GEO Group Northwest, Inc.
Geotechnical Engineers, Geologists
& Environmental Scientists

February 4, 2016

G-3837

Mr. William C. Summers
MI Treehouse, LLC
P.O. Box 261
Medina, Washington 98039

Subject: Response to November 18, 2015, Geotechnical Third Party Review Letter, Proposed Residence, 5637 East Mercer Way, Mercer Island, Washington.

Reference: Geotechnical Third Party Review, 5637 E. Mercer Way, Mercer Island, Washington (Perrone Consulting Project #15124). Perrone Consulting, Inc., P.S., November 18, 2015.

Dear Mr. Summers:

Per your request, GEO Group Northwest, Inc. has prepared this letter which presents our responses to comments in the above-referenced geotechnical third party review letter dated November 18, 2015, by Perrone Consulting, Inc. (Perrone), regarding the proposed residence to be constructed at 5637 East Mercer Way in Mercer Island, Washington.

Comment #1: Requested Subsurface Information

We understand that the requested subsurface information was received by Perrone.

Comment #2: Subsurface Profile

The loose sandy soils comprising Unit 1 in Table 1 consist of both in-situ advance outwash soils and landslide or mass wasting deposits in our opinion. The soils that were encountered in Boring B-3 and assigned to this unit largely consist of in-situ advance outwash deposits that have been exposed to typical weathering and biogenic disturbance processes. The soils that were encountered in Borings B-1 and B-2 and assigned to this unit largely consist of landslide or mass

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wasting deposits. The engineering properties of the soils comprising Unit 1 are appropriate, as agreed by Perrone, even though the origins of the soils vary. Therefore, the usefulness of the stability analysis results is not adversely affected by the soils classification in Table 1.

Comment #3: Additional Slope Stability Analysis

Additional stability analysis was performed for the site profile A-A' following receipt of the November 18, 2015, review comments. For the additional analyses, the SPT data obtained for the borings were adjusted for soil overburden effects, using the methodology described by Liao and Whitman (1986)[1]. Analysis for the seismic case was modified to consider a lateral peak ground acceleration of 0.6g, instead of the 0.3g used in our previous analysis. A pseudo-static seismic coefficient, k_h, of 0.15 was used for the analysis, following recommendations from research by Melo and Sharma (2004)[2], instead of the typical assumed value of 0.5. The analyses were performed using the Slide 7.0 computer software program using the same material properties and the Bishop method of vertical slices as previously used in the XSTABL analysis.

The additional stability analysis calculations indicate that the slope profile has an FS value of 1.22 for stability in its existing configuration for the static case, and an FS value of 1.01 for the modeled seismic case. The small change in the static-case FS value is attributed to minor adjustments in the soil layer thicknesses resulting from using the adjusted SPT data. The increase in the seismic-case FS value is due to the use of the updated peak ground acceleration and seismic coefficient values used for the analysis.

For both the static and seismic cases, the most critical failure surfaces for the existing slope condition consist of arc-shaped failures that involve the loose sand soils. These failures surfaces generally are limited to the unsaturated soils upslope from the proposed residence location in the static case. These most critical failure surfaces are illustrated in the analysis plots provided in Attachment 1.

Based upon the results from the subsurface investigation and slope stability analysis that we have completed, it is our opinion that the existing steep slope in proximity to the proposed residence location is relatively stable under static conditions, with the exception of some possible surficial raveling if exposed or disturbed.

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1 Liao, S.S.C., and R.V. Whitman, Overburden Correction Factors for SPT in Sand. Journ. of Geotechnical Eng., 10.1061/(ASCE)0733-9410(1986)112:3(373), 373-377.
2 Melo, C., and S. Sharma, Seismic Coefficients for Pseudostatic Slope Analysis. 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, August 1-6, 2004, Paper No. 369.

Under the seismic conditions used in the analysis, the loose soils on the slope and in the residence location appear to be marginally stable with respect to movement of the loose soils. We anticipate, from experience and knowledge of typical slope movement events in the local region, that the nature of soil movement on the steep slope would consist primarily of sloughing rather than block sliding.

In our opinion, supporting the residence on a pile foundation system can adequately mitigate potential damage due to potential soil movement or settlement below the residence. Installing a catchment wall along or near the base of the steep slope can adequately mitigate possible damage due to potential soil movement on the steep slope.

Temporary Excavation during Construction

We recommend that passive shallow dewatering be performed at the start of construction by using shallow trenches/swales at southeast, south, and west perimeter of disturbance area. The trenches should be sloped to drain toward stream. Staged deepening of excavation for the working pad for building can then be performed as the lowering of the water table proceeds. We recommend that the temporary excavation slopes be limited to 1H:1V in inclination and that they be backfilled to approximate original grade after the building foundations and walls have been constructed and restrained. The results of slope stability analysis of this temporary configuration indicate a static condition factor of safety of 1.15 provided that dewatering of the excavation area is achieved. The stability analysis results are provided in Attachment 1.

Comment #4: Catchment Wall

Protection of the residence from slope failure as identified from the slope stability analysis results can be provided by constructing an engineered catchment/retaining wall at or near the base of the steep slope. The wall alignment should run south of the residence and continue around the southwest corner a distance of another approximately 20 feet. We recommend that the wall have a minimum height of 8 feet above final grade as measured on its upslope side.

The wall should be supported on a pile foundation system to provide vertical support. Laterally staggered piles, battered piles, or helical anchors can be used to provide lateral support. We recommend the wall be designed to resist an active lateral earth pressure of 60 pounds per cubic foot (pcf) to its full height to accommodate the capture of potential soil debris from adjacent slope.

Drainage of potential water accumulation behind the wall should be managed by installing a 4"-diameter rigid perforated HDPE or Schedule 80 PVC drain pipe along the back of the wall, surrounding the pipe with at least 6" of clean crushed or drain rock, and surrounding the rock with a layer of durable non-woven geotextile filter fabric. The drain line should be sloped to direct flow to an appropriate discharge point or tightline.

Comment #5: Down-drag Effects on Pipe Piles

In our opinion, the potential for liquefaction or settlement of the loose sandy soils to exert significant down-drag forces on the steel pipe piles proposed for the project is relatively low. We have analyzed the potential down-drag force associated with settlement of these soils due to friction against the piles. For the case of an 18-foot thickness of saturated, loose, sandy soils (the maximum thickness scenario near boring B-2), full mobilization of potential down-drag forces is calculated to result in a down-drag load of approximately 2.0 tons per 4"-diameter pile and approximately 3.0 tons for per 6"-diameter pile.

Based on above analysis, it is our opinion that 4"-diameter Schedule 40 steel pipe piles can be considered to have an allowable net axial bearing capacity of 8 tons each (10 tons minus the 2.0 tons for potential down-drag) when successfully driven to the applicable refusal criteria described in our geotechnical report dated March 13, 2015. Net capacity for 6"-diameter Schedule 40 steel piles can be assigned as 12 tons each (15 tons minus the 3.0 tons for potential down-drag).

Supplemental Recommendations: Design and Construction Recommendations Regarding Augered Concrete Piles

As an alternative to small-diameter pipe piles, the proposed residence can be supported on a system of auger-cast concrete piles that are founded in the medium dense to dense native soils encountered below a depth of approximately 15 to 20 feet below the existing ground surface. At a minimum, we recommend the piles have an embedment length of at least 10 feet into these soils.

Recommended allowable axial capacities for concrete piles having diameters of 16, 18, and 24 inches are presented below in Table 1. A safety factor of 3.0 is included in the tabulated capacities. The capacities were calculated based on the soil conditions encountered in the borings B-1 and B-2 which were completed in or near the proposed residence location. The calculated capacities are based on skin friction and end bearing resistance in the medium dense to dense soils that were found below a depth of approximately 20 feet. Estimated negative skin friction resistance values (also referred to as "down-drag") associated with potential settlement of the upper approximately 15 to 20 feet of saturated loose sandy soils are provided below in the table, as are net allowable pile capacities which include the estimated potential down-drag.

The calculated pile capacities apply to individual piles that have a spacing of at least three times the pile diameter. Single pile capacities for grouped piles are less than for independent piles, and should be evaluated based on the proposed grouping configuration, where appropriate. A one-third increase in the above allowable pile capacities can be used when considering short-term transitory wind or seismic loads.

TABLE 1: Allowable Axial Pile Capacities

Pile Diameter (inches)	Embedment Length (feet)	Allowable Capacity (tons)	Pile Down-drag (tons)	Net Allowable Capacity* (tons)	Uplift Capacity (tons)
16	10	28	8	**20**	10
18	10	42	9	**33**	16
24	10	71	12	**59**	39

*Note: Net Allowable Capacity equals Allowable Capacity minus Down-drag.

We estimate that the maximum total post-construction settlement of the pile-supported building should be ¼-inch or less, and the differential settlement across the building width should be ⅛-inch or less, based on our past experience with similar structures supported on piles of this type.

Lateral loads can be resisted by passive earth pressures acting against pile caps or grade beams. To fully mobilize the passive pressure resistance, excavated areas around the pile caps or grade beams should be backfilled with compacted granular fill. An allowable passive soil pressure of 300 pcf (equivalent fluid weight), and a friction factor of 0.35 can be used for compacted granular fill around the pile caps.

Pile performance is dependent upon how the piles are installed and into what embedment and bearing strata the piles are installed. Since a completed pile in the ground cannot be directly observed, it is critical that judgment and experience be used as a basis for determining the embedment length and acceptability of a pile. Therefore, we recommend that GEO Group Northwest, Inc. be retained to monitor the pile installation operation, collect and interpret installation data and verify suitable bearing stratum. We also suggest that the contractor's equipment and installation procedure be reviewed by GEO Group Northwest, Inc. before work help avoid problems which may adversely affect the progress of the work.

Closing

Please feel welcome to contact us if you have any questions.

Sincerely,

GEO GROUP NORTHWEST, INC.

Keith Johnson
Project Geologist

William Chang, P.E.
Principal

Enclosure:

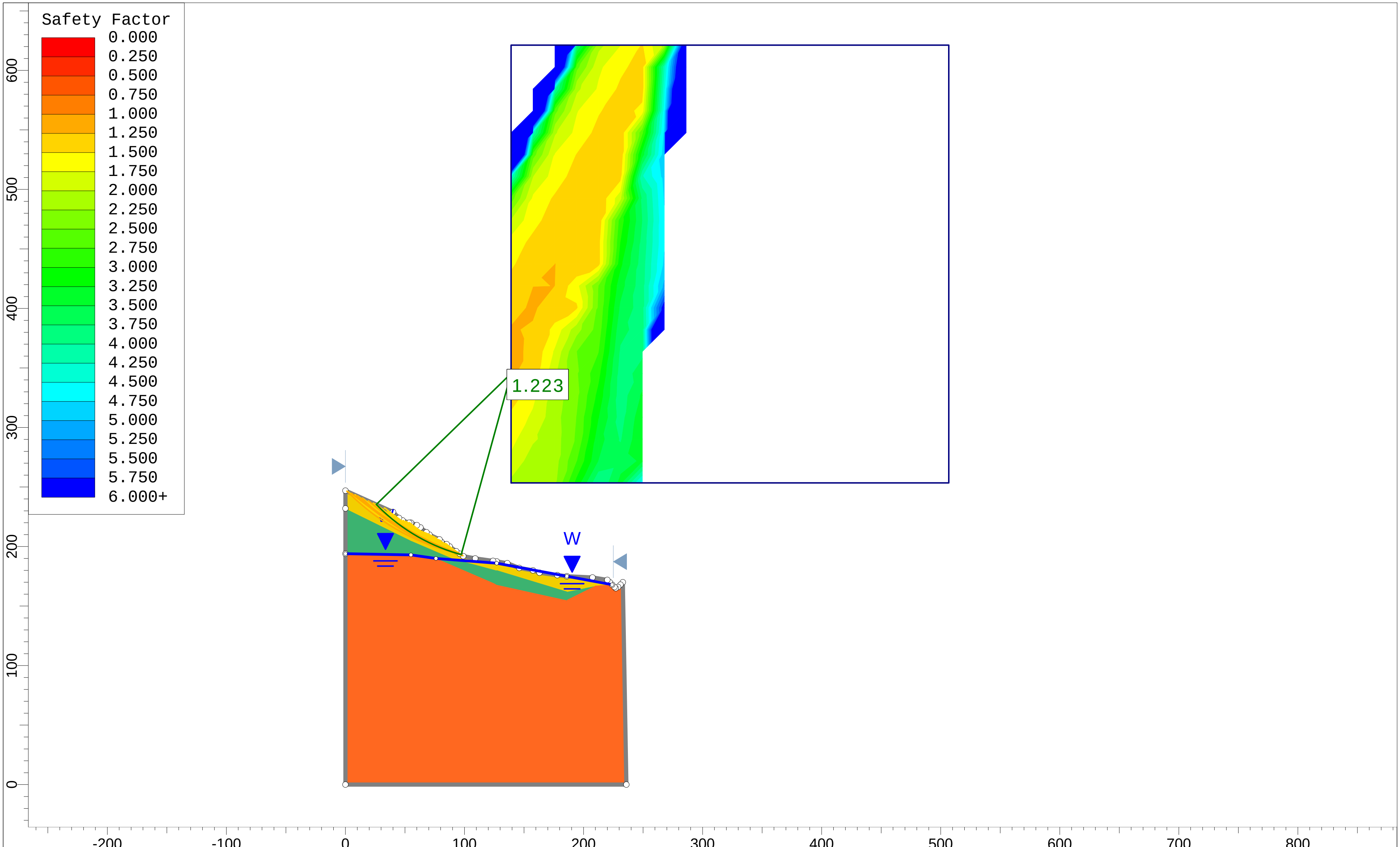
Attachment 1 – Additional Slope Stability Analysis Results

ATTACHMENT 1

G-3837

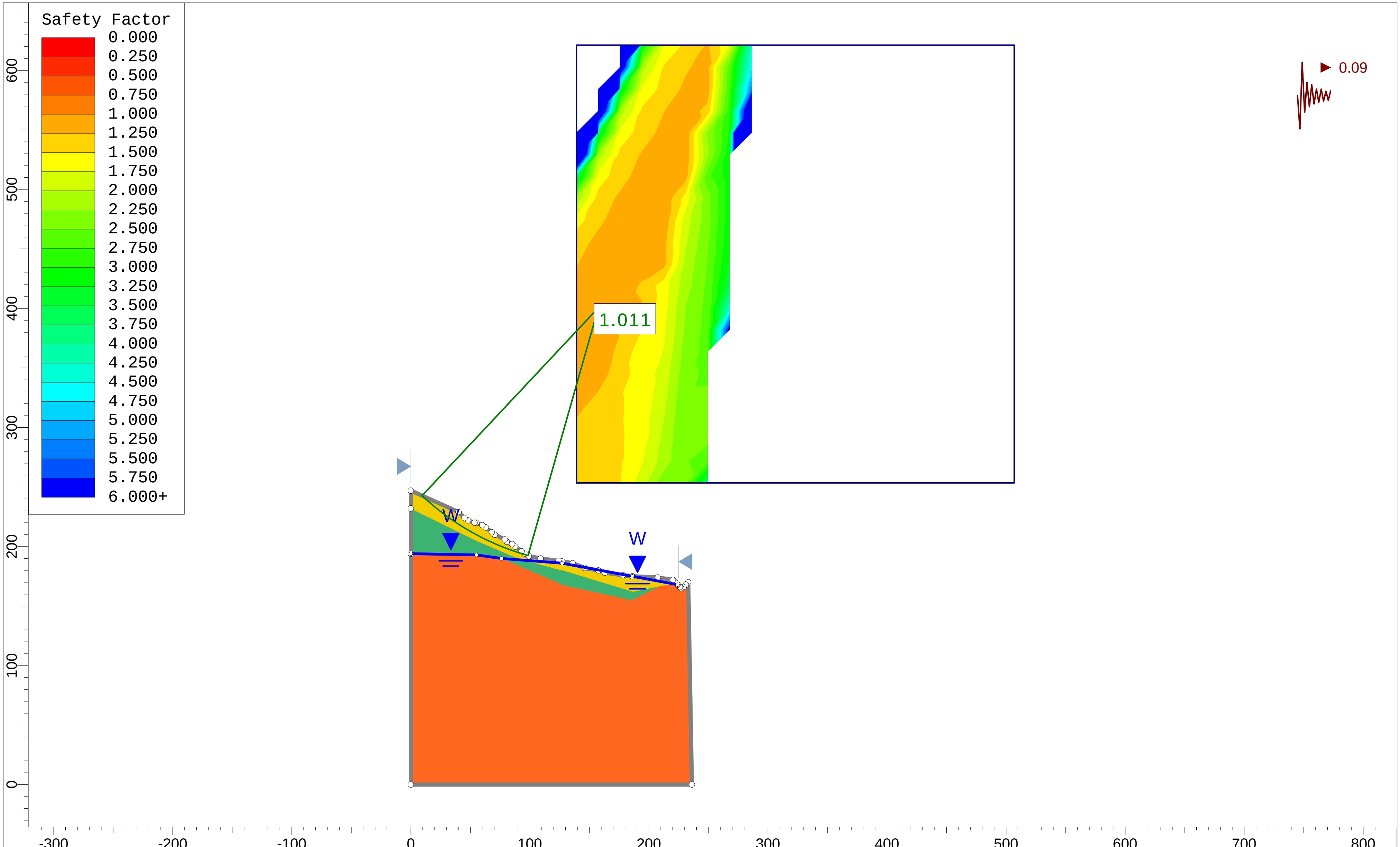
ADDITIONAL SLOPE STABILITY ANALYSIS RESULTS

GEO Group Northwest, Inc.



Project	SLIDE - An Interactive Slope Stability Program
Analysis Description	MI Treehouse - Existing Condition, Static Case
Drawn By: KJ	Scale: 1:1338
Company	
Date: 2/4/2016	File Name: MI 1.slim

SLIDEINTERPRET 7.010



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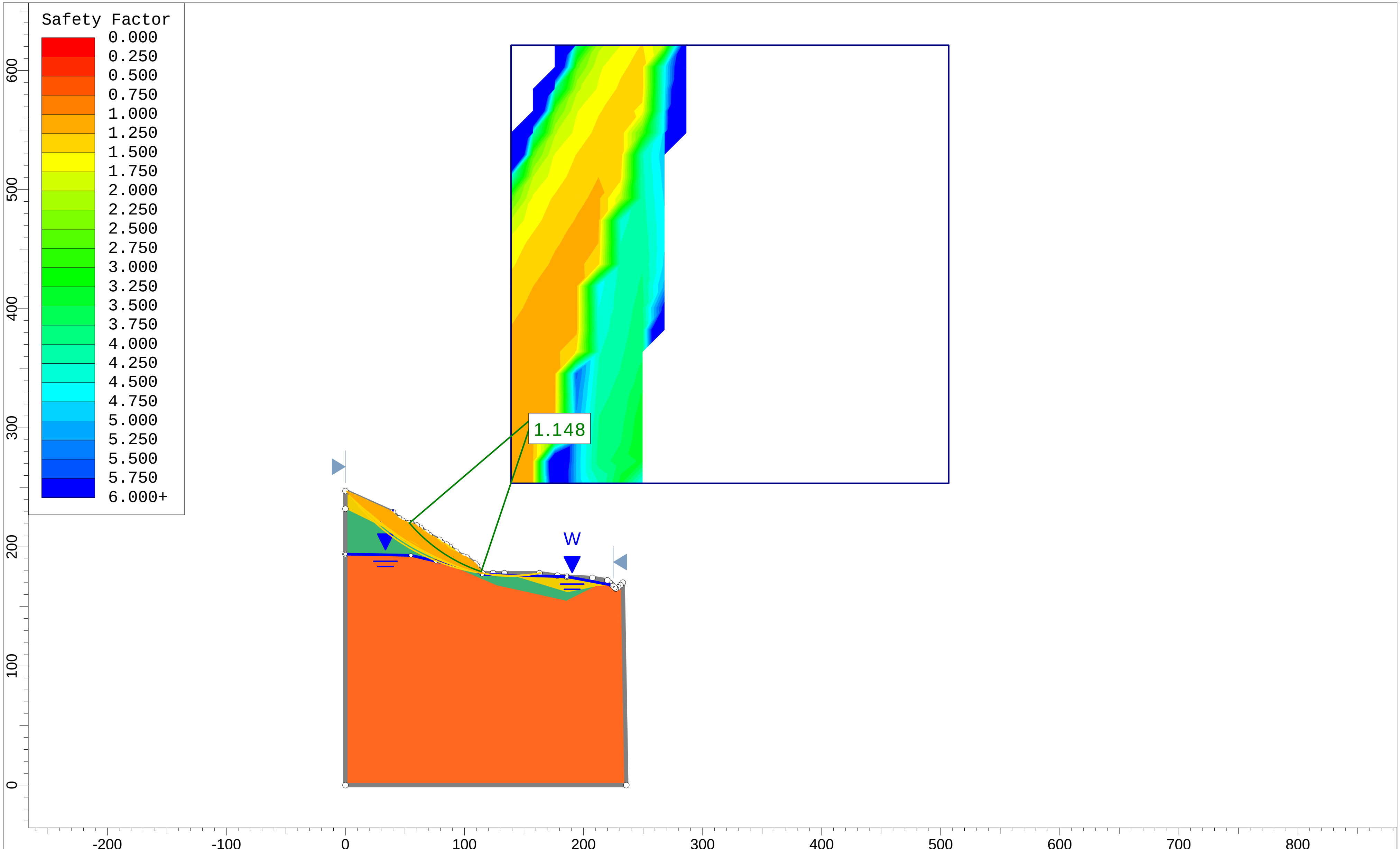
Analysis Description	MI Treehouse - Existing Condition, Seismic Case		
Drawn By	KJ	Scale	1:1338
Company			
Date	2/4/2016	File Name	MI 1S09.slim

SLIDEINTERPRET 7.010



SLIDEINTERPRET 7.010

Project	SLIDE - An Interactive Slope Stability Program		
Analysis Description	MI Treehouse - Temporary Excavation, Static Case		
Drawn By KJ	*Scale* 1:1338	*Company*	
Date 2/4/2016		*File Name* MI 2.slim	