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March 19, 2018

Mr. Matthew Steel
15600 NE 8th Street, B-1, #414
Bellevue, Washington 98008

Geotechnical Engineering Report
4717 Ames Lake Carnation Road
Redmond, Washington
RN File No. 3211-001A

Dear Mr. Steel:

This letter serves as a transmittal for our report for the project located at 4717 Ames Lake Carnation Road in Redmond, Washington. Development plans consist of constructing a 3-story single family residence with a daylight basement. The proposed development is located within a steep slope critical area and buffer. It must therefore meet the requirements of a critical area alteration exception per Section 21A.24.070(3) of the King County Municipal Code (KCMC). Our analysis determined that the proposed development meets the criterion set forth in this section. The subsurface soils encountered are capable of providing support for the planned building.

We appreciate the opportunity of working with you on this project. If you have any questions regarding this report, please contact us.

Sincerely,

R.B. Powell

Rick B. Powell, PE
Principal Engineer

JHA:BAG:RBP:am

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INTRODUCTION

This report presents the results of our geotechnical engineering investigation at your proposed single-family residential project in the Redmond area of King County, Washington. The site is located at 4717 Ames Lake Carnation Road, as shown on the Vicinity Map in Figure 1.

You have requested that we complete this report to evaluate subsurface conditions and provide recommendations for site development. For our use in preparing this report, we have been provided with a Boundary and Topographic Survey prepared by DOWL dated July 7, 2017, a Critical Areas Designation Letter prepared by King County Department of Permitting and Environmental Review dated February 2, 2017, and an undated Site Plan and Elevation drawings prepared by the project designer Joe Molina.

PROJECT DESCRIPTION

The development will consist of a 3-story single-family residence with a daylight basement. Proposed grading includes cuts on the order of 10 to 12 feet in height for the daylight basement stem wall and the driveway access. A fill wall is proposed on the eastern side of the site with an exposed height of 8 feet. We have not been provided with a drainage plan.

SCOPE

The purpose of this study is to explore and characterize the subsurface conditions and present recommendations for site development. Specifically, our scope of services as outlined in our Services Agreement, dated May 25, 2017, includes the following:

1. Review available geologic maps for the site.
2. Mark the site for utility locates.
3. Explore the subsurface soil and groundwater conditions in the area of the planned building and on the slope with test pits with a subcontracted excavator.
4. Evaluate pertinent physical and engineering characteristics of the soils encountered in the test pits.
5. Perform a slope stability analysis.
6. Prepare a geotechnical report containing the results of our subsurface explorations and our conclusions and recommendations for geotechnical design elements of the project.

SITE CONDITIONS

Surface Conditions

The project site is about 0.86 acres in size and has maximum dimensions of approximately 161 feet in the north-south direction and approximately 252 feet in the east-west direction. Access to the site is provided by Ames Lake Carnation Road to the east. The site is bordered by existing residential acreage to the north, south and west. A layout of the site is shown on the Site Plan in Figure 2.

The ground surface within the site is generally moderately to steeply sloping down to the east. Previous grading for old roads and trails was observed within the entirety of the slope. The grading appears to be localized cuts and fills for site access. A small fill area exists in the southwestern corner of the site. Downslope to the east we observed existing cuts for old roads that were on the order of 8 to 12 feet in height. Within the middle portion of the site exists a flat to gently sloping area below an existing excavation cut that contained a small shed with a second small outbuilding located to the northwest, further upslope. An old graded driveway extends from the southeastern corner of the lot up to the graded flatter area of the parcel Old growth stumps approximately 75 to 100 years in age were observed on the western portion of the slope. Small to medium sized evergreen trees were also observed on the slope with an age range of approximately 50 to 75 years. Deciduous trees with underbrush vegetation are located within the entirety of the slope. Landscaped areas with grassy lawn are located around the two outbuildings in the middle portion of the site. Small hand dug explorations for a septic system design were observed in the western quarter of the site.

Geology

Most of the Puget Sound Region was affected by past intrusion of continental glaciation. The last period of glaciation, the Vashon Stade of the Fraser Glaciation, ended approximately 14,000 years ago. Many of the geomorphic features seen today are a result of scouring and overriding by glacial ice. During the Vashon Stade, areas of the Puget Sound region were overridden by over 3,000 feet of ice. Soil layers overridden by the ice sheet were compacted to a much greater extent than those that were not. Part of a typical glacial sequence within the area of the site includes the following soil deposits from newest to oldest:

> **Artificial Fill (af)** – Fill material is often locally placed by human activities, consistency will depend on the source of the fill. The thickness and expanse of this material will be dependent on the extent of fill required to grade land to the desired elevations. Density of the fill will depend on earthwork activities and compaction efforts made during the placement of the material.
>
> **Recessional Outwash (Qvr)** – These deposits were derived from the stagnating and receding Vashon glacier and consist mostly of stratified sand and gravel, but include unstratified ablation and melt-out deposits. Recessional deposits were not compacted by the glacier and are typically not as dense as those that were.
>
> **Vashon Till (Qvt)** – The till is a non-sorted mixture of clay, sand, pebbles, cobbles and boulders, all in variable amounts. The till was deposited directly by the ice as it advanced over and eroded irregular surfaces of previously deposited formations and sediments. The till was well compacted by the advancing glacier and exhibits high strength and stability. Drainage is considered very poor in the till.
>
> **Advance Outwash (Qva)** – The advance outwash typically is a thick section of mostly clean, pebbly sand with increasing amounts of gravel higher in the section. The advance outwash was placed by the advancing glaciers and was overridden and well compacted by the glacier.
>
> **Advance Glaciolacustrine Deposits (Qglv)** – This fine-grained glacial flour was deposited in proglacial lakes in the Puget Lowland. The clayey silt, silty clay, fine sand

and diamicton is typically in a very stiff to hard or dense condition and contain lenses of hard organics.

The geologic units for this area are mapped on the Geologic Map of the Carnation 7.5-minute Quadrangle, Snohomish and King County, Washington, by Joe D. Dragovich, Heather A. Littke, Megan L. Anderson, Gregory R. Wessel, Curtis J. Koger, Jennifer H. Saltonstall, James H. Macdonald Jr. Shannon A. Mahan, and S. Andrew Dufrane (U.S. Geological Survey, 2010). The site is mapped as being underlain by advance glaciolacustrine deposits. Our explorations encountered fill and glaciolacustrine deposits.

Explorations

We explored subsurface conditions at the site on June 22nd, 2017 by excavating six test pits with a mini trackhoe. The test pits were excavated to depths of 6.0 to 9.0 feet below the ground surface. We also performed small hand dug explorations against the existing cuts in the western portion of the site. The explorations were located in the field by a representative from this firm who also examined the soils and geologic conditions encountered, and maintained logs of the test pits. The approximate locations of the test pits are shown on the Site Plan in Figure 2. The soils were visually classified in general accordance with the Unified Soil Classification System, a copy of which is presented as Figure 3. The logs of the test pits are presented in Figures 4 through 5.

Subsurface Conditions

A brief description of the conditions encountered in our explorations is included below. For a more detailed description of the soils encountered, review the Test Pit Logs in Figures 4 through 5.

In general our site explorations encountered a surficial layer of topsoil and fill that consisted of loose dark brown silty sand with roots and silty fine to medium sand with gravel and trace cobbles. In all explorations but Test Pit 2 this was underlain by a weathered zone consisting of medium dense silty sand with gravel and medium stiff silt with fine sand. Underlying the topsoil in Test Pit 2 and beneath the weathered zone in the remaining test pits, we encountered medium dense to very dense silty fine to coarse sand with gravel and trace cobbles to the depths explored.

Hand dug explorations against the westernmost existing cuts exposed dense/stiff silty sand with gravel and cobbles to sandy silt with trace gravel. Within the middle portion of the site the exposed cut consisted of very dense silty fine sand with trace gravel.

Hydrologic Conditions

Groundwater seepage was not encountered in our explorations although rust staining was observed in Test Pit 1 at depths ranging from 4 to 6 feet. The medium dense to dense glaciolacustrine deposits interpreted to underlie the site are considered poorly draining. During the wetter times of the year, we expect perched water conditions will occur as pockets of water on top of cemented or finer grained layers. Perched water does not represent a regional groundwater "table" within the upper soil horizons. Volumes of perched groundwater vary depending upon the time of year and the upslope recharge conditions.

CONCLUSIONS AND RECOMMENDATIONS

General

The proposed development is located within a steep slope critical area and buffer. For the alteration to be approved by the governing jurisdiction of King County it must meet the requirements of an Alteration Exception for a nonlinear alteration per Section 21A.24.070(3) of the KCMC. We have provided analysis for each criterion of the Alteration Exception in addition to performing slope stability analysis of the proposed development. Our analysis determined that, provided the recommendations of this report are followed, the proposed development meets the criteria for an alteration exception for a nonlinear alteration. There is the potential that the surficial soils on the steeper sections of the slope could slough over time. Any slough events are expected to be surficial, and are affected by surface water and man-made impacts.

Walls proposed on slopes should be embedded such that at least 10 feet of separation from the wall to the face of the slope is maintained. Based on our analysis of the proposed wall locations on the slope east of the proposed residence, we estimate this will require about 6 feet of wall embedment.

It is our opinion that the site is compatible with the planned development. The underlying medium dense to very dense glacial deposits are capable of supporting the planned structures and pavements. We recommend that the foundations for the structures extend through any fill, topsoil, loose, or disturbed soils, and bear on the underlying medium dense or firmer, native glacial soil, or on structural fill extending to these soils. Based on our site explorations, we anticipate these soils will generally be encountered at typical footing depths.

Geologic Hazards

Landslide Hazards: The core of the site is inferred to be composed of glacially overridden soils. We consider these soils to be of high strength and stable with regard to deep-seated slope failures. We did not observe indications of surficial seepage on the site, nor did we observe indications of shallow or deep-seated slope failures. The steepest portions of the slope appeared to have been created through cut excavation for old roads and trails. We estimate the age of these cuts to be in the range of 50 to 75 years based on the tree growth on and around the cuts. These cuts appeared to be performing well. The mature evergreen trees present on the slope were straight and did not exhibit curving that would indicate past slope movement. There is a potential that the surficial soils on the steeper sections of the slope could slough over time. Any slough events are expected to be surficial, and are affected by surface water and man-made impacts. The risk of slough events can be minimized if proper drainage is installed, vegetation on the slope is maintained, and yard waste and other debris are kept off the slopes. We expect if a slough event were to occur, it would be small in scale and relatively shallow. We did not observe any indication of recent sloughing on the site.

The Critical Areas Designation letter prepared by the King County Department of Permitting and Environmental Review identified steep slope hazard areas within the parcel. Steep slope hazard areas are defined in KCMC Section 21A.06.1230 as an area with a slope of forty percent inclination or more and with a vertical elevation change of at least ten feet. We have identified on the Site Plan those areas of the parcel that meet this definition.

The planned development proposes alteration of a steep slope hazard area. In this case the development must meet the requirements of an Alteration Exception for a nonlinear alteration to a steep slope hazard area, found in Section 21A.24.070(3) of the KCC. For the alteration to be approved by the county all of the following criteria listed in below must be met. For clarity we have provided our response to each criterion in italics.

a) **There is no feasible alternative to the development proposal with less adverse impact on the critical area.** *The topography within the parcel is such that any building footprint location requires a daylight basement for the development to be feasible. The proposed footprint location is selected to use the flattest topographical portion of the parcel behind the house. Locating the footprint in this area minimizes the amount of site grading to provide sufficient ground surface slope for proper drainage from the house foundation per IBC Section 1804.3. This location also minimizes the height of excavation cuts needed to construct the building foundation and driveway. Other footprint location options would result in the ground surface sloping directly to the back of the house with no buffer to the toe of the slope and would require significant additional grading to create a buffer behind the house.*

b) **The alteration is the minimum necessary to accommodate the development proposal.** *The development proposal is sited within the property to utilize the natural topography within the parcel to the maximum extent possible and minimize the grading required. This was done by selecting the footprint location with the flattest topographical portion of the parcel behind the house. This minimized excavation cut heights for the daylight basement, driveway access and additional grading to create a buffer from the toe of the slope behind the house and to provide drainage away from the structure.*

c) **The approval does not require the modification of a critical area development standard established by this chapter, except as set forth in subsection A.2.i of this section.** *The other critical areas identified within the parcel are a Category I critical aquifer recharge area and erosion hazard areas. The development standards for erosion hazard areas have been addressed in detail in the* ***Erosion Hazard*** *section of this report. The modification of the steep slope critical area does not require modification of the erosion hazard area development standards. The development standards for Critical Aquifer Recharge Areas are found in Section 21A.24.316 of the KCC. The applicable standard to the proposed development is A.13 of this section which states that on lots smaller than one acre, an on-site septic system is not allowed unless: the system is approved by the Washington State Department of Health and has been listed by the*

Washington State Department of Health as meeting treatment standard N as provided in WAC chapter 426-172A; or the Seattle-King County department of public health determines that the systems required under subsection A.13.a will not function on the site. Provided this condition is met the modification of the steep slope critical area will not require modification of the Critical Aquifer Recharge area development standards.

d) **The development proposal does not pose an unreasonable threat to the public health, safety or welfare on or off the development proposal site and is consistent with the general purposes of this chapter and the public interest.** *As discussed further in the Slope Stability Analysis section of this report we performed a slope stability analysis to assess the safety factor of the proposed development as it relates to the surrounding area. We analyzed two cross sections of the proposed development, A-A' and B-B', under static and dynamic conditions to compare the factor of safety to the existing condition of the slope. The typical factor of safety benchmark in the region for the static condition is 1.5 and for dynamic analysis the safety factor benchmark is 1.15.The developed conditions for the static and seismic cases were found to be above these benchmarks and did not decrease the factor of safety when compared to the existing conditions. Based on this analysis it is our opinion that the development does not pose an unreasonable threat to the public health, safety or welfare on or off the development proposal site.*

e) **For dwelling units, no more than five thousand square feet or ten percent of the site, whichever is greater, may be disturbed by structures, building setbacks, or other land alteration, including grading, utility installations and landscaping, but not including the area used for a driveway or for an on-site sewage disposal system.** *It is estimated that on the order of 3,000 square feet will be disturbed by the scope of the proposed alteration. This does not include the area used for driveway access.*

f) **To the maximum extent practical, access is located to have the least adverse impact on the critical area and the critical area buffer.** *The proposed driveway access for the development has been located to follow the alignment of the old previously graded roadway present on the slope. This minimizes the amount of excavation needed to cut in the driveway within the critical area as compared to all other access options.*

g) **The critical area is not used as a salmonid spawning area.** *There are no mapped salmonid spawning areas within the project parcel.*

h) **The director may approve an alteration in a category II, II and IV wetland for development of a public school facility.** *There are no mapped wetlands located within the project parcel.*

i) **The director may approve an alteration to the elevation or dry flood proofing standards in KCC 21A.24.240.F.1 or 21A.24.240.F.2 for nonresidential agricultural**

accessory buildings that equal or exceed a maximum assessed value of sixty-five thousand dollars if the development proposal meets the criteria in subsection A.2 of this section and the standards in KCC 21A.24.240.F.4 through 21A.24.240.G. *This criteria is not applicable to the scope of this report.*

Slope Stability Analysis: We analyzed global stability using a computer program by Rocscience known as Slide, version 6.0. Slide is a two-dimensional, limit-equilibrium, slope stability program for evaluating the safety factor or probability of failure, of circular or non-circular failure surfaces in soil or rock slopes. Slide analyzes the stability of slip surfaces using vertical slice limit equilibrium methods. The sections were analyzed using the Simplified Bishops method of slices. Slide generates random potential failure surfaces and determines their corresponding factors of safety with respect to failure. The factor of safety is defined as the ratio of the internal soil strength divided by the gravity driving forces that cause failure. By generating a large number of random surfaces, the factor of safety can be obtained as the lowest number calculated.

Existing conditions were used to calculate the static slope stability in Cross Sections A-A' and B-B'. These results were then compared to analyses of the same cross sections in the fully developed condition. A third analysis was performed on all cross sections with seismic loads evaluated for the developed conditions. The cross sections are shown on the Site Plan in Figure 2. The results of the existing and developed condition analyses are shown in Figures 6 through 11. Analysis for each condition was performed across the entire cross section however Figures 7 and 8, and 10 and 11 have been zoomed in to the developed areas.

The typical factor of safety benchmark in the region for the static condition is 1.5 and for seismic analysis the safety factor benchmark is 1.15.The existing conditions in cross sections A-A' and B-B' were both found to be above the 1.5 factor of safety benchmark. The developed condition analysis for A-A' presumed sufficient embedment of the wall located on the east side of the proposed residence to provide at least 10 feet of separation to the face of the slope. In the analysis this required at least 6 feet of wall embedment. The developed condition analyses for both A-A' and B-B' determined that, provided the recommendations of this report are followed, the proposed development will not reduce the factor of safety of the slope below pre-development conditions. In addition, the seismic factors of safeties were above 1.15 for each section.

Erosion Hazard: The Critical Areas Designation letter prepared by the King County Department of Permitting and Environmental Review identified erosion hazard areas within the parcel. Erosion hazards areas are defined in Section 21A.06.415 of the KCC as an area underlain by soils that are subject to severe erosion when disturbed. These soils include any of the following when they occur on slopes inclined at fifteen percent or more:

A. The Alderwood gravely sandy loam (AgD)
B. The Alerwood and Kitsap soils (AkF)
C. The Beausite gravely sandy loam (BeD and BeF)

D. The Kitsap silt loam (KpD)
E. The Ovall gravely loam (OvD and OvF)
F. The Ragnar fine sandy loam (RaD)
G. The Ragnar-Indianola Association (RdE)

We reviewed the Web Soil Survey by the Natural Resources Conservation Service (NRCS) to determine the SCS classification of the site surface soils. The site surface soils are classified as Alderwood gravelly sandy loam (AgD).

Section 21A.24.220 provides development standards that apply to development proposals and alterations on sites containing erosion hazards areas. These standards state that clearing in an erosion hazards area is allowed only from April 1 to October 1. There are three exceptions to this standard as follows; clearing of up to fifteen-thousand square feet within the erosion hazard area may occur at any time, clearing of noxious weeds may occur at any time, and forest practices regulated by the department are allowed at any time with a clearing and grading permit.

It is estimated that the proposed development will not require clearing greater than 5,000 square feet within the erosion hazard area. Best Management Practices (BMP) for erosion and sediment control should be used at the site both on a temporary and permanent basis. We do not anticipate that the project will require unusual or extreme methods.

Seismic Hazard: It is our opinion based on our subsurface explorations that the Soil Profile in accordance with the 2015 International Building Code (IBC) is Site Class C with Seismic Design Category D. We used the US Geological Survey program "U.S. Seismic Design Maps Web Application." The design maps summary report for the 2012/15 IBC is included in this report as Appendix A.

Additional seismic considerations include liquefaction potential and amplification of ground motions by soft soil deposits. The liquefaction potential is highest for loose sand with a high groundwater table. The underlying very dense glacial soils are considered to have a very low potential for liquefaction and amplification of ground motion and seismically induced lateral spread.

The project is mapped on Faults and Earthquakes in Washington State (Jessica L. Czajkowski and Jeffrey D. Bowman, USGS OFR 2014-05) as located within the Rattlesnake Mountain Fault Zone, which is a southward extension of the Southern Whidbey Island Fault Zone. This is a class A thrust fault with a slip rate category of 0.6 mm/yr. Surface-deforming earthquakes occurred within this fault zone about 2,700 years ago. The potential for surface displacement is considered low based on the amount of glacially derived overburden.

Site Preparation and Grading

The first step of site preparation should be to strip the vegetation, topsoil, or loose soils to expose medium dense or firmer native soils in pavement and building areas. The excavated material should be removed from the site, or stockpiled for later use as landscaping fill. The resulting subgrade should be compacted to a firm, non-yielding condition. Areas observed to pump or yield should be repaired prior to placing hard surfaces.

The on-site glacial soil likely to be exposed during construction is considered highly moisture sensitive, and the surface will disturb easily when wet. We expect these soils would be difficult, if not impossible, to compact to structural fill specifications in wet weather. We recommend that earthwork be conducted during the drier months. Additional expenses of wet weather or winter construction could include extra excavation and use of imported fill or rock spalls. During wet weather, alternative site preparation methods may be necessary. These methods may include utilizing a smooth-bucket trackhoe to complete site stripping and diverting construction traffic around prepared subgrades. Disturbance to the prepared subgrade may be minimized by placing a blanket of rock spalls or imported sand and gravel in traffic and roadway areas. Cutoff drains or ditches can also be helpful in reducing grading costs during the wet season. These methods can be evaluated at the time of construction.

Structural Fill

General: All fill placed beneath buildings, pavements or other settlement sensitive features should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards, and is observed by an experienced geotechnical professional or soils technician. Field observation procedures would include the performance of a representative number of in-place density tests to document the attainment of the desired degree of relative compaction.

Materials: Imported structural fill should consist of a good quality, free-draining granular soil, free of organics and other deleterious material, and be well graded to a maximum size of about 3 inches. Imported, all-weather structural fill should contain no more than 5 percent fines (soil finer than a Standard U.S. No. 200 sieve), based on that fraction passing the U.S. 3/4-inch sieve. The use of on-site soil as structural fill will be dependent on moisture content control. Some drying of the native soils may be necessary in order to achieve compaction. During warm, sunny days this could be accomplished by spreading the material in thin lifts and compacting. Some aeration and/or addition of moisture may also be necessary. We expect that compaction of the native soils to structural fill specifications would be difficult, if not impossible, during wet weather.

Fill Placement: Following subgrade preparation, placement of the structural fill may proceed. Fill should be placed in 8- to 10-inch-thick uniform lifts, and each lift should be spread evenly and be thoroughly compacted prior to placement of subsequent lifts. All structural fill underlying building areas, and within a depth of 2 feet below pavement and sidewalk subgrade, should be compacted to at least 95 percent of its maximum dry density. Maximum dry density, in this report, refers to that density as determined by the ASTM D1557 compaction test procedure. Fill more than 2 feet beneath sidewalks and pavement subgrades should be compacted to at least 90 percent of the maximum dry density. The moisture content of the soil to be compacted should be within about 2 percent of optimum so that a readily compactable condition exists. It may be necessary to overexcavate and remove wet surficial soils in cases where drying to a compactable condition is not feasible. All compaction should be accomplished by equipment of a type and size sufficient to attain the desired degree of compaction.

Temporary and Permanent Slopes

Temporary cut slope stability is a function of many factors, such as the type and consistency of soils, depth of the cut, surcharge loads adjacent to the excavation, length of time a cut remains open, and the presence of surface or groundwater. It is exceedingly difficult under these variable conditions to estimate a stable temporary cut slope geometry. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations, since the contractor is continuously at the job site, able to observe the nature and condition of the cut slopes, and able to monitor the subsurface materials and groundwater conditions encountered.

For planning purposes, we recommend that temporary cuts in the near-surface weathered soils be no steeper than 1.5 Horizontal to 1 Vertical (1.5H:1V). Cuts in the dense to very dense glacial deposits may stand at a 1H:1V inclination.If groundwater seepage is encountered, we expect that flatter inclinations would be necessary.

We recommend that cut slopes be protected from erosion. Measures taken may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes. We do not recommend vertical slopes for cuts deeper than 4 feet, if worker access is necessary. We recommend that cut slope heights and inclinations conform to local and WISHA/OSHA standards.

Final slope inclinations for granular structural fill and the native soils should be no steeper than 2H:1V. Lightly compacted fills, common fills, or structural fill predominately consisting of fine grained soils should be no steeper than 3H:1V. Common fills are defined as fill material with some organics that are "trackrolled" into place. They would not meet the compaction specification of structural fill. Final slopes should be vegetated and covered with straw or jute netting. The vegetation should be maintained until it is established.

Foundations

Conventional shallow spread foundations should be founded on undisturbed, medium dense or firmer soil. If the soil at the planned bottom of footing elevation is not suitable, it should be overexcavated to expose suitable bearing soil. Footings should extend at least 18 inches below the lowest adjacent finished ground surface for frost protection. Minimum foundation widths should conform to IBC requirements. Standing water should not be allowed to accumulate in footing trenches. All loose or disturbed soil should be removed from the foundation excavation prior to placing concrete.

For foundations constructed as outlined above, we recommend an allowable design bearing pressure of 2,500 pounds per square foot (psf) be used for the footing design. IBC guidelines should be followed when considering short-term transitory wind or seismic loads. Potential foundation settlement using the recommended allowable bearing pressure is estimated to be less than 1-inch total and ½-inch differential between footings or across a distance of about 30 feet. Higher soil bearing values may be appropriate with wider footings. These higher values can be determined after a review of a specific design.

Lateral Loads

The lateral earth pressure acting on retaining walls is dependent on the nature and density of the soil behind the wall, the amount of lateral wall movement, which can occur as backfill is placed, and the inclination of the backfill. Walls that are free to yield at least one-thousandth of the height of the wall are in an "active" condition. Walls restrained from movement by stiffness or bracing are in an "at-rest" condition. Active earth pressure and at-rest earth pressure can be calculated based on equivalent fluid density. Equivalent fluid densities (EFW) for active and at-rest earth pressure of for a range of back-slope angles are provided in the table below. These values assume that the on-site soils or imported granular fill are used for backfill, and that the wall backfill is drained. The preceding values do not include the effects of surcharges, such as due to foundation loads or other surface loads. Surcharge effects should be considered where appropriate.

Back-slope inclination	Active pressure EFW (pcf)	At-rest pressure EFW (pcf)
Level	35	55
3H:1V	44	69
2H:1V	52	81

Seismic lateral loads are a function of the site location, soil strength parameters and the peak horizontal ground acceleration (PGA) for a given return period. We used the US Geological Survey program "U.S. Seismic Design Maps Web Application" to compute the PGA for the site. The Design Maps Summary Report is included in Appendix A. The above drained active and at-rest values should be increased by a uniform pressure of 7H psf, respectively, when considering seismic conditions. H represents the wall height.

The above lateral pressures may be resisted by friction at the base of the wall and passive resistance against the foundation. A coefficient of friction of 0.4 may be used to determine the base friction in the native glacial soils. An equivalent fluid density of 250 pcf may be used for passive resistance design. To achieve this value of passive pressure, the foundations should be poured "neat" against the native dense soils, or compacted fill should be used as backfill against the front of the footing, and the soil in front of the wall should extend a horizontal distance at least equal to three times the foundation depth. A resistance factor of 0.67 has been applied to the passive pressure to account for required movements to generate these pressures.

All wall backfill should be well compacted. Care should be taken to prevent the buildup of excess lateral soil pressures due to overcompaction of the wall backfill.

Slabs-On-Grade

Slab-on-grade areas should be prepared as recommended in the **Site Preparation and Grading** subsection. Slabs should be supported on medium dense or firmer native soils, or on structural fill extending to these soils. Where moisture control is a concern, we recommend that slabs be underlain by 6 inches of pea gravel for use as a capillary break. A suitable vapor barrier, such as heavy plastic sheeting, should be placed over the capillary break. An additional 2-inch-thick damp sand blanket can be used to cover the vapor barrier to protect the membrane and to aid in curing the concrete. This will also help prevent cement paste bleeding down into the capillary break through joints or tears in the vapor barrier. The capillary break material should be connected to the footing drains to provide positive drainage.

Drainage

We recommend that runoff from impervious surfaces, such as roofs, driveway and access roadways, be collected and routed to an appropriate storm water discharge system. The finished ground surface should be sloped at a gradient of 5 percent minimum for a distance of at least 10 feet away from the buildings, or to an approved method of diverting water from the foundation, per IBC Section 1804.3. Surface water should be collected by permanent catch basins and drain lines, and be discharged into a storm drain system.

We recommend that footing drains be used around all of the structures where moisture control is important. Where used, footing drains should consist of 4-inch-diameter, perforated PVC pipe that is surrounded by free-draining material, such as pea gravel. Footing drains should discharge into tightlines leading to an appropriate collection and discharge point. For slabs-on-grade, a drainage path should be provided from the capillary break material to the footing drain system. Roof drains should not be connected to wall or footing drains.

Utilities

Our explorations indicate that deep dewatering will not be needed to install standard depth utilities. Anticipated groundwater is expected to be handled with pumps in the trenches. We also expect that some groundwater seepage may develop during and following the wetter times of the year. We expect this seepage to mostly occur in pockets. We do not expect significant volumes of water in these excavations.

The soils likely to be exposed in utility trenches after site stripping are considered highly moisture sensitive. We recommend that they be considered for trench backfill during the drier portions of the year. Provided these soils are within 2 percent of their optimum moisture content, they should be suitable to meet compaction specifications. During the wet season, it may be difficult to achieve compaction specifications; therefore, soil amendment with kiln dust or cement may be needed to achieve proper compaction with the on-site materials.

Pavement Subgrade

The performance of roadway pavement is critically related to the conditions of the underlying subgrade. We recommend that the subgrade soils within the driveway be prepared as described in the **Site Preparation and Grading** subsection of this report. Prior to placing base material, the subgrade soils should be compacted to a non-yielding state with a vibratory roller compactor and then proof-rolled with a piece of heavy construction equipment, such as a fully-loaded dump truck. Any areas with excessive weaving or flexing should be overexcavated and recompacted or replaced with a structural fill or crushed rock placed and compacted in accordance with recommendations provided in the **Structural Fill** subsection of this report.

ADDITIONAL SERVICES

We expect there will be many decisions on how this project will be designed and constructed. We recommend that we complete a plan review to confirm the geotechnical design parameters recommended in this report are satisfied. We should be retained to provide observation and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, and to provide recommendations for design changes, should the conditions revealed during the work differ from those anticipated. As part of our services, we would also evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications.

USE OF THIS REPORT

We have prepared this report for Mr. Matt Steel and his agents, for use in planning and design of this project. The data and report should be provided to prospective contractors for their bidding and estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of subsurface conditions.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractors' methods, techniques, sequences or procedures, except as specifically described in our report, for consideration in design. There are possible variations in subsurface conditions. We recommend that project planning include contingencies in budget and schedule, should areas be found with conditions that vary from those described in this report.

Within the limitations of scope, schedule and budget for our services, we have strived to take care that our services have been completed in accordance with generally accepted practices followed in this area at the time this report was prepared. No other conditions, expressed or implied, should be understood.

o O o

We appreciate the opportunity to be of service to you. If there are any questions concerning this report or if we can provide additional services, please call.

Sincerely,
Robinson Noble, Inc.

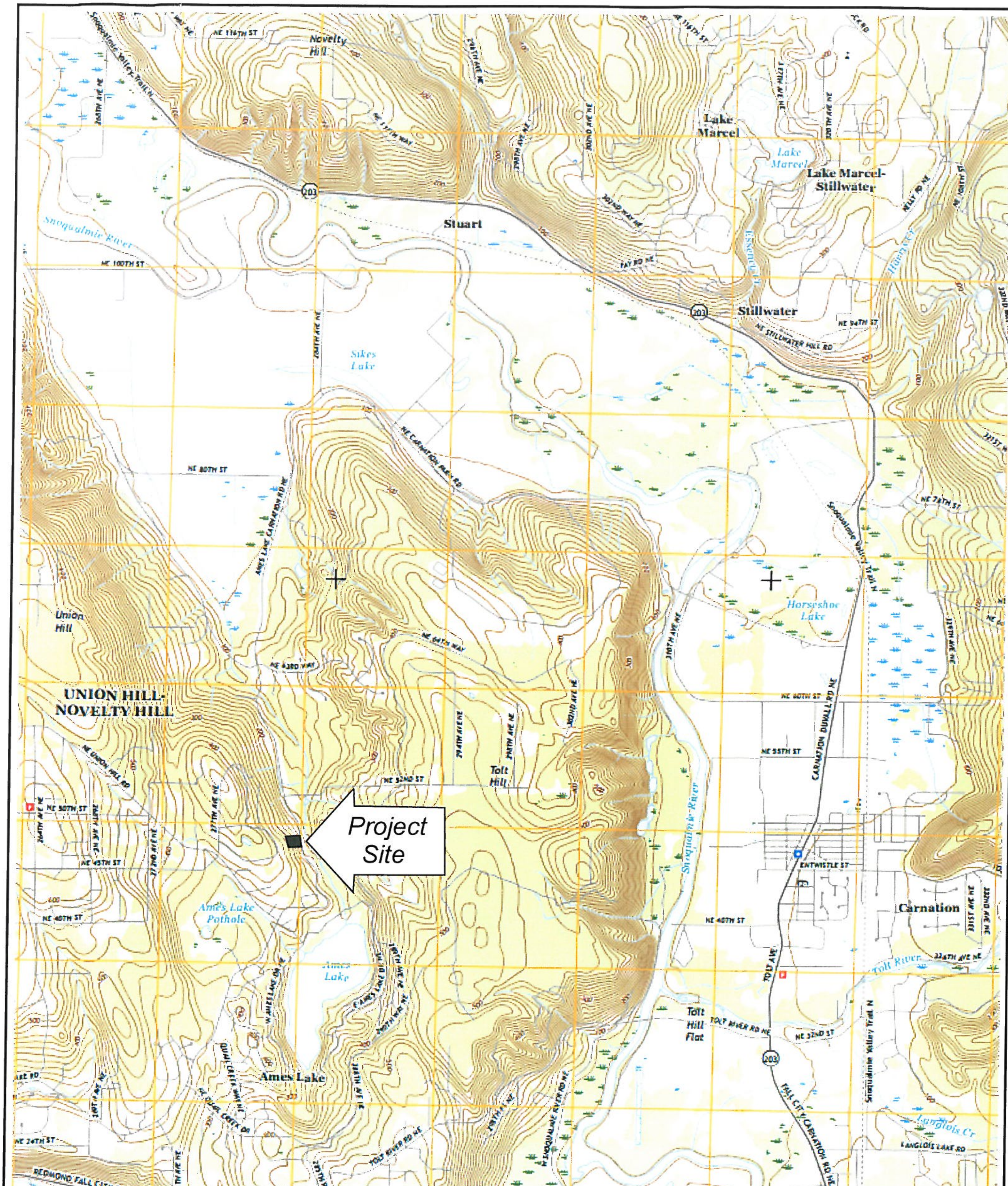
John H. Anderson, EIT
Staff Engineer

3/19/2018

Barbara A. Gallagher, PE
Senior Engineer

JHA:BAG:RBP:am

Eleven Figures
Appendix A



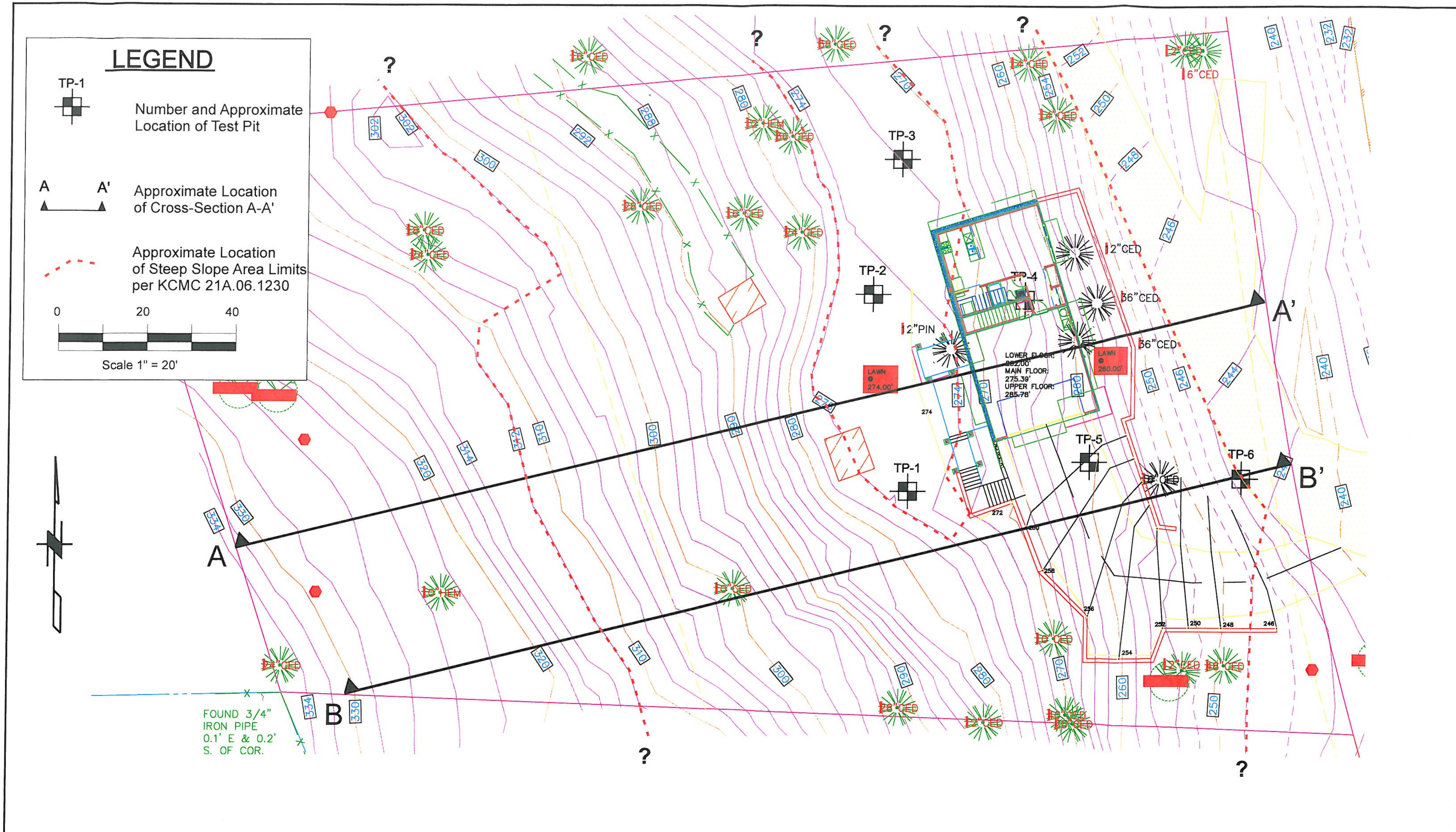
ROBINSON NOBLE

Note:
Basemap taken from USGS US Topo 7.5 minute map for Carnation, WA

Figure 1

Vicinity Map

Matt Steel: 4717 Ames Lake Carnation Road SFR



Note: Basemap taken from "Boundary and Topographic Survey prepared by DOWL dated 07/07/2017

PM: BAG
March 2018
3211-001A

ROBINSON NOBLE

Figure 2
Site Plan
Matt Steel: 4717 Ames Lake - Carnation Road NE

Unified Soil Classification System

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50% RETAINED ON number 200 SIEVE	GRAVEL MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED GRAVEL, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50% OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY-GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE - GRAINED SOILS MORE THAN 50% PASSES NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50%	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50% OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1) Field classification is based on visual examination of soil in general accordance with ASTM D 2488-83.
2) Soil classification using laboratory tests is based on ASTM D 2487-83.
3) Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS

Dry- Absence of moisture, dusty, dry to the touch

Moist- Damp, but no visible water

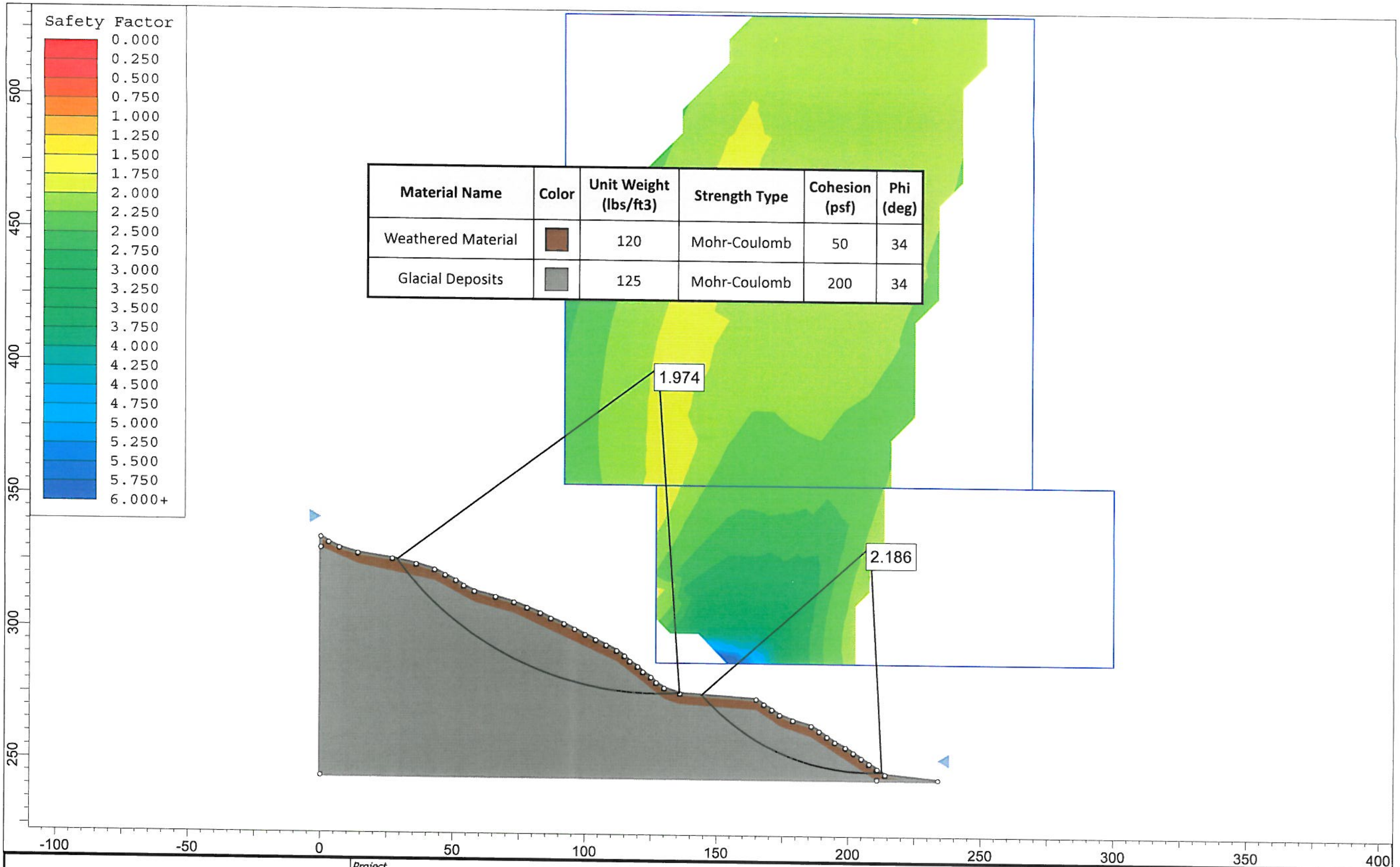
Wet- Visible free water or saturated, usually soil is obtained from below water table

LOG OF EXPLORATION

DEPTH	USC	SOIL DESCRIPTION
TEST PIT ONE		
0.0 – 0.3	SM	Dark brown silty sand with roots (loose, moist) (**Topsoil**)
0.3 – 1.5	SM	Brown gray slightly cemented silty fine to medium sand with gravel trace cobbles (medium dense to dense, moist) (**Fill**)
1.5 – 4.0	SM	Red brown silty fine to medium sand with gravel (medium dense, moist) (**Weathered Zone**)
4.0 – 6.0	SM	Gray brown trace rust staining moderately cemented silty fine to medium sand with gravel trace to with cobbles (very dense, moist)
		Samples were collected at 1.0, 2.0, 4.5, and 5.0 feet Groundwater seepage was not encountered Test pit caving was encountered from 2.0 to 4.0 feet Test pit was completed at 6.0 feet on 6/22/2017
TEST PIT TWO		
0.0 – 0.3	SM	Dark brown silty sand with roots (loose, moist) (**Topsoil**)
0.3 – 3.0	SM/SP-SM	Gray brown silty fine to medium sand to fine to medium sand with silt trace gravel (medium dense to dense, moist)
3.0 – 9.0	SM	Gray silty fine sand (dense, moist)
		Samples were collected at 0.5 and 9.0 feet Groundwater seepage was not encountered Test pit caving was not encountered Test pit was completed at 9.0 feet on 6/22/2017
TEST PIT THREE		
0.0 – 3.0	SM	Brown silty fine to medium sand with gravel trace cobbles (medium dense, moist) (**Fill**)
3.0 – 4.0	ML	Red brown silt with fine sand and roots trace gravel (medium stiff, moist) (**Weathered Zone**)
4.0 – 8.0	GP-GM	Gray brown gravel with silt and sand trace cobbles (medium dense, moist)
		Sample was collected at 1.0, 3.5, and 7.0 feet Groundwater seepage was not encountered Test pit caving was encountered from 4.5 to 6.5 feet Test pit was completed at 8.0 feet on 6/22/2017

LOG OF EXPLORATION

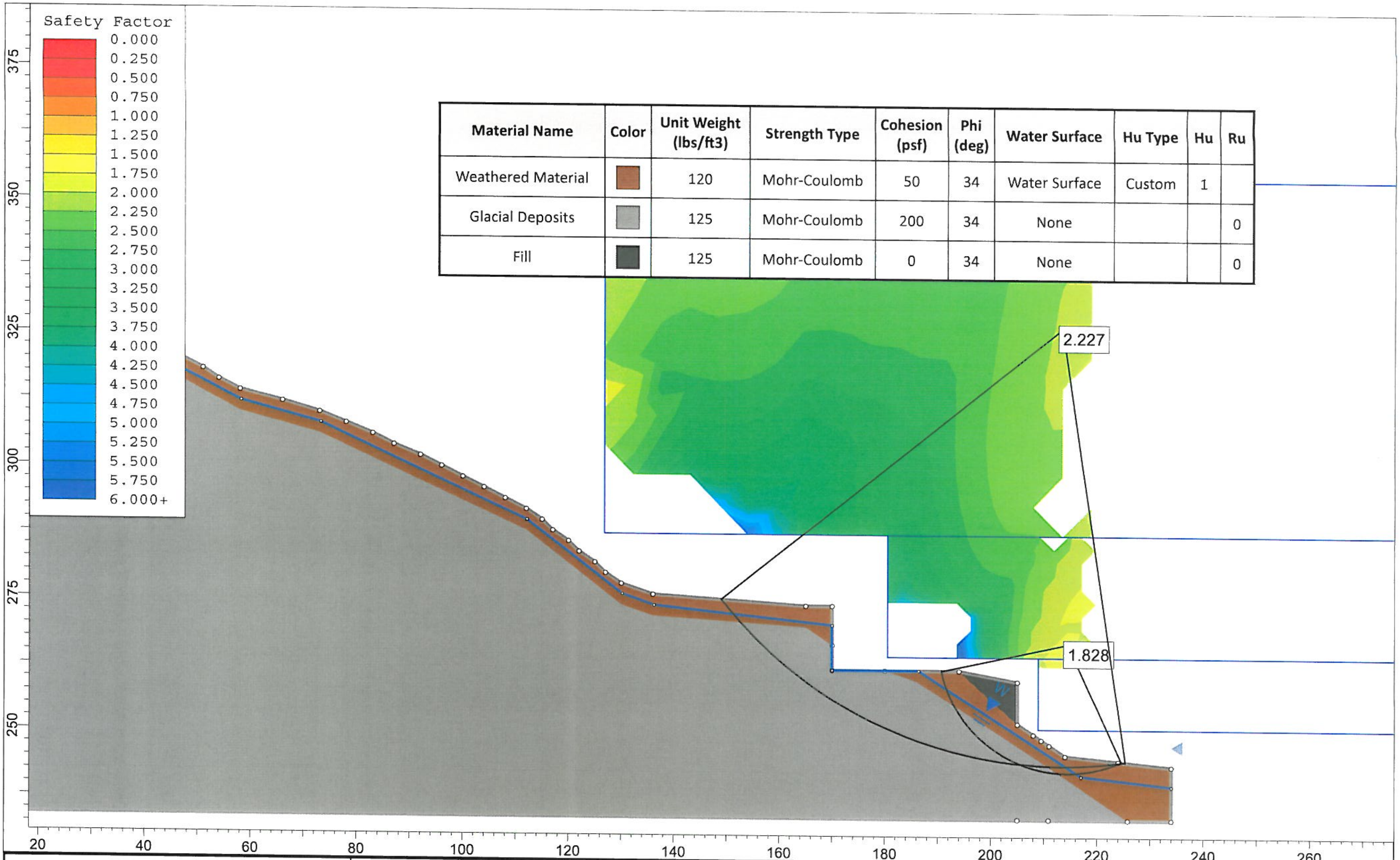
DEPTH	USC	SOIL DESCRIPTION
TEST PIT FOUR		
0.0 – 0.3	SM	Dark brown silty fine sand with roots (loose, moist) (**Topsoil**)
0.3 – 4.0	SM	Brown silty fine to medium sand with gravel trace cobbles and roots (medium dense, moist)
4.0 – 6.0	SM	Gray silty fine to medium sand with gravel trace cobbles (dense to very dense, moist)
		Samples were collected at 2.0 and 5.0 feet Groundwater seepage was not encountered Test pit caving was not encountered Test pit was completed at 6.0 feet on 6/22/2017
TEST PIT FIVE		
0.0 – 0.3	SM	Dark brown silty fine sand with roots (loose, moist) (**Topsoil**)
0.3 – 1.5	SM	Brown silty fine to medium sand with gravel trace cobbles and roots (loose to medium dense, moist) (**Fill**)
1.5 – 2.0	SM	Brown silty sand with roots trace gravel (loose to medium dense, moist) (**Buried Topsoil**)
2.0 – 6.0	SM	Red brown silty fine to medium sand with gravel trace cobbles (medium dense, moist) (**Weathered Zone**)
6.0 – 7.0	SM	Brown gray silty fine to medium sand with gravel trace cobbles (dense, moist)
		Samples were collected at 1.0, 3.0, and 7.0 feet Groundwater seepage was not encountered Test pit caving was encountered from 4.0 to 6.0 feet Test pit was completed at 7.0 feet on 6/22/2017
TEST PIT SIX		
0.0 – 0.3	SM	Dark brown silty fine sand with roots (loose, moist) (**Topsoil**)
0.3 – 2.0	SM	Red brown silty sand with gravel trace cobbles (medium dense, moist) (**Weathered Zone**)
2.0 – 6.0	SM	Gray silty fine to coarse sand with gravel trace cobbles (dense, moist)
6.0 – 9.0	SM	Gray silty fine sand trace gravel (dense, moist)
		Samples were collected at 5.0 and 7.0 feet Groundwater seepage was not encountered Test pit caving was not encountered Test pit was completed at 9.0 feet on 6/22/2017



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Weathered Material		120	Mohr-Coulomb	50	34
Glacial Deposits		125	Mohr-Coulomb	200	34

SLIDEINTERPRET 6.039

Project	4717 Ames Lake Carnation Road, Figure 6		
Analysis Description	A-A' Existing Conditions		
Drawn By	JHA	Scale	1:600
Company			
Date	1/31/2018, 1:56:17 PM	File Name	A-A' Existing.slim

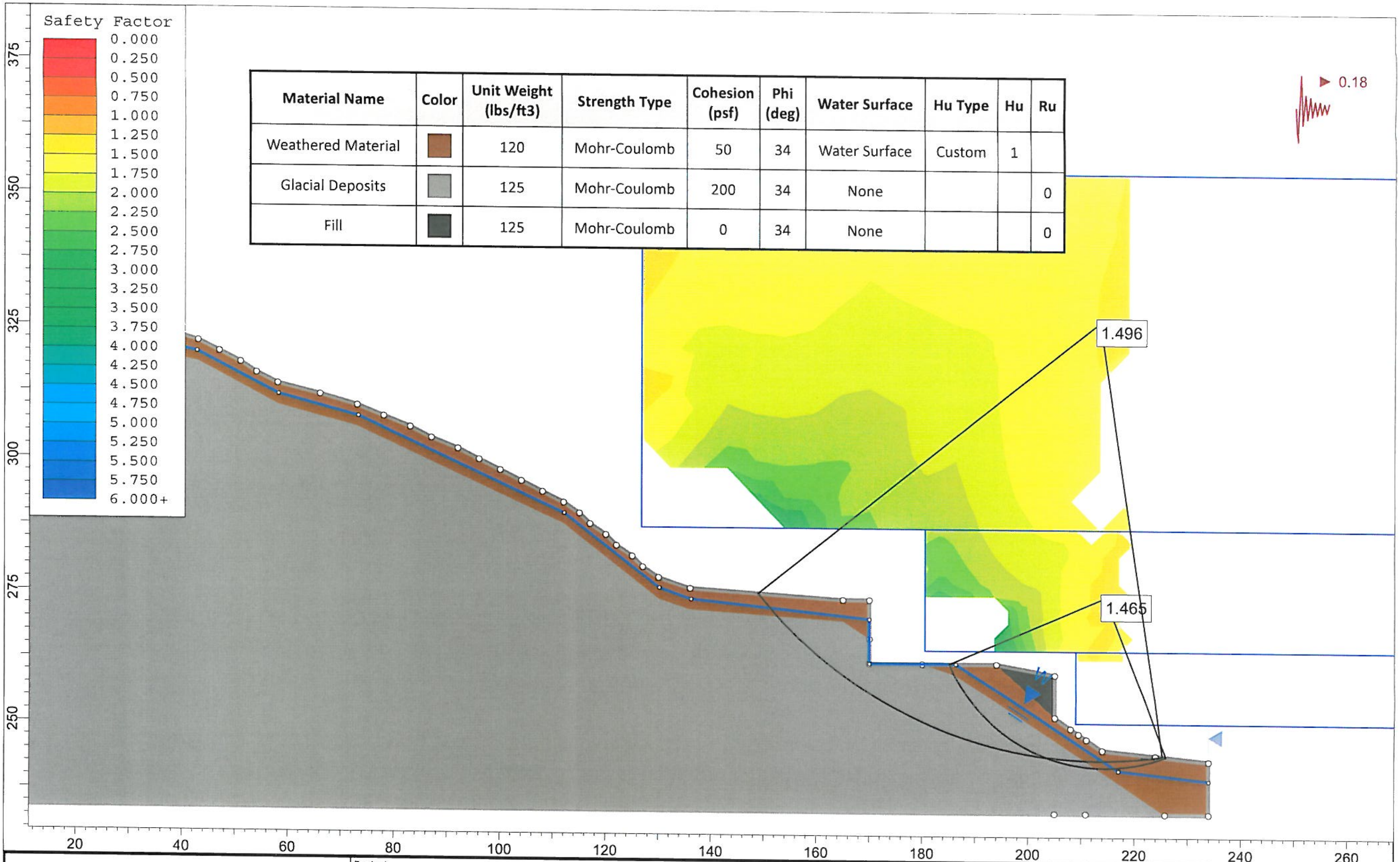


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Weathered Material		120	Mohr-Coulomb	50	34	Water Surface	Custom	1	
Glacial Deposits		125	Mohr-Coulomb	200	34	None			0
Fill		125	Mohr-Coulomb	0	34	None			0

Project	4717 Ames Lake Carnation Road, Figure 7		
Analysis Description	A-A' Developed Conditions		
Drawn By	JHA	Scale	1:300
Company			
Date	1/31/2018, 1:56:17 PM	File Name	A-A' Developed, localized fill wall, water surface.slim

rocscience

SLIDEINTERPRET 6.039

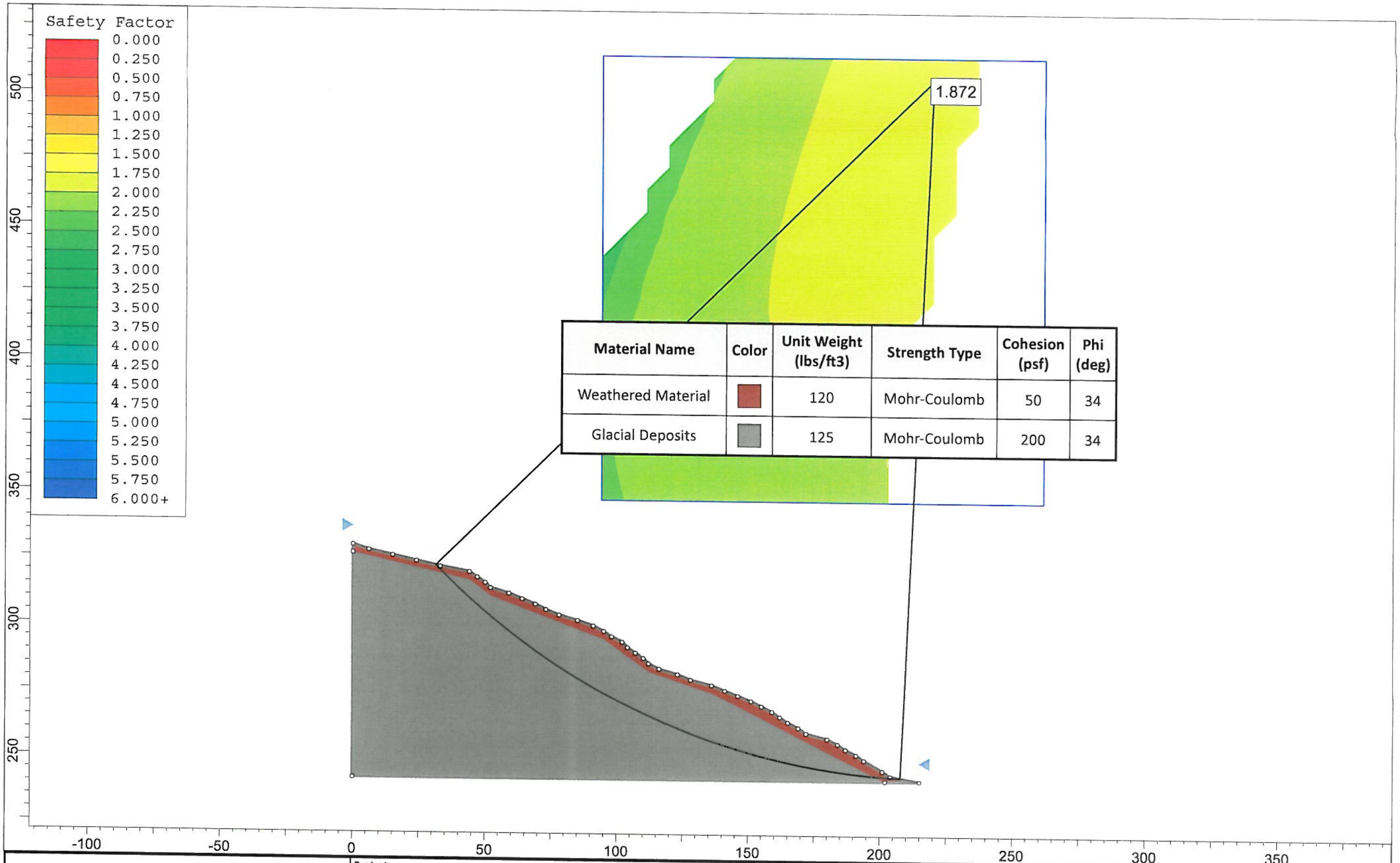


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Weathered Material		120	Mohr-Coulomb	50	34	Water Surface	Custom	1	
Glacial Deposits		125	Mohr-Coulomb	200	34	None			0
Fill		125	Mohr-Coulomb	0	34	None			0

rocscience

SLIDEINTERPRET 6.039

Project	4717 Ames Lake Carnation Road, Figure 8
Analysis Description	A-A' Developed Conditions, Seismic
Drawn By	JHA
Scale	1:300
Company	
Date	1/31/2018, 1:56:17 PM
File Name	A-A' Developed, localized fill wall, water surface, seismic.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Weathered Material		120	Mohr-Coulomb	50	34
Glacial Deposits		125	Mohr-Coulomb	200	34

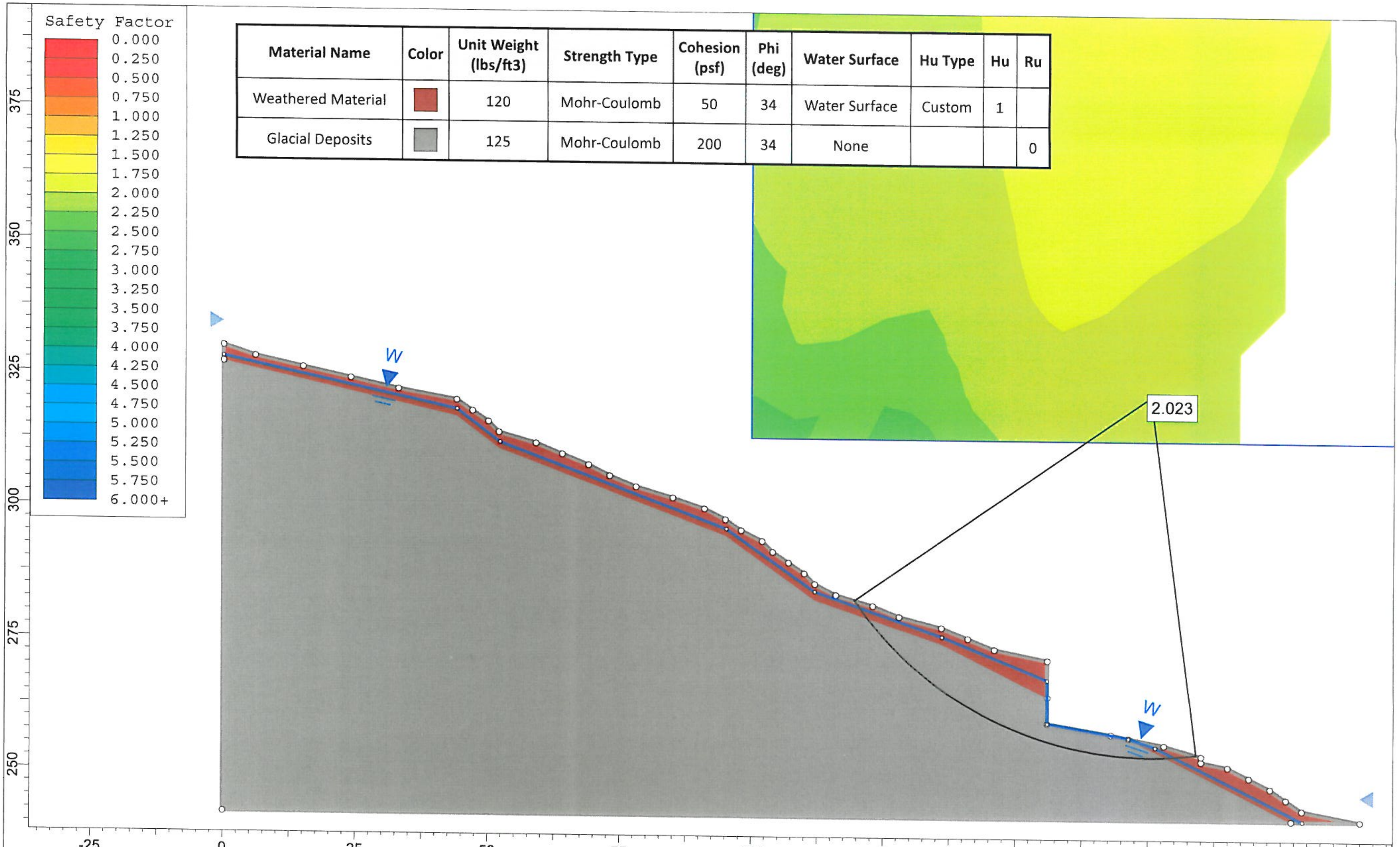
SLIDEINTERPRET 6.039

Project
4717 Ames Lake Carnation Road, Figure 9

Analysis Description
B-B' Existing Conditions

Drawn By JHA | *Scale* 1:600 | *Company*

Date 1/31/2018, 2:39:13 PM | *File Name* B-B' Existing.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Weathered Material		120	Mohr-Coulomb	50	34	Water Surface	Custom	1	
Glacial Deposits		125	Mohr-Coulomb	200	34	None			0

SLIDEINTERPRET 6.039

Project: 4717 Ames Lake Carnation Road, Figure 10

Analysis Description: B-B' Developed Conditions

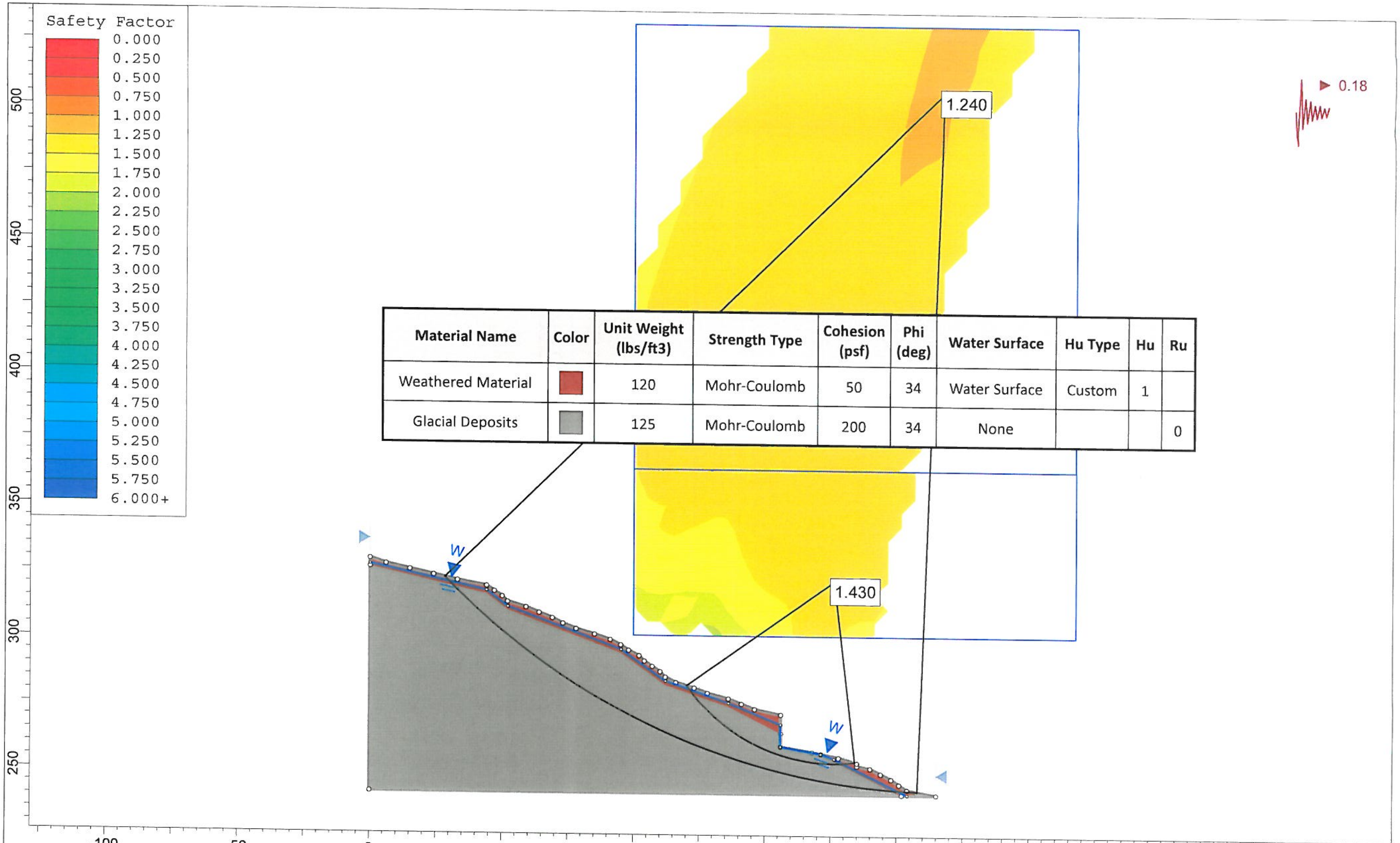
Drawn By: JHA

Scale: 1:300

Company:

Date: 1/31/2018, 2:39:13 PM

File Name: B-B' Developed, water surface.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Weathered Material		120	Mohr-Coulomb	50	34	Water Surface	Custom	1	
Glacial Deposits		125	Mohr-Coulomb	200	34	None			0

Project	4717 Ames Lake Carnation Road, Figure 11
Analysis Description	B-B' Developed Conditions, Seismic
Drawn By	JHA
Scale	1:600
Company	
Date	1/31/2018, 2:39:13 PM
File Name	B-B' Developed, water surface, seismic.slim

SLIDEINTERPRET 6.039

APPENDIX A

USGS Design Maps Summary Report

User–Specified Input

Building Code Reference Document 2012/2015 International Building Code
(which utilizes USGS hazard data available in 2008)

Site Coordinates 47.64956°N, 121.96211°W

Site Soil Classification Site Class D – "Stiff Soil"

Risk Category I/II/III

USGS–Provided Output

S_S =	1.193 g	S_{MS} =	1.220 g	S_{DS} =	0.813 g
S_1 =	0.454 g	S_{M1} =	0.702 g	S_{D1} =	0.468 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.