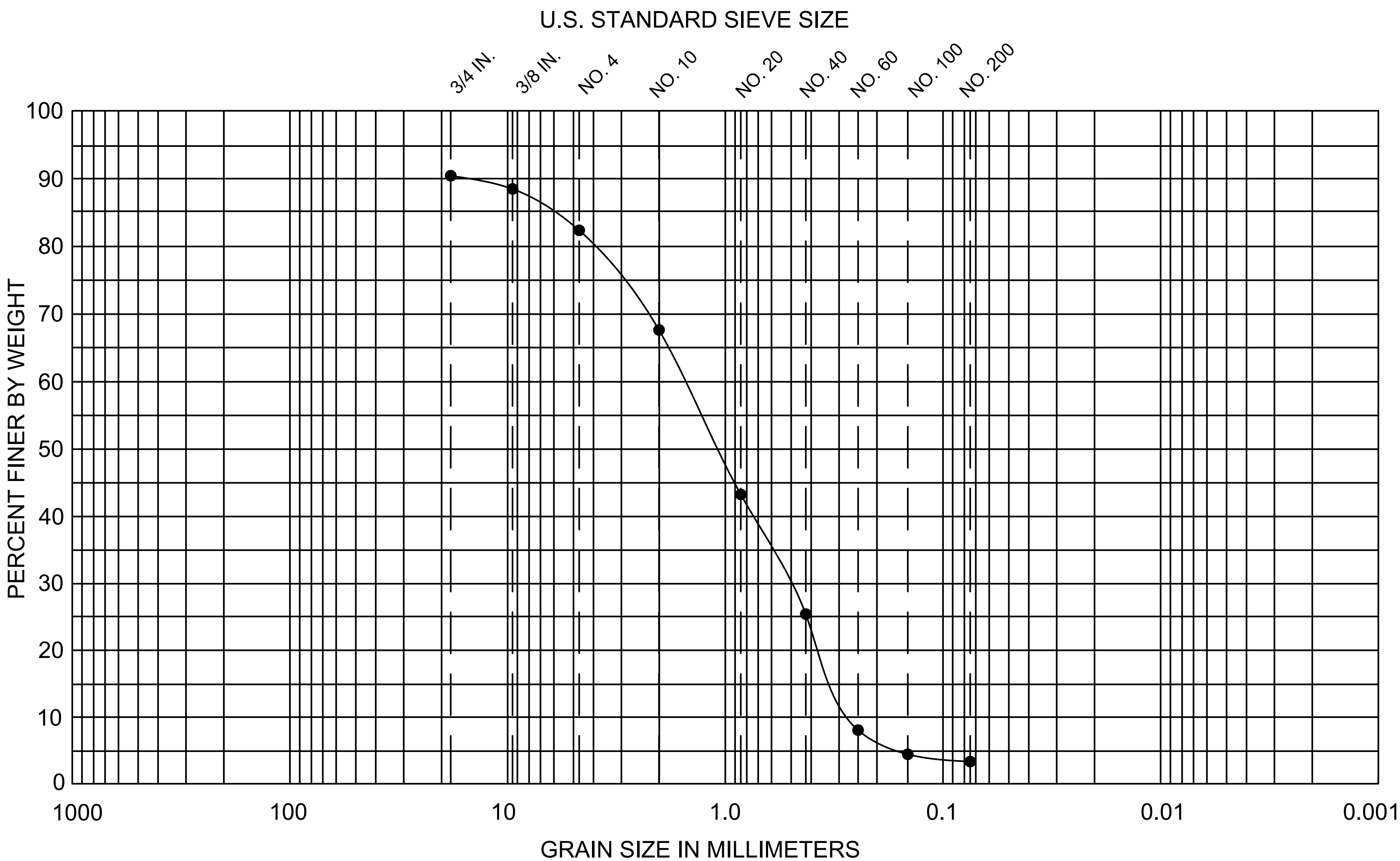

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
● SW	B-2	20.0 feet	Brown, well-graded sand with gravel	Gravel = 18% Sand = 79% Silt/Clay = 3%

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Intrachat-Hoang Residence Development
Sieve Analysis

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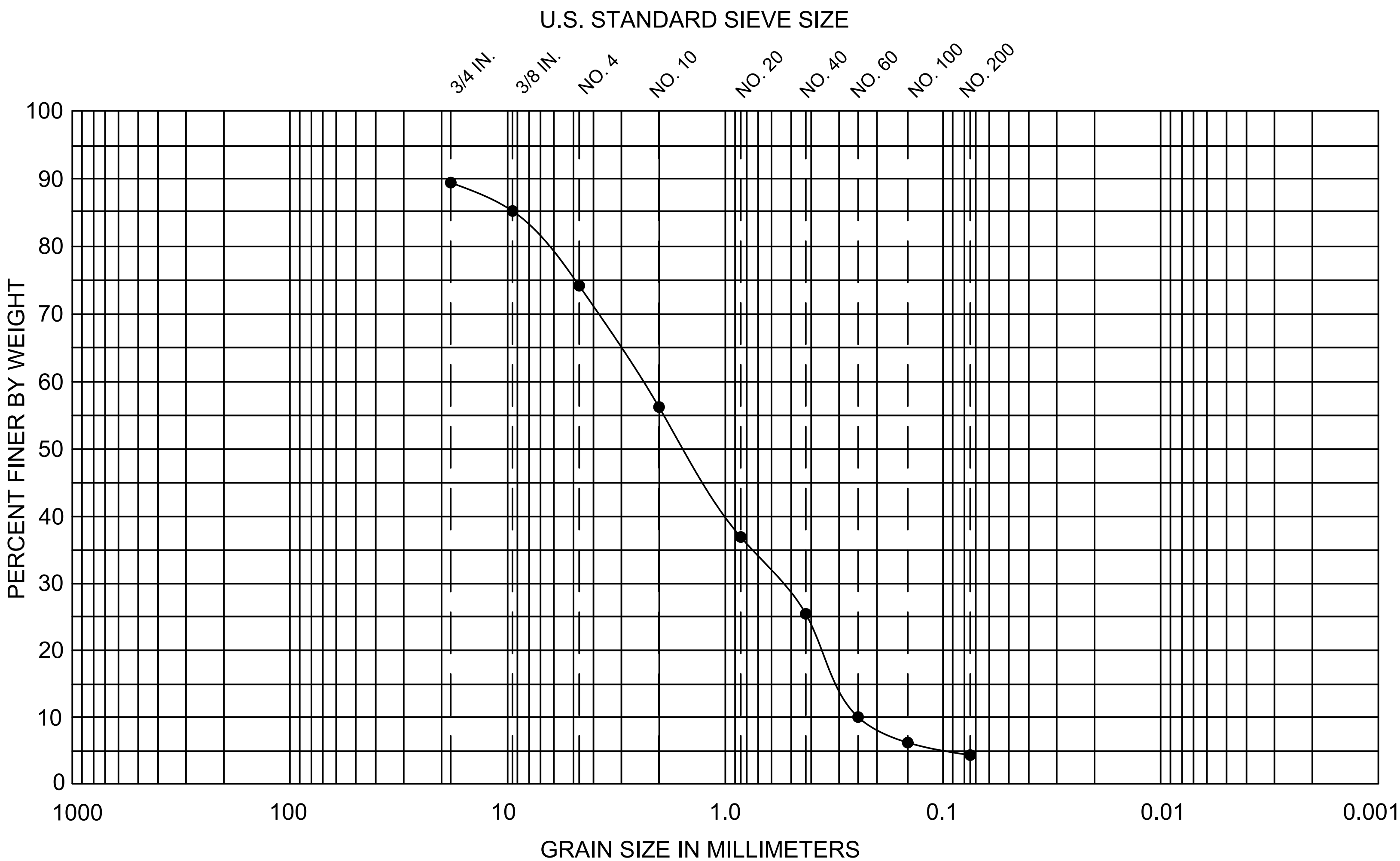
Figure 3



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

U.S.C. SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
●SW	B-2	30.0 feet	Brown, well-graded sand with gravel	Gravel = 26% Sand = 70% Silt/Clay = 4%

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Figure 4

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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
F_{PGA}	1.1
F_v	1.0
S1	0.504
β	0.734
α	0.367
K_{av}	0.252
K_s	0.13g

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

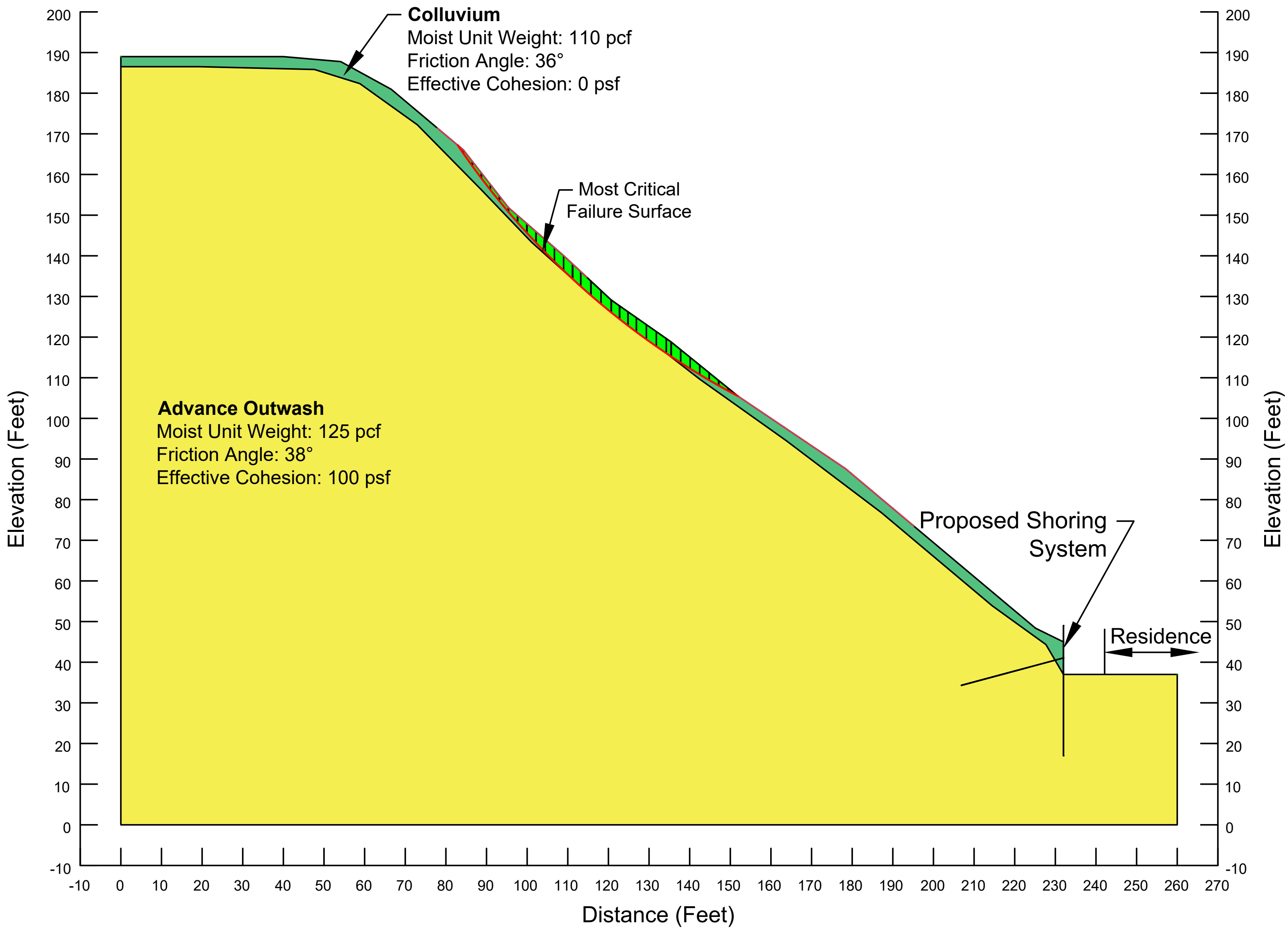
Eqn 6-5 $k_s = 0.5 \times \alpha \times F_{PGA} \times PGA$

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

Figure 5

Slope Stability Analysis - Cross Section A-A' Static Conditions

Morgenstern-Price most critical surface with minimum FOS = 1.2



NOTES: Not to scale
Elevations are Approximate

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Intrachat-Hoang
Residence Development
Cross-Section A-A' Static
Slope Stability Analysis

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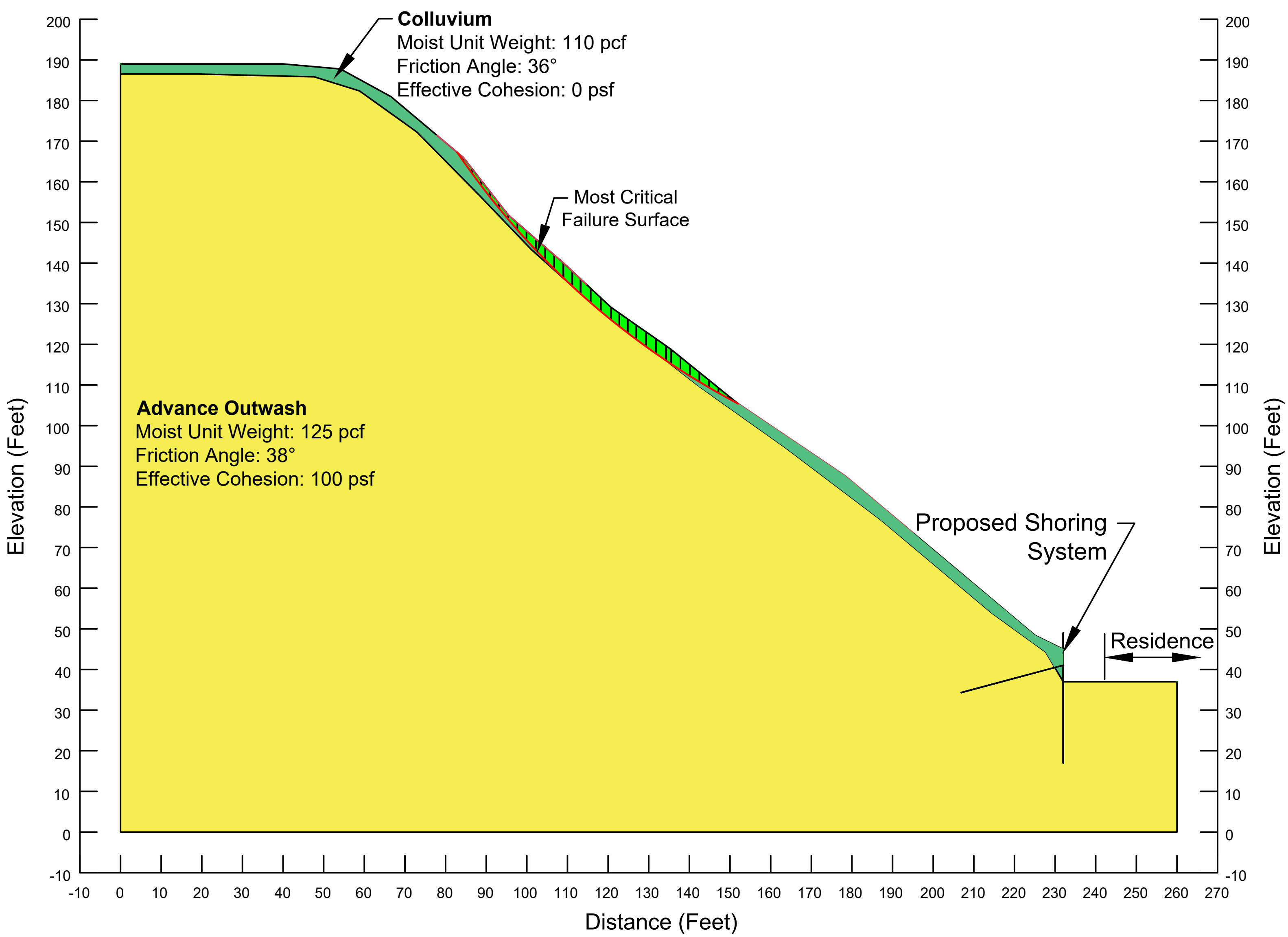
Figure 6

Slope Stability Analysis - Cross Section A-A' Seismic Conditions

Morgenstern-Price most critical surface with minimum FOS = 0.9
with a seismic coefficient of ground acceleration = 0.13g



NOTES: Not to scale
Elevations are Approximate

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Intrachat-Hoang
Residence Development
Cross-Section A-A' Seismic
Slope Stability Analysis

Project Number
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Figure 7

LIQUEFACTION ANALYSIS

Intrachat Hoang Residence

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

Magnitude=6.5
Acceleration=0.3g