NELSON GEOTECHNICAL
ASSOCIATES, INC.

17311-135th Ave. N.E. Suite A-500
Woodinville, WA 98072
(425) 486-1669
www.nelsongeotech.com

May 26, 2023

Catherine and Edward Moran
Via Email: catherine.b.moran@gmail.com
edmoran82@gmail.com

c/o William Gottlieb
Plan One Fine Home Design
Via Email: wmgottlieb@planone.biz

Geotechnical Comment Response
Moran Residence Development
5028 West Mercer Way
Mercer Island, Washington
NGA File No. 1211520

Dear Catherine and Edward:

This letter documents our response to geotechnical city review comments regarding your planned residence at **5028 West Mercer Way on Mercer Island, Washington.**

INTRODUCTION

We prepared a revised geotechnical report titled ***"Geotechnical Engineering Evaluation (REV2) – Moran Residence Development – 5000 West Mercer Way – Mercer Island," dated September 27, 2021.*** We also prepared a plan review memorandum dated August 6, 2021, a critical areas study dated December 17, 2021, and a comment response letter dated April 14, 2023.

For our use in preparing this letter, we were provided with the following documents:

- ***Sheets 1 through 16 prepared by Plan One Fine Home Design dated December 8, 2022.***
- ***Sheets S-1 through S-8 and SH-1 through SH-3 prepared by Kia Co dated March 31, 2023.***
- ***Sheets SH-1 through SH-3 prepared by Kia Co dated December 8, 2022.***
- ***Sheets C-01 through C-06 prepared by JMJ Team dated December 16, 2022.***

MERCER ISLAND REVIEW COMMENTS

We have been provided with geotechnical review comments from the City of Mercer Island for the proposed residence development in a plan set markup dated April 27, 2023. In the following section we summarize the review comment followed by our response.

COMMENT:

This comment is a follow-up to our previous response regarding the debris catchment measures recommended for this site. In summary, the comment requests additional information and justification regarding the debris runout analysis discussed in our previous response memorandum.

RESPONSE:

General: The debris analysis was reworked to justify the debris catchment capacity and configuration recommended in our original geotechnical report. As noted in our previous documentation, significant landslide activity on the site slopes is not expected due to slopes' relatively shallow gradients, full vegetation, and competent native glacial material forming the core of the slopes. Static slope stability modeling does not predict a critical failure and supports our assumptions of stable conditions. The following debris runout analyses is based on a seismic scenario, where a critical failure with factor of safety for sliding is less than 1.0.

Slope Stability Analysis: We characterized the subject slopes landslide potential utilizing the computer software Slope/W, a limit-equilibrium slope stability analysis program. Cross-Section A-A' was generated utilizing the site's topographic survey along with information from King County IMap for offsite portions of the slope. Previous soil explorations along the eastern slope noted approximately 6-inches of topsoil underlain by weathered native glacial soils ranging from loose to medium dense. Based on evaluation and observation in the field the depth of loose material was typically 1.5- to 2.0-feet. Localized undocumented fill with variable density ranging from loose to medium dense was encountered up to 5-feet below the ground surface. The majority of the slope is well vegetated and does not appear to have historically been modified by clearing or grading, so we anticipate the 5-feet of fill is most likely localized and potentially associated with the residential property immediately above and to the northeast of the subject site. Nonetheless, we conservatively modeled the slope with a mantle of 3-feet of less competent material overlying the dense or better glacial material interpreted to form the core of the site slope. The material strength properties, parameters, and results of our slope stability analyses are shown in the Slope Stability Model on Figure 1. We evaluated the slope for most critical failure surface under static and seismic conditions, respectively. Per the ASCE 7 hazard assessment tool, the site's peak ground acceleration (PGA) is 0.619 with the geometric mean of the PGA (PGA_m) being defined as 0.681.

For the seismic scenario analysis, a pseudo-static horizontal acceleration parameter was applied to the slope, which was selected as 0.34g, using a standard ½ reduction to the PGA_m site-specific value. Under seismic load, the modeling indicates the most critical failure surface consists of a shallow skin slide failure with factor of safety of approximately 0.7. The behavior of the landslide modeled is consistent with our interpretation and assumed shallow translational type slide that may be expected on this site. Strong seismicity and/or long-duration rainfall would be scenarios where such a slide is more probable and could be expected to impact the upper loose and more permeable soils. Utilizing the same soil and seismic parameters, we also filtered through the various failure surfaces and selected a less likely (higher factor of safety) slide event just below a factor of safety of 1.0 to verify storage volume and proposed catchment capacity.

Volume Estimates: We selected a 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 47 cubic feet, which is the value considered for the storage capacity analysis. Note that the most critical failure surface with a factor of safety of 0.7 contains a volume of approximately 26 cubic feet. The resting slope surface of the deposited slide debris against the upper 1.5-foot catchment and 4-foot-tall structural house foundation wall is 15 degrees, respectively. With this assumed configuration, the upper 18-inch catchment and lower concrete house wall can contain approximately 30- and 29-cubic feet, respectively, equating to a combined volume of 59 cubic feet. This is based on the most critical section, where the soldier pile wall and reinforced house foundation wall are 6-feet apart. The combined storage volume of approximately 59 cubic feet, obtained from the slope stability analysis exceeds the anticipated slide volume of 47 cubic feet, therefore potential slide debris can be accommodated by the designed debris catchment systems. Furthermore, the analysis does not consider the effects of debris spreading laterally upon impact or the greater 12-foot separation along some portions of the house, and as such, it is our opinion that the above analysis presents a conservative estimate.

Mobility and Deposition: Mobility is generally a function of slope geometry, material composition, and surface roughness. Per the work of Benda et al. (1990) and Guthrie et al. (2010), slide initiation typically occurs on slopes in the range of 60- to 70-percent while deposition occurs on shallower gradients in the range of 14- to 27-percent. For the subject site, we have considered the upper steeper portion of the slope beginning at the crest and extending downslope for about 35-feet at an average grade of 47 percent to be the most likely region for slide initiation, while the lower 30 feet along the slope surface contains shallower gradients averaging about 25 percent and would likely result in a depositional zone. With this in mind, the modeled slide mass will decelerate at the start of the depositional zone, such that once it reaches the upper debris catchment system, the final debris velocity will be significantly lower than the initial velocity. 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 10.5 ft/second.

Rate of deposition was estimated with the empirical methods of Van Gassen and Cruden (1989). The formulas allow calculation of estimated debris runout distance and velocity at specific distances from initial point of deposition as a function of initial slide velocity, runout slope angle, apparent slope angle, and friction angle of the debris. The literature suggests utilizing an apparent slope angle of friction of debris of 25 degrees for open hillside failures and estimated debris volumes less than 400 cubic meters (D.O.K. Lo, 2000). The distance between the initial zone of deposition and the face of the upper debris catchment wall was measured to be approximately 31 feet. Using the above noted initial slide velocity, and average slope gradient in the depositional zone of 17 degrees and an apparent angle of friction of debris of 25 degrees, we calculated an instantaneous velocity of 3.12 ft/second at 31 feet, which is the initial impact velocity.

Runout and Impact Force: We assumed that the debris flow will have an average thickness of 0.5 feet and its velocity will drop significantly as the debris builds up against the catchment wall. This is due to internal friction within the debris mass. Assuming that the average deposition velocity of the debris flow is one half the initial impact velocity (1.55 ft/sec) it will take about 10 seconds for the debris to fill up the gap behind the debris wall. With the above noted storage volume behind the upper catchment wall of 7 cubic feet and a loose unit weight of debris of 95 pounds per cubic feet, a total impact mass of 665 pounds was considered with an average impact velocity of 1.55 ft/s and momentum of 1030 pound-ft per second. The impact force on the wall will develop over the above-mentioned 10 second average deposition time and thus resulting in an approximate force of 103 pounds. Given the 1.5-foot-tall catchment height, this equates to an equivalent fluid density force acting on the wall of approximately 90 pounds per cubic foot. The lower debris wall will experience a much smaller impact but nonetheless, it was also designed to resist 90 pcf debris load.

COMMENT:

Per the City of Mercer Island Arborist, the geotechnical engineer of record should review and approve landscape plans and verify they conform to the geotechnical recommendations for this site.

RESPONSE:

We reviewed the landscape mitigation plans sheets L-1 through L-5 titled “Moran Residence,” dated February 23, 2023 and prepared by Tree Solutions, Inc. Plans indicate 8 trees to be removed, primarily along the proposed residence location and near vicinity, and replacement of 36 new trees to the east, south, and west of the proposed residence. Approximately 22 new trees, mostly mountain hemlock and douglas fir, are proposed above the planned shoring wall and along the moderate to steep west-facing slope. The plans indicate log rounds should be deliberately placed against the slope to serve as nurse logs. Additionally, temporary irrigation is recommended for new trees. In general, the proposed planting does not present significant geotechnical concerns, however, we do not recommend placing large log debris on steeply sloping areas but could be placed elsewhere on the site in areas of level to gently sloping terrain. Irrigation systems should generally be shutoff during the wet season between October 1 to April 1, as to not contribute to additional saturation of the soils along steeply sloping portions of the site. Upon reactivation of irrigation, the system should be checked and maintained as needed.

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

- ***Benda, L. and T.W. Cundy (1990). Predicting deposition of debris flows in mountain channels. Canadian Geotechnical Journal. V. 27, 409-417.***

- ***Guthrie, R.H., A. Hockin, L. Colquhoun, T. Nagy, S.G. Evans, and C. Ayles (2010). An examination of controls of debris flow mobility: evidence from coastal British Columbia. Geomorphology 114, 601–613.***

- ***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).***

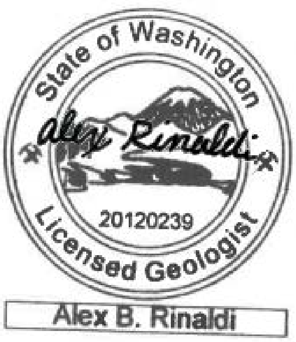
- ***Van Gassen, W. and Cruden, D.M., (1989) Momentum Transfer and Friction in the Debris of Rock Avalanches. Canadian Geotechnical Journal, 26, 623-628.***

- ***D.O.K. Lo, (2000) Review of Natural Landslide Debirs-resisting Barrier Design, GEO Report No. 104, 91p.***

We appreciate the opportunity to provide service to you on this project. If you have any questions or require further information, please call.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.



Alex Rinaldi, LG, EIT
Project Geologist

5.26.2023

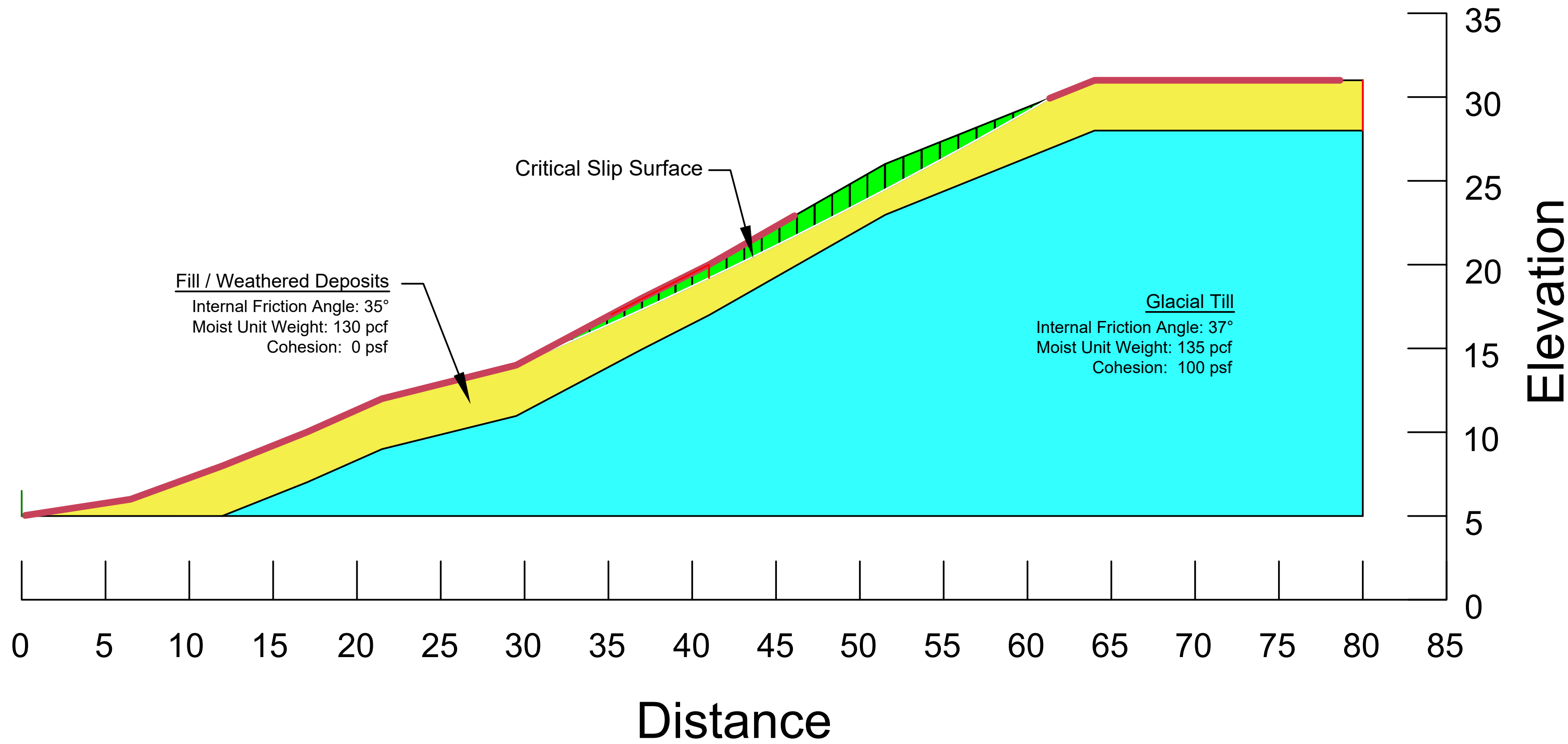
Khaled M. Shawish, PE
Principal

ABR:KMS:dy

Two Figures Attached

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

Morgenstern-Price most critical surface with minimum FOS = 0.7
with a seismic coefficient of ground acceleration = 0.34g



NOTES: Not to scale
Elevations are Approximate

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Figure 1

Moran Residence Development
Cross Section A-A' Existing Seismic
Slope Stability Analysis

NELSON GEOTECHNICAL ASSOCIATES, INC
www.nelsongeotech.com

Woodinville Office
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax: 481-2510

Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
(509) 665-7696 / Fax: 665-7692

No.	Date	Revision	By	CK
1	5/17/23	Original	DPN	ABR

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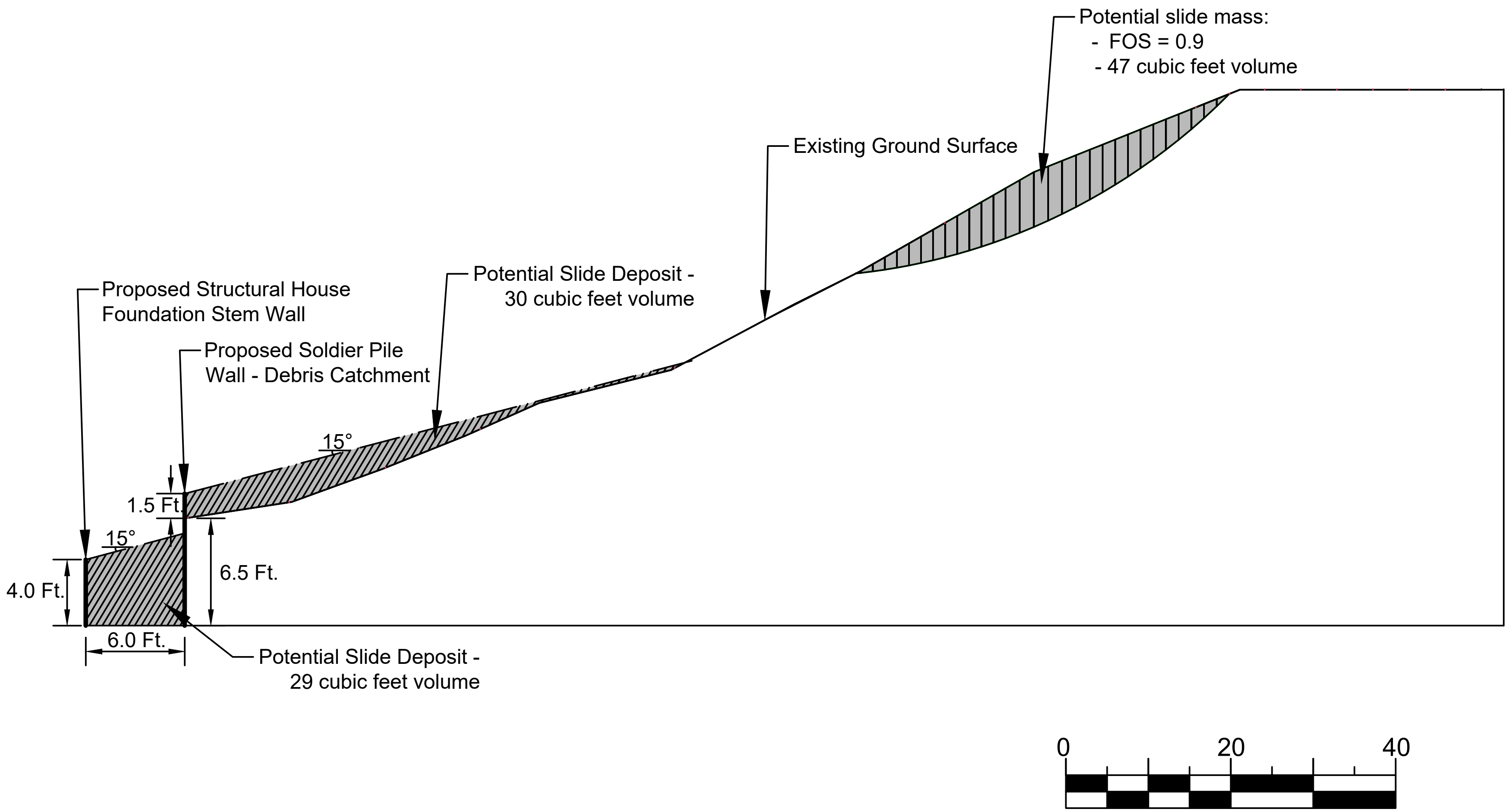
Schematic Slope Stability Section - Debris Volume Estimate



Approximate Scale: 1 inch = 20 feet

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Figure 2

Moran Residence Development
Schematic Slope Stability Section
Debris Volume Estimate

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Woodinville Office
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax: 481-2510

Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
(509) 665-7696 / Fax: 665-7692

www.nelsongeotech.com

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