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Report of Supplemental Geologic Reconnaissance & Preliminary Slope Stability Analysis for Eastern Portion of OSU Cascades 46-Acre Site 1707 & 1757 SW Simpson Avenue Bend, Oregon

CGT Project Number G1303959.B

Prepared for:

OSU-Cascades
Attn: Ms. Kelly Sparks / AVP Finance & Strategic Planning
650 SW Columbia Street, Suite 7250
Bend, Oregon 97702

May 21, 2014

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Dear Ms. Sparks:

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing our supplemental geologic reconnaissance and preliminary slope stability analyses for the eastern portion of the proposed OSU Cascades 46-acre development site. This report is considered an addendum to our January 30, 2014, preliminary geotechnical report for the project. The overall project site spans two parcels located at 1707 and 1757 SW Simpson Avenue in Bend, Oregon. Our services were provided in general accordance with the scope of work detailed in a March 14, 2014, email between CGT and our client. Verbal authorization for our services was provided on March 19, 2014. Written authorization for our services was provided on March 24, 2014, by Amendment No. 1 to the December 11, 2013, "Retainer Contract Supplement, OUS Retainer Contract for Professional Consultants, Supplement No. OSU-433-P-13-90, Cascades Campus Geotechnical Engineering".

Respectfully Submitted,
CARLSON GEOTECHNICAL

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ATTACHMENTS

1.0 INTRODUCTION & BACKGROUND

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing our supplemental geologic reconnaissance and preliminary slope stability analyses for the eastern portion of the proposed OSU Cascades 46-acre development site. CGT previously performed a preliminary geotechnical investigation for the site, the results of which were presented in our Report of Preliminary Geotechnical Investigation, dated January 30, 2014. This report is considered supplemental to the referenced geotechnical report.

1.1 Project Description

Plans for development at the site are generally consistent with those described in our January 30, 2014, preliminary geotechnical report for the project. In summary, development planned at the site includes construction of several academic and dormitory buildings ranging in height from 2 to 5 stories, with appurtenant onsite roads, parking, and utility infrastructure.

1.2 Previous Work

As part of our preliminary geotechnical investigation, we observed the advancement of eighteen drilled borings and thirty-five test pits at the site. Logs of those explorations were presented in Appendix C of the referenced preliminary geotechnical report. Data collected from those explorations were used, as needed, to supplement our field observations and laboratory data obtained for this assignment.

1.3 Correspondence with Project Design Team

As indicated in the referenced geotechnical report, the eastern roughly 20 acres of the project site is occupied by a former open pit pumice mine. The pit side slopes range in height from about 30 to 90 feet, with gradients ranging from near-vertical to about 1H:1V (Horizontal:Vertical). Table 6 in the referenced report presents four options for consideration by OSU Cascades with regard to development near these slopes. Options 2 and 3 included quantitative slope stability analysis to refine slope setback requirements and re-grading (flattening or shortening) the site slopes, respectively.

Conceptual grading plans have been developed by the project design team, with three primary scenarios under consideration for master planning of the project: The “Rim”, “Canyon”, and “Terrace” scenarios. The scenarios differ primarily in the degree of infilling of the former pumice pit (from least to most fill, respectively) and the location and arrangement of the proposed buildings and roadways. We understand the Canyon scenario, which maintains slopes up to about 90 feet tall, is the preferred conceptual model at this time.

We understand the site is currently considered a permitted, active aggregate mine by the mine permitting agency, the Oregon Department of Geology and Mineral Industries (DOGAMI). The mine permit must be closed prior to redevelopment of the site. One of the requirements for closing the permit is implementation of an approved reclamation plan. DOGAMI requirements for reclamation vary depending on the post-reclamation use, and typically include grading permanent slopes to gradients of 1½H:1V (Horizontal:Vertical) or flatter, as well as establishment of vegetation and other measures to stabilize the site soils and slopes. Alternative reclamation plans are allowed by DOGAMI, provided they are stamped by a Certified Engineering Geologist (CEG) licensed to practice in the State of Oregon.

At the request of OSU Cascades, CGT prepared this report to assist in the development of a mine reclamation plan as well as post-reclamation grading and development plans, specific to the eastern portion of the mine pit. This report addresses the subject portion of the mine pit, as shown on the attached Site Plan, Figure 1.

1.4 Scope of Work

CGT's scope of work for this assignment was to evaluate various slope configurations with regard to long-term slope stability and present recommendations for maximum slope gradients of site slopes constructed exclusively in cut. Our specific scope of services included:

- Supplemental Geologic Reconnaissance: Visit the site to perform further geologic reconnaissance of the site slopes (pit sidewalls). The reconnaissance was performed by Certified Engineering Geologists (CEGs), and included identification of geologic materials, stratigraphy, and geologic discontinuities comprising the slopes, including:
  - Type (contact, joint, fault, etc.)
  - Orientation (strike and dip)
  - Spacing
  - Degree of separation, infilling, and roughness of joint/fault surfaces
  - Continuity/length (persistence)
  - Field characterization of rock strengths and degree of weathering using simplified field tests and Schmidt hammer readings
- Analysis – Geologic Structure and Strength Characteristics: Based on the results of our fieldwork to date and review of published geologic mapping and literature, characterize the structure and strength characteristics of the geologic materials as they relate to slope stability.
- Analysis – Slope Stability: Develop representative geologic profiles to evaluate with regard to slope stability. Analyze a variety of slope configurations, as needed, to obtain an acceptable factor of safety.
- Report: Provide this written report summarizing the results of our geologic reconnaissance and preliminary slope stability analyses.

2.0 GEOLOGIC SETTING

As described in the referenced geotechnical report, available geologic mapping of the area[1,2] indicates that the site and immediate vicinity are underlain by Pleistocene pyroclastic deposits. These include the Tumalo Tuff, Bend Pumice, and Desert Spring Tuff.

The Tumalo Tuff (Qtu) and Bend Pumice (Qb) are thought to represent a single eruptive sequence with an age of approximately 200,000 to 400,000 years. The deposits consist of a lower airfall tephra deposit (Qb) and overlying pyroclastic flow deposit (Qtu). These deposits are generally light gray to pinkish gray and are dominated by rhyolitic to dacitic ash and lapilli pumice with varying basalt and other lithic fragments. Welding or vapor phase crystallization has produced hardened zones within these units and is variable. A maximum thickness of about 80 feet is indicated in the literature.

1 Sherrod, David R., et al., 2004. Geologic Map of the Bend 30- X 60-Minute Quadrangle, Central Oregon. United States Geological Survey, Geologic Investigations Series Map I-2683.

2 Mimura, Koji, 1992. Reconnaissance Geologic Map of the West Half of the Bend and the East Half of the Shevlin Park 7½' quadrangles, Deschutes County, Oregon. . United States Geological Survey, Miscellaneous Field Studies Map MF-2189.

The Desert Spring Tuff (Qds) is a rhyodacitic ash flow tuff with an age of approximately 600,000 to 700,000 years. The ashy matrix ranges in color from dark gray to brownish orange, and contains dark-gray, pumiceous lapilli and basaltic lithic fragments. The lower portion of the unit is partially welded and displays columnar jointing. A thickness of about 15 to 35 feet is indicated in the literature.

3.0 RECONNAISSANCE & LABORATORY TESTING

3.1 Supplemental Geologic Reconnaissance

CGT engineering geologists Ryan Houser, CEG, and Jeff Jones, CEG, performed a supplemental geologic reconnaissance of the site between March 24 and 26, 2014. The purpose of the reconnaissance was to refine our understanding of site geology as it pertains to stability of the site slopes, and included surface observations and measurements to characterize the geologic materials comprising the site slopes and geologic discontinuities exposed in the slope faces.

Strike and dip measurements were made on joints and fractures exposed in the slope faces. These measurements were made using an analog magnetic compass with inclinometer (Brunton compass) and conventional methods.

In order to evaluate rock strength characteristics, simplified field estimations were performed in general accordance with ODOT[3], ISRM[4], and Wyllie and Mah (adapted from Hoek)[5]. The field estimation approach is based solely on field observation and provides a qualitative evaluation of rock characteristics, including a range of compressive strength and a geologic strength index (GSI) value. The field methods include striking and scratching the rocks with a geologic hammer and qualitative characterization of the degree of rock weathering, jointing, and discontinuities. These methods are somewhat subjective, but are generally considered a reliable basis for simple rock slope stability analyses.

We also performed numerous measurements of rock strength using a Schmidt hammer (Type N). The Schmidt hammer is a device to characterize the compressive strength of concrete or rock based on the rebound of a spring-loaded mass impacting the surface of the material being tested. The hammer impacts the test material at a defined energy, and its rebound is dependent on the hardness of the material being tested. The hammer body includes a mechanical indicator that shows the amount of rebound relative to an arbitrary numeric scale that ranges from 10 to 100. The rebound value for each test is recorded by the user for later use.

3.2 Material Sampling and Laboratory Testing

A CGT representative returned to the site on April 1, 2014, to collect samples of unit Qds (Desert Spring tuff) for uniaxial compressive strength testing in the laboratory. Intact blocks, ranging in size from about 1 to 2 feet in dimension, were pried from the slope face. The in-place orientation of each block was marked on the samples, which were returned to our soils laboratory. Our laboratory staff obtained core samples from the blocks using a rotary coring machine equipped with a 3-inch diameter core barrel. The bulk samples were oriented consistent with their original, in-place orientation (i.e. with up and down consistent with the in-place

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[3] Oregon Department of Transportation, 1987. Soil and Rock Classification Manual.

[4] International Society for Rock Mechanics (ISRM), 1981. Rock Characterization, Testing and Monitoring; ISRM Suggested Method. Pergamon Press, Oxford, UK.

[5] Wyllie, Duncan C. and Mah, Christopher W., 2004. Rock Slope Engineering, Civil and Mining, 4th Edition. Spon Press, New York, NY.

orientation) prior to coring. The core samples were then trimmed so that the ends were square with the sides. Each trimmed core sample was measured (diameter and height) and weighed for unit weight determination, then capped for compressive strength testing. Results of the laboratory tests are presented in Appendix A of this report.

4.0 OBSERVATION & DISCUSSION

4.1 Stratigraphy

Based on the results of our geologic reconnaissance, review of geologic mapping, and previous subsurface explorations, the site slopes are characterized by three primary stratigraphic scenarios: (1) slopes comprised entirely of ash and pumice deposits (units Qtu and Qb), (2) slopes comprised entirely of ash flow tuff (unit Qds), and (3) slopes comprised of unit Qds overlain by units Qb and/or Qtu. Outside the pit, these materials are mantled with fill and surface soils. In addition to stratigraphic differences, varying degrees of jointing and faulting were also identified. These materials are discussed individually below.

The geologic contacts observable within the pit sidewalls were exposed due to recent (roughly the past 20 years) human excavation and earthwork activities and had not developed adequate exposure (for instance, due to differential weathering, erosion, etc.) to allow for direct measurement of strike and dip. The contact between unit Qb and Qds was noticeably irregular and convoluted in places, likely reflecting uneven surface topography at the time of deposition of unit Qb. The contact between units Qtu and Qb was gently undulating, generally mimicking the underlying contact between units Qb and Qds and consistent with pyroclastic airfall or flow deposition. Locally, the contact between unit Qtu and Qb were essentially planar and gently dipping along the slope face. Photographs 1 and 2 in Figure 2 shows a representative exposure of the materials and contacts described in this report.

4.1.1 Fill and Surface Soils

The native surface soils outside the pit consist of loose to medium dense, silty sand (SM) with varying amounts of gravel. The silty sand is generally on the order of 5 to 10 feet thick, as observed in our previous explorations and along the rim of the pit. A fill berm was present along the majority of the southern rim of the pit. The berm was up to about 20 feet tall and up to about 70 feet wide. The composition of the berm is unknown, but is assumed to consist primarily of silty sand with varying amounts of rock derived from nearby surface soils. In addition to the berm, similar fill soils were apparent along the eastern portion of the north slope, just west of the paved access road leading to the site from SW Simpson Road. Anecdotal evidence suggests fill in this area is likely associated with the former landfill located immediately north of the pit.

4.1.2 Tumalo Tuff (Qtu)

The Tumalo tuff was generally extremely soft (R0)[6] to very soft (R1) rock comprised of dry to moist, light gray to orange-brown, volcanic ash with varying amounts of pumice and fragments of welded tuff. The upper portion of the Tumalo tuff was welded, forming a medium hard (R3) to hard (R4), light brown, capping unit. The capping unit was generally on the order of 5 to 10 feet thick and displayed moderately developed, near-vertical, columnar jointing. Photograph 3 on Figure 3 shows a representative exposure of unit Qtu and its capping unit.

[6] "R" values assigned in general accordance with Oregon Department of Transportation (1987) Soil and Rock Classification Manual.

4.1.3 Bend Pumice (Qb)

The Bend Pumice was generally unconsolidated, light gray to brown, subangular to angular, dry to moist, pumice lapilli with varying amounts of ash and scattered basaltic fragments. This material resembled a loose to medium dense, very weakly cemented gravel.

A layer of silty sand with varying amounts of gravel was generally present at the base of unit Qb. This layer was generally on the order of 4 to 8 feet thick. Erosion of this layer appeared to progress at a more rapid rate than the overlying pumice, resulting in small overhangs in the steepest slopes.

4.1.4 Desert Spring Tuff (Qds)

The Desert Spring Tuff was generally soft (R2) to medium hard (R3), fresh to slightly weathered, dark brown to black, ash flow tuff and contained varying amounts of pumice, lithics, and scoria. The rock mass was characterized as generally massive, with little to no significant bedding.

Fracturing and jointing was observed throughout the rock mass, and was generally characterized as closed (little to no separation), clean (little to no infilling or secondary mineralization), rough, and arcuate. Fracture/joint spacing generally varied from a few inches to several feet. Fracture persistence was difficult to discern, though continuous fractures with lengths exceeding 20 to 30 feet were observed across the rock face in places. In general, the fractures were steeply dipping to near-vertical, with strikes ranging from east-west to north-south.

Several open joints were observed within the south pit wall, near cross section J-J' (shown on Figure 1). Joint separation ranged from less than 1 inch to in excess of 1 foot, and the joints appeared to extend 5 to 10 feet (possibly more) into the slope. Adjacent rock also showed signs of intersecting joints and preferential erosion, forming rough blocks up to 10 to 20 feet or more in dimension. Several of the open joints had previously been filled with concrete. Anecdotally, we interpret this was done to help stabilize the individual blocks and reduce the risk of rockfall during operation of the pit. Photograph 4 on Figure 3 shows this area.

4.2 Surface Processes

Abundant signs of erosion and rockfall were observed during our reconnaissance. As noted previously, conditions within the pit were largely the result of recent human excavation and earthwork activities. As such, our observations of surface processes within the pit are indicative of short-term performance and not necessarily an indication of long-term performance.

On slopes where units Qtu and Qb were exposed, we observed a near-constant shedding of sand- and gravel-sized particles from the slope face. Accumulation of this material was apparent at the toe of the slopes, and ranged in thickness from a few inches to more than 20 feet. The variability in accumulation is likely the result of removal in places during previous earthmoving operations in the pit and variable exposure height. Signs of on-going raveling and shallow sloughing were also apparent on these slopes. No obvious signs of deep-seated failures were observed on these slopes.

Evidence of recent rockfall was observed near the toe of slopes in various locations around the pit. Evidence included cobbles and boulders of the Qtu welded tuff capping unit that ranged in size from less than 1 foot to more than 3 feet in dimension. Cobbles and boulders of unit Qds ranged in size from less than 1 foot to more than 4 feet in dimension. The primary areas where we observed evidence of rockfall from unit Qds was in the southern wall of the pit, where exposures were tallest and slope gradients steeper. Where slopes comprised of Qds were graded flatter than about ½H:1V (primarily in the western portion of the pit, outside the area addressed by this report), we did not observe significant evidence of recent rockfall.

5.0 SLOPE STABILITY ANALYSIS

Based our geologic reconnaissance, we identified five geologic cross sections (H-H', I-I', J-J', L-L', and N-N') considered representative of the varying conditions at the site. The locations of the cross sections are shown on the attached Site Plan, Figure 1.

We performed slope stability analyses using the software program Slope/W (version 7.23) developed by Geo-Slope International, Ltd. The results of a slope stability analysis express the relative stability of a slope as a factor of safety against sliding for a potential failure surface. A factor of safety of 1.0 corresponds to the condition in which the driving and resisting forces are equal, and failure could occur as a result of small changes in the resisting or driving forces. Based on current standard practice in this region, the minimum recommended factor of safety for long-term slope stability under static conditions is 1.5, and 1.1 for short-term stability under seismic loading. Recognizing the type of development proposed at the site, the referenced factors of safety were considered minimum allowable values for permanent slopes at the site.

Quantitative slope stability analyses require geometric properties of the slope, stratigraphy, soil and rock strength parameters, and groundwater conditions. The following sections describe the methods used in determining these properties for our analyses.

5.1 Topography and Stratigraphy

Locations of our cross sections were provided to the project civil engineering consultant, KPFF Consulting Engineers, who generated profiles of existing topography for each cross section using three dimensional modeling software (AutoCAD Civil 3D). Stratigraphy for each cross section was based on field observations and review of previous subsurface explorations. For the purposes of our models, the geologic contacts were assumed to be essentially planar and relatively level, unless data from our previous subsurface explorations or field observations indicated otherwise. As noted in Section 4.1, it should be noted that the geologic contacts are, in reality, likely not as uniform as those shown in our cross sections.

5.2 Rock Structure

5.2.1 <u>Tumalo Tuff (Qtu) and Bend Pumice (Qb)</u>

No significant jointing was observed within the non-welded Qtu or unit Qb. Some bedding was apparent in both of these units. In unit Qtu, the bedding appeared to represent changes in eruptive activity at the time of deposition. Unit Qb was similar, with the exception of the lower, silty sand layer that likely represents inter-eruptive alluvial deposition. No major variation in material properties was observed across the bed contacts and the bedding is not considered a structurally controlling feature. Accordingly, these materials were modeled as a homogeneous material.

The Qtu capping unit (welded tuff) exhibited moderately developed, near-vertical, columnar jointing, with spacing on the order of 2 to 6 feet. In-place blocks of this material ranged in size from about 5 to 10 feet in long dimension. As described later in this report, CGT recommends that this material be removed from the top of slopes at the site and therefore was not included in our models incorporating final grading conditions.

5.2.2 Desert Spring Tuff (Qds)

Based on review of the strike and dip data, the jointing observed in unit Qds is generally steeply dipping to near-vertical. No significant pattern of orthogonal or other dominant pattern of joint intersection was apparent for the majority of this unit. Accordingly, this material was generally modeled as a fractured rock mass.

The jointing observed in the area of cross section J-J' appeared to consist of two sets of orthogonal joints. As noted above, some of the joints in this area were open, with separation in excess of 1 foot, and formed rough blocks up to 10 to 20 feet in dimension. Existing slope gradients ranged from about ½H:1V to near-vertical with localized overhangs in this area. The floor of the pit in this area showed signs of relatively recent, tracked equipment operation, suggesting that earthmoving had been performed in the recent past. Any signs of rockfall were therefore lacking. Safety concerns precluded direct access to this portion of the slope. Data regarding persistence (length) of jointing, condition of the rock within the slope, etc. is not available at this time. Accordingly, this material was not included in our analyses. See Section 6.3.2 for further discussion of this material.

5.3 Soil and Rock Strength

Selection of soil and rock strength parameters for slope stability analyses was performed based on field estimation, laboratory data, review of literature pertinent to geotechnical properties of pyroclastic deposits[7,8], geologic engineering judgment, and experience with similar materials.

5.3.1 Tumalo Tuff (Qtu) and Bend Pumice (Qb)

As described in Section 4.1.2, unit Qtu generally resembled an extremely soft (R0) to very soft (R1) rock that was highly friable and reduced to silty sand under hand pressure. For the purposes of our slope stability analyses, this material was treated as medium dense, weakly cemented, silty sand.

As described in Section 4.1.3, unit Qb was generally unconsolidated, subangular to angular, pumice lapilli with varying amounts of ash and scattered basaltic fragments. For the purposes of our slope stability analyses, this material was treated as medium dense, very weakly cemented gravel. The lower silty sand layer was treated as a loose to medium dense, silty sand.

Strength parameters for these units were initially assumed based on experience and judgment. The parameters were refined by back calculation, assuming existing slopes comprised of these materials were marginally stable (factor of safety slightly greater than unity). This assumption was based on the observed raveling, observation of similar slopes in the area of the site, and previous experience with similar materials. For each primary material, we modeled an imaginary slope of similar height and gradient as the existing (observed) exposures, then varied the strength parameters to achieve a factor of safety slightly greater than

[7] Cecconi, M., Scarapazzi, M., and Viggiani, G., 2010. On the geology and the geotechnical properties of pyroclastic flow deposits of the Colli Albani, Bulletin of Engineering Geology and the Environment, May 2010, Volume 69, Issue 2, pp 185-206.
[8] Bommer, J.J., Rolo, R., Mitroulia, A., Berdousis, P., 2002. Geotechnical properties and seismic slope stability of volcanic soils (Electronic resource) (Paper no. 695), The 12th European conference on earthquake engineering, Pages:1-10.

one (unity). Final strength parameters for these units and those assumed for the fill and surface soils are presented in Table 1.

Table 1: Material Parameters Used in Slope Stability Analyses

Material	"Idealized" Material Model	Angle of Internal Friction, Φ (degrees)	Cohesion, c (psf)	Total Unit Weight, γ_T (pcf)
Berm Fill & Surface Soil	Silty sand	30	0	110
Unit Qtu	Weakly cemented silty sand	36	575	80
Unit Qb	Weakly cemented gravel	40	190	75
Unit Qb – base layer	Silty sand	35	0	85

5.3.2 Desert Spring Tuff (Qds)

As described in Section 4.1.4, the Desert Spring Tuff was generally soft (R2) to medium hard (R3), fresh to slightly weathered, fractured, partially welded, ash flow tuff. The rock mass was characterized as generally massive, with little to no significant bedding. For the purposes of our slope stability analyses, this material was generally treated as a fractured rock mass, with shear strength parameters assigned using the Generalized Hoek-Brown Criterion[9]. The Generalized Hoek-Brown Criterion is a method to determine the strength of fractured rock masses in which the shear strength is represented as a non-linear shear strength curve. Conventional limit equilibrium analyses can be carried out using equivalent Mohr-Coulomb shear strength parameters provided in this manner.

Four input parameters are required by the Generalized Hoek-Brown Criterion. The input parameters were assigned in general accordance with the methods discussed by Wyllie and Mah[10], and are presented in Table 2. Uniaxial compressive strength and unit weight were assigned based on the results of laboratory data and our field estimations. The material constant (m_i) corresponds to a typical value for tuff, as indicated in Table 4.5 of Wyllie and Mah. The Geological Strength Index (*GSI*) was assigned based on the criteria shown in Table 4.3 of Wyllie and Mah. The value in parentheses is the range of values we assigned, with the average value used in the analyses. The rock mass disturbance factor (*D*) was assigned based on Table 4.6 of Wyllie and Mah and the assumption that excavation of the slopes was accomplished primarily by mechanical excavation with minor blasting.

9 Hoek, E., Carranza-Torres, C. and Corkum, B., 2002. Hoek-Brown Failure Criterion – 2002 Edition. Proceedings of the North American Rock Mechanics Society meeting in Toronto in July 2002, in "Stability Modeling with Slope/W 2007" by Geo-Slope International.

10 Wyllie, Duncan C. and Mah, Christopher W., 2004. Rock Slope Engineering, Civil and Mining, 4th Edition. Spon Press, New York, NY.

Table 2: Input Parameters for Generalized Hoek-Brown Criterion for Unit Qds

Parameter	Value
Uniaxial compressive strength σ_{ci} (psf)	85,320
Material constant m_i	13
Geological Strength Index (*GSI*) (0-100)	75 (63-87)
Rock mass disturbance factor *D* (0-1)	0.7
NOTES: Parameters assigned in general accordance with the methods discussed by Wyllie and Mah (2004). Value of GSI represents the average of the range of values assigned and shown in parentheses.	

5.4 Groundwater

Our geologic reconnaissance was conducted during a thawing period in early spring. We did not observe any signs of past or ongoing seepage from the site slopes during our reconnaissance. As discussed in Section 5.2 of our geotechnical report, static groundwater levels at and near the site are anticipated at depths in excess of 200 feet below ground surface. Accordingly, groundwater was not modeled in our stability analyses.

5.5 Seismic Considerations

In order to evaluate the stability of the slope during a design-level earthquake, we performed pseudostatic analyses that incorporate an additional lateral force to simulate cyclic ground acceleration during an earthquake. A peak ground acceleration (PGA) of 0.16g was determined for the site in accordance with the 2010 Oregon Structural Specialty Code (OSSC), as referenced in Section 10.4 of our preliminary geotechnical report. This calculation is allowed by Oregon structural codes in the absence of a site-specific evaluation of ground response from a design-level seismic event. A seismic coefficient (k_h) equal to one-half of the ground surface PGA (0.08g) was used in the pseudostatic analyses, in accordance with standard practice.

5.6 Results

5.6.1 <u>Slope Gradient Considerations</u>

Final slope gradients incorporated into our analyses were based on a combination of factors, and were influenced by our field observations of rockfall and erosion potential. Once strength parameters were established as described in Section 5.3.1 above, we then modified (flattened) the modeled slope gradient until an acceptable factor of safety was achieved. Accordingly, our stability models incorporated a maximum slope gradient of 1H:1V for units Qtu and Qb.

As discussed in Section 4.2 above, evidence of on-going rockfall was observed along the near-vertical slopes comprised of unit Qds (primarily along the south wall of the pit). Where slopes comprised of Qds were graded to about ½H:1V or flatter (primarily in the western portion of the pit, outside the area addressed by this report), we did not observe significant evidence of recent rockfall. Accordingly, our stability models incorporated a maximum slope gradient of ½H:1V for unit Qds.

5.6.2 Global Stability

Incorporating the maximum allowable gradients for slope materials discussed in Section 5.6.1, we performed slope stability analyses along four cross sections (H-H', I-I', L-L', and N-N'). The locations of the cross sections are shown on Figure 1. The results of our analyses for the modeled conditions are presented in Table 3. Graphical outputs of each of the conditions analyzed are presented in Figures 4 through 7.

Table 3: Factors of Safety for Slope Stability

Cross Section*	Factor of Safety	
	Static Loading	Seismic Loading**
H-H'	1.5	1.2
I-I'	2.7	2.4
L-L'	1.9	1.6
N-N'	2.6	1.5

NOTES: * Models incorporated trimming slopes to maximum gradients discussed in Section 5.6.1
** For pseudostatic analyses, seismic coefficient k_h = PGA/2 = 0.08g

6.0 RECOMMENDATIONS

Based on the results of our geologic reconnaissance and stability analyses, we derived recommended final gradients for slopes comprised of the primary geologic materials observed at the site. The recommended slope gradients reflect global stability, local stability, and rockfall. These are presented below by material type.

6.1 Existing Fill and Surface Soils

As indicated in the referenced geotechnical report, undocumented fill materials were encountered within portions of the pit. We understand that these fill materials may be removed and replaced with structural fill during site development. Additionally, a fill berm was present along the southern rim of the pit. For preliminary planning purposes, the existing fill should be removed or, if left in place in landscaping areas, graded to 3H:1V or flatter. This is due primarily to variability of the fill materials.

The native, surface soils (SM) at the top of permanent slopes should be similarly graded to 3H:1V or flatter. This is due primarily to the high erosion potential of the native, silty sand.

6.2 Tumalo Tuff (Qtu) and Bend Pumice (Qb)

6.2.1 Qtu Capping Unit

The Qtu capping unit (columnar jointed, welded tuff) is prone to rockfall. This material should be removed from the crest of permanent slopes at the site. This unit was generally on the order of 5 to 10 feet thick. As a guideline, the minimum distance between the crest of the slope and face of the remaining Qtu capping unit should be a minimum of 5 feet or twice the thickness of the capping unit, whichever is greater. For instance, if the design case considers a 5-foot thick capping unit, the setback distance should be at least 10 feet.

6.2.2 Qtu and Qb

Permanent cut slopes comprised of units Qtu (below the capping unit) and Qb should be graded to 1H:1V or flatter. Where exposed, the ash and pumice deposits (units Qtu and Qb) are susceptible to mass wasting (erosion) due to water, wind, and freeze-thaw action. In the long term, erosion on slopes comprised of these materials may adversely impact their stability. We recommend that permanent erosion control/surface stabilization measures be implemented to minimize erosion and help ensure long-term slope stability. Vegetation, armoring, mechanical stabilization (e.g. anchored mesh), etc. may be considered suitable erosion control/surface stabilization measures. If slopes comprised of these materials can be graded to 3H:1V or flatter, erosion control/surface stabilization measures may not be necessary. The engineering geologist and geotechnical engineer should be contacted to review finalized plans and selected gradients for cut slopes to provide supplemental recommendations for surfacing features.

6.3 Desert Spring Tuff (Qds)

6.3.1 Fractured Rock Mass (Dominant Type)

Unit Qds is also prone to rockfall, though to a lesser degree. Permanent cut slopes comprised of unit Qds that are left exposed should be graded to ½H:1V or flatter. The permanent slope gradients recommended herein will reduce, but not eliminate, the risk of rockfall occurring. The faces of permanent slopes should be scaled to remove loose materials. Rockfall protections measures should be implemented on such slopes. Rockfall protection measures may include catchment areas, rockfall barriers, mechanical stabilization (e.g. draping or anchoring), or other suitable measures, used alone or in combination with one another. Rockfall protection requirements will depend to a large degree on the location of people and improvements in relation to the slopes, as well as the nature of materials comprising the slope and the slope height. The engineering geologist and geotechnical engineer should be contacted to review finalized grading plans to assess the need for rockfall protection.

6.3.2 Open-Jointed Rock Mass (Special Case)

As described previously, numerous open joints were observed within the south pit wall, near cross section J-J'. Joint separation ranged from less than 1 inch to in excess of 1 foot, and the joints appeared to extend 5 to 10 feet (possibly more) into the slope. Adjacent rock also showed signs of intersecting joints and preferential erosion, forming rough blocks up to 10 to 20 feet or more in dimension. As indicated in Section 5.2.2, the lack of data regarding the rock structure in this area precluded quantitative stability analysis.

For preliminary planning purposes, we recommend that cut slopes in the open-jointed Qds be graded to 1H:1V or flatter. This will remove the majority of the observable, open-jointed rock and effectively lower the center of mass of the underlying jointed blocks. It is our opinion that this recommended grading should result in a stable rock slope. However, rockfall protection measures, as discussed in Section 6.3.1, are strongly recommended on these slopes. Additional evaluation, scaling, or other remediation may be warranted, depending on conditions encountered as excavation progresses. The engineering geologist should be contacted to review slope conditions during excavation and provide specific recommendations.

7.0 CORRESPONDENCE & RECOMMENDED ADDITIONAL SERVICES

Subsequent to completion of our analyses, but prior to issuance of this report, the results of our slope stability analyses and recommendations for permanent cut slope gradients were conveyed to the project design team via email and telephone conversations in the middle of April 2014. KPFF produced a conceptual grading plan reflecting grading proposed under the referenced “Canyon” plan. That plan incorporated the recommendations for maximum slope cut gradients presented in this report, as well as recommendations for construction of fill slopes presented previously in the preliminary geotechnical report.

At the time of this report, it is our understanding site grading and reclamation plans have not been finalized. Once those plans are nearing completion, we recommend the CGT engineering geologist be contacted review the proposed construction and provide supplemental recommendations for site grading, mine reclamation considerations, slope surface stabilization, rockfall protection, and other details.

8.0 LIMITATIONS & CLOSURE

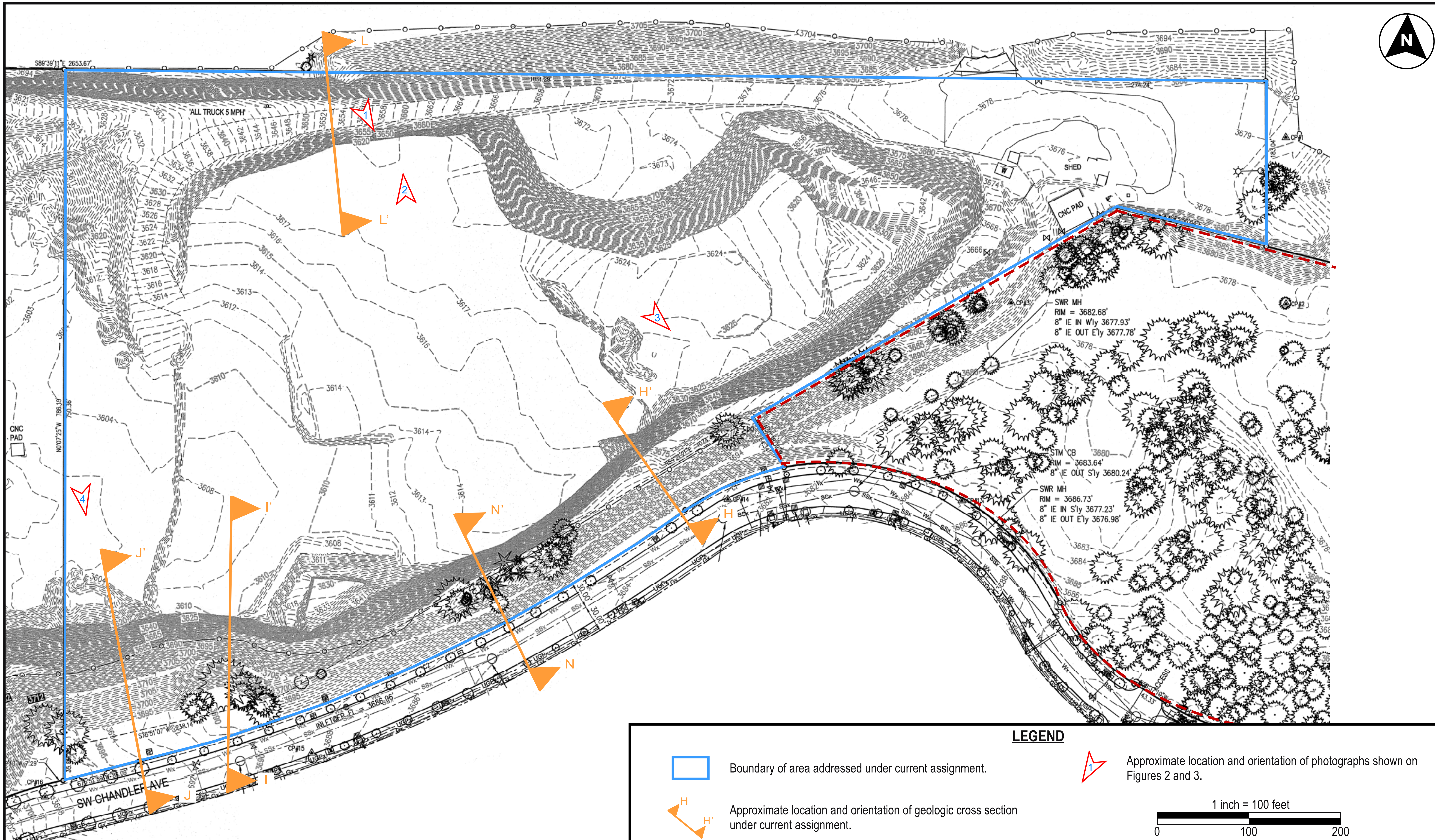
We have prepared this report for use by OSU Cascades and other members of the design and construction team for the proposed development. The opinions and recommendations contained within this report are not intended to be, nor should they be construed as, a warranty of subsurface conditions, but are forwarded to assist in the planning and design process.

We have made observations based on our explorations that indicate the soil conditions at only those specific locations and only to the depths penetrated. These observations do not necessarily reflect soil types, strata thickness, or water level variations that may exist between or away from those explorations. If subsurface conditions vary from those encountered in the site explorations, CGT should be alerted to the change in conditions so that we may provide additional recommendations, if necessary. Observation by experienced personnel should be considered an integral part of the construction process.

The owner is responsible for insuring that the project designers and contractors implement our recommendations. When the design has been finalized, prior to releasing bid packets to contractors, we recommend that the design drawings and specifications be reviewed by our firm to see that our recommendations have been interpreted and implemented as intended. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification. Design review is beyond the scope of our current assignment, but can be provided for an additional fee.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor’s methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Geotechnical engineering and the geologic sciences are characterized by a degree of uncertainty. Professional judgments presented in this report are based on our understanding of the proposed construction, familiarity with similar projects in the area, and on general experience. Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared; no warranty, expressed or implied, is made. This report is subject to review and should not be relied upon after a period of 3 years.



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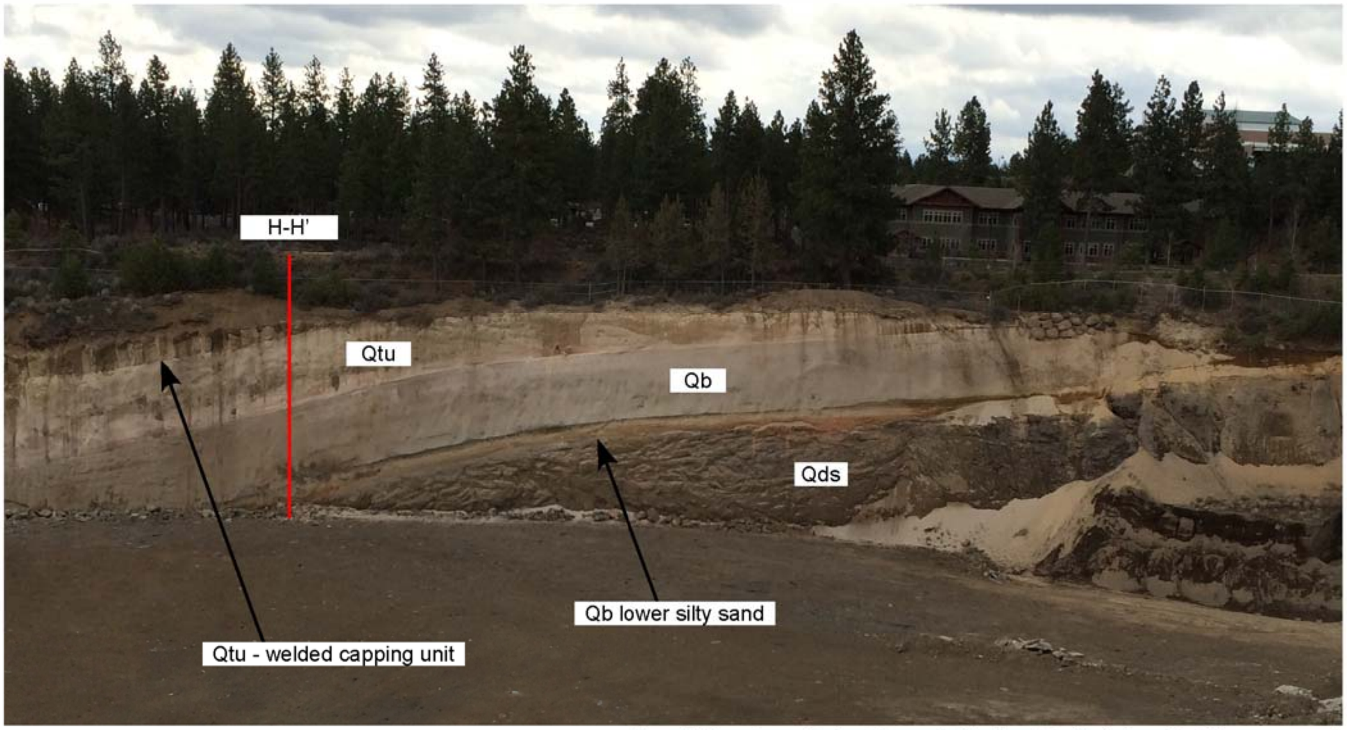
SITE PLAN SHOWING LOCATIONS OF GEOLOGIC CROSS SECTIONS

EASTERN PORTION OF OSU CASCADES 46-ACRE SITE - BEND, OREGON

CGT Job No. G1303959.B

FIGURE 1

EASTERN PORTION OF OSU CASCADES 46-ACRE SITE - BEND, OREGON
SITE PHOTOGRAPHS



Photograph 1: South pit wall. Qtu, Tumalo Tuff; Qb, Bend Pumice; Qds, Desert Spring Tuff. Line H-H' represents location of cross section H-H' for reference.

Photograph 2: Example of convoluted contact between unit Qb and Qds.

NOTE: See Figure 1 for photograph locations and orientations

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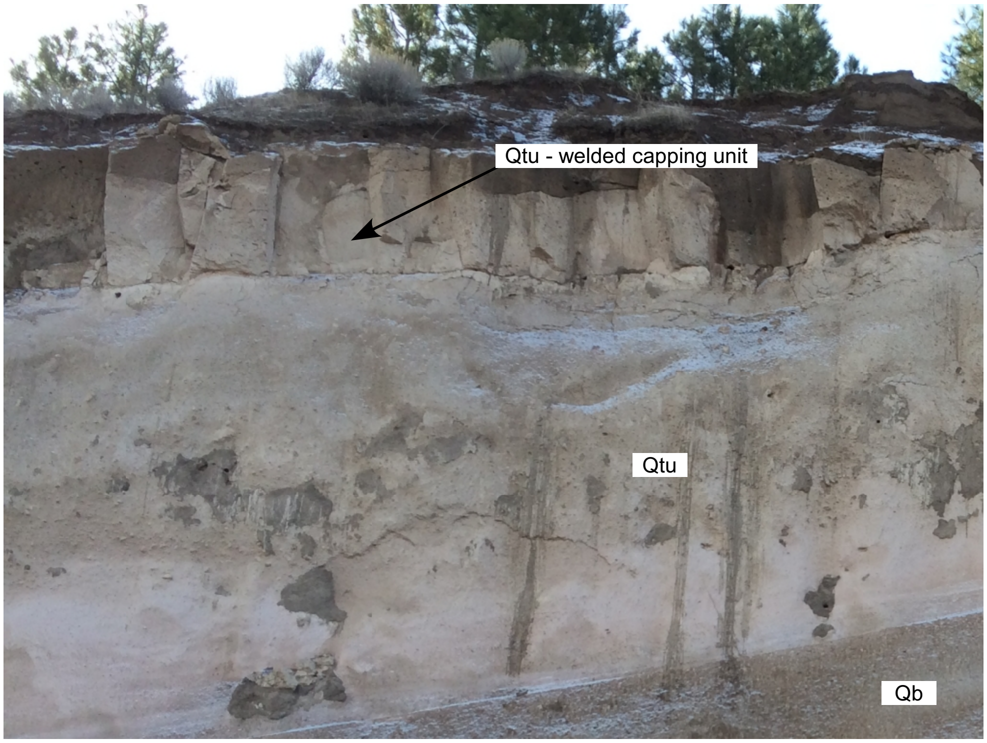
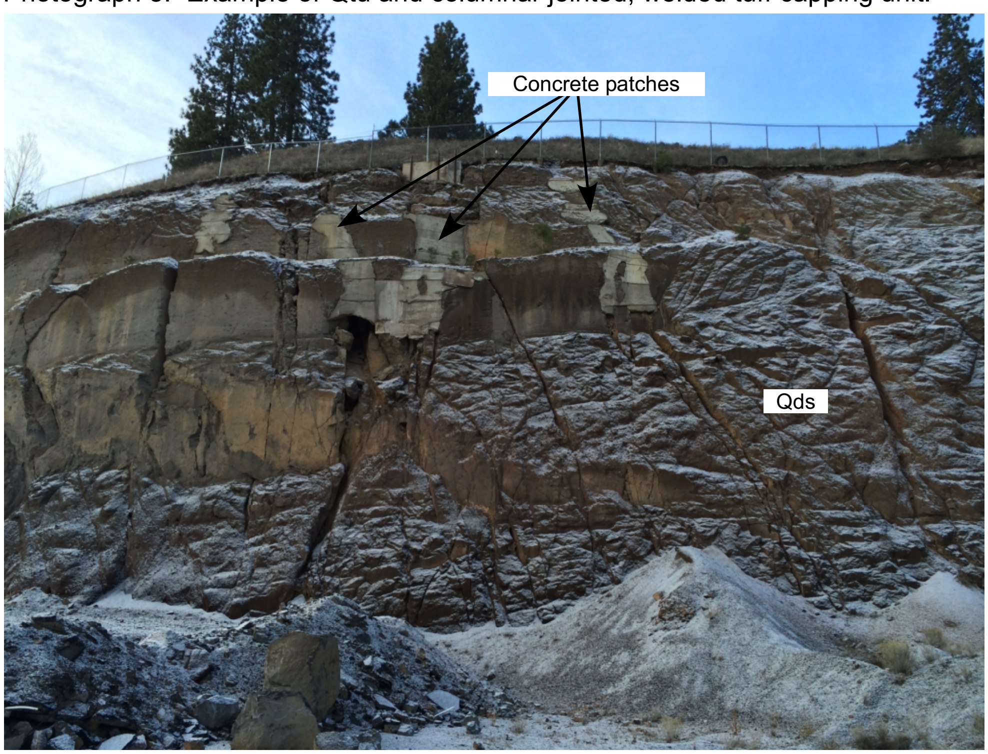
EASTERN PORTION OF OSU CASCADES 46-ACRE SITE - BEND, OREGON
SITE PHOTOGRAPHS



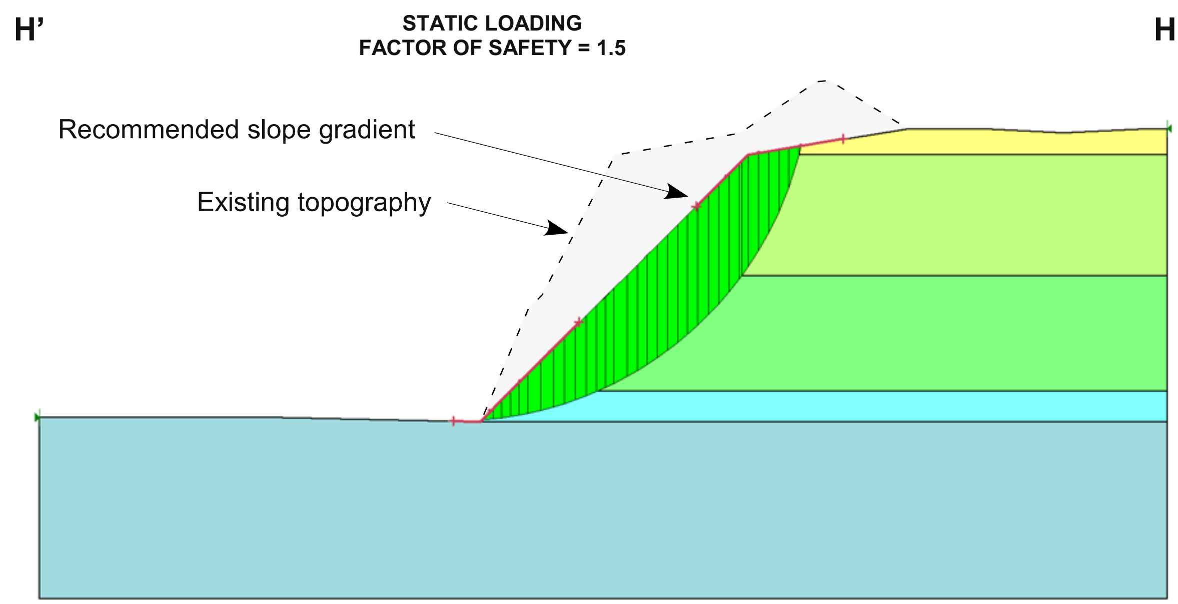
Photograph 3: Example of Qtu and columnar jointed, welded tuff capping unit.

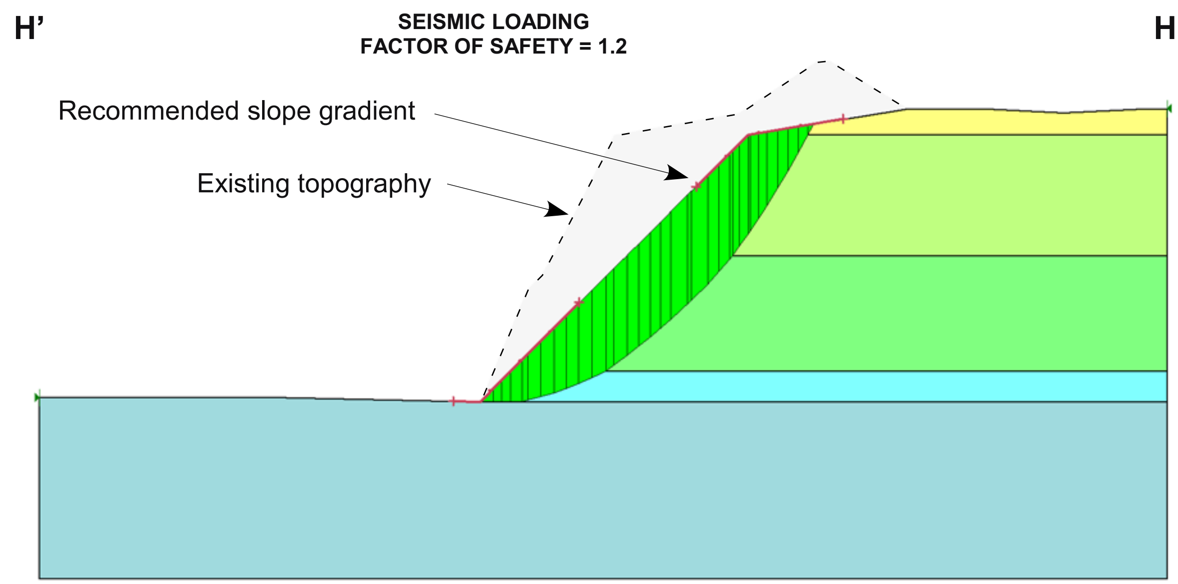


Photograph 4: Example of open joints and concrete patches in unit Qds.

NOTE: See Figure 1 for photograph locations and orientations

EASTERN PORTION OF OSU CASCADES 46-ACRE SITE - BEND, OREGON
SLOPE STABILITY PROFILE H-H'





Fill & Surface Soil	Bend Pumice (Qb)	Desert Spring Tuff (Qds)
Tumalo Tuff (Qtu)	Bend Pumice SM Base Layer	

See Figure 1 for the location and orientation of Cross Section

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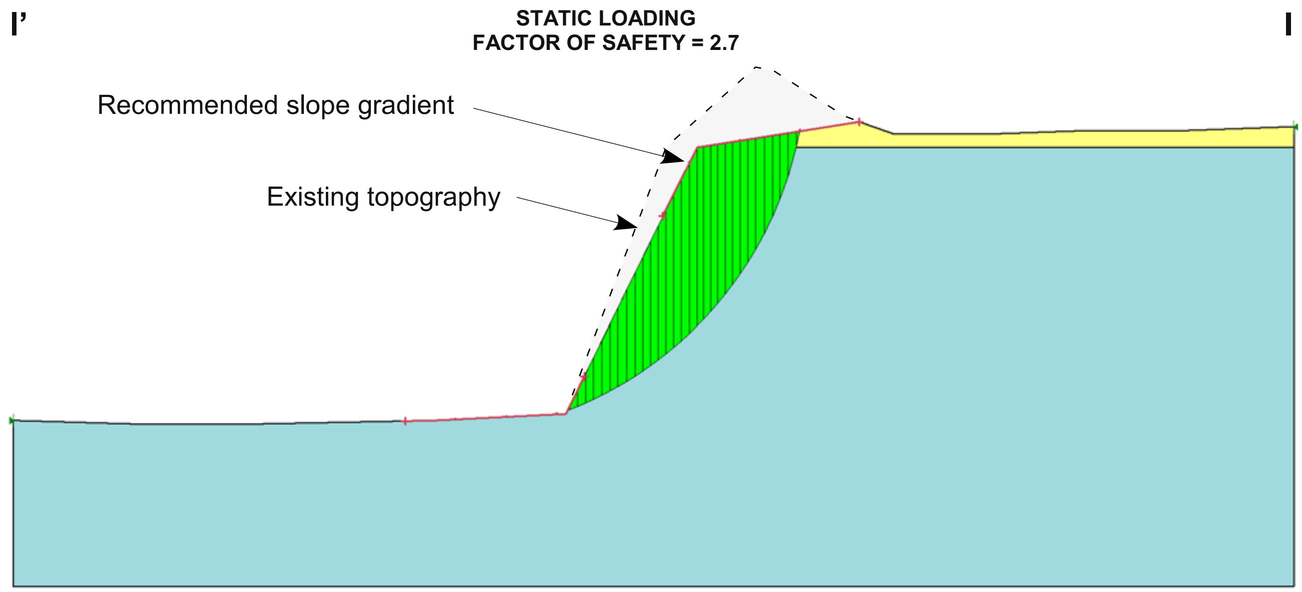
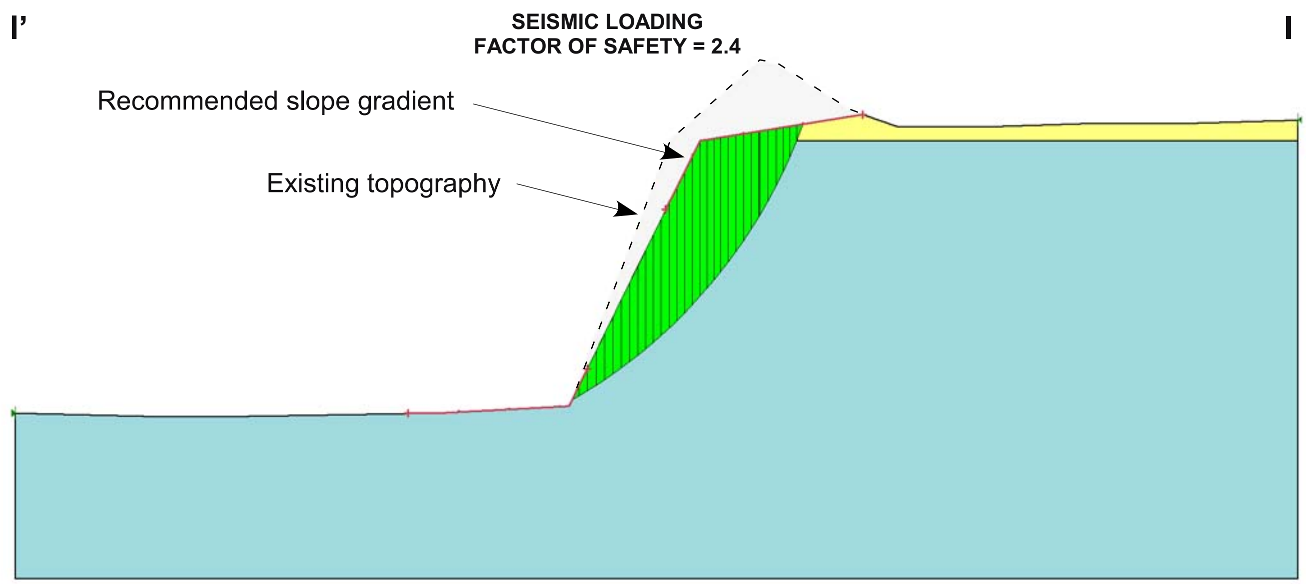
FIGURE 4

EASTERN PORTION OF OSU CASCADES 46-ACRE SITE - BEND, OREGON
SLOPE STABILITY PROFILE I-I'





Fill & Surface Soil

Desert Spring Tuff (Qds)

See Figure 1 for the location and orientation of Cross Section

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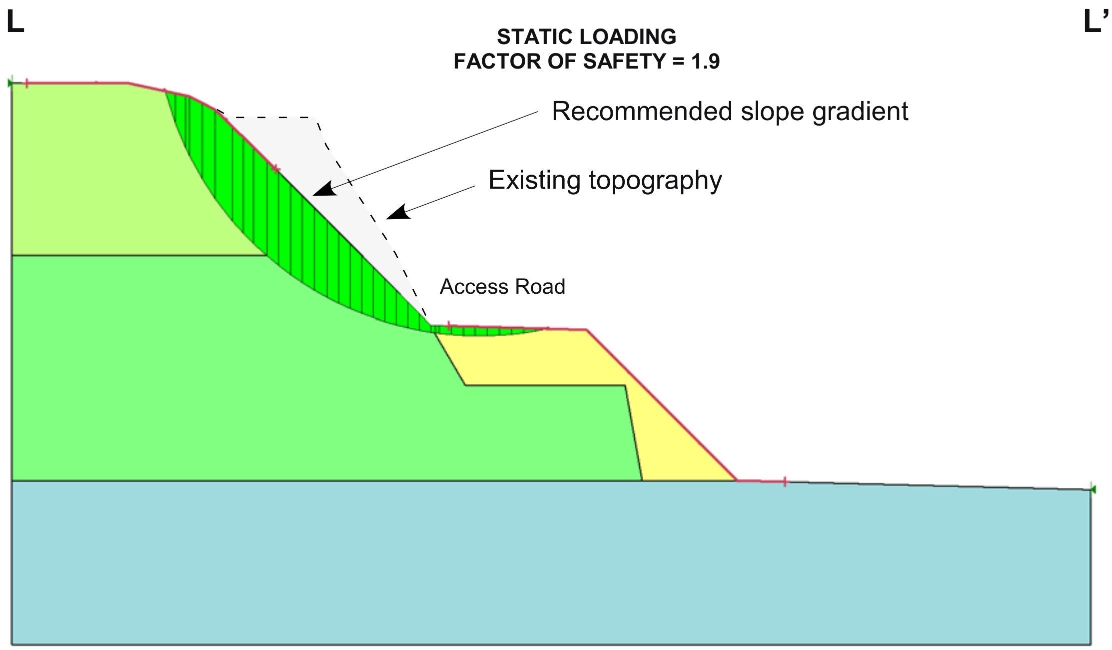
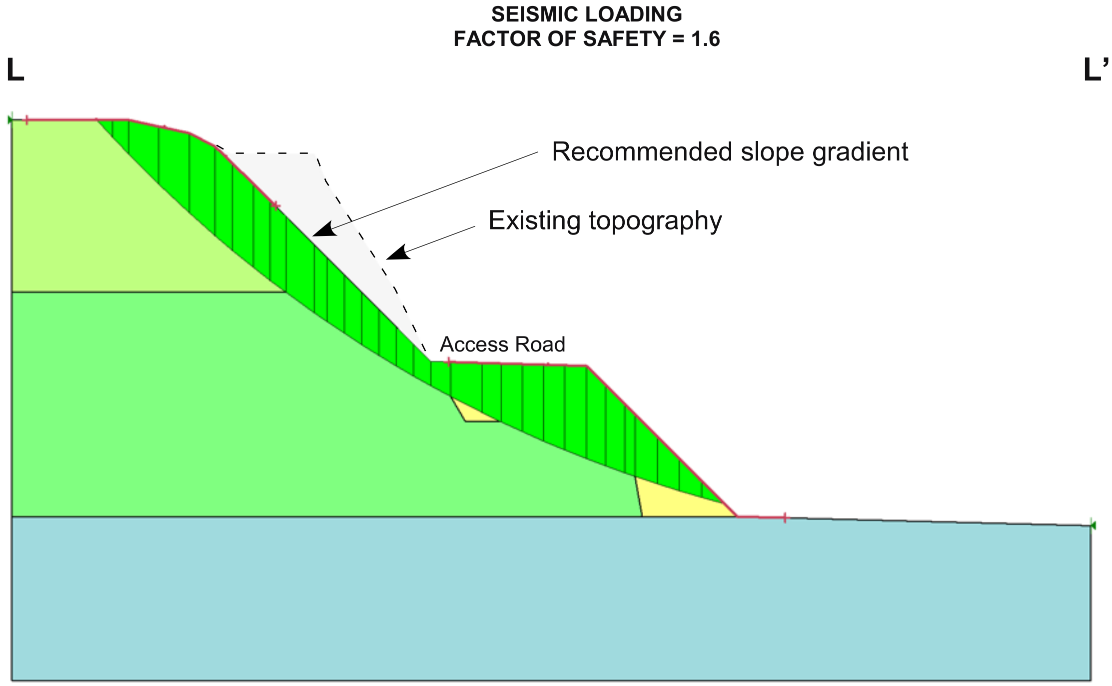
FIGURE 5

EASTERN PORTION OF OSU CASCADES 46-ACRE SITE - BEND, OREGON
SLOPE STABILITY PROFILE L-L'





Fill & Surface Soil
Bend Pumice (Qb)
Desert Spring Tuff (Qds)
Tumalo Tuff (Qtu)

See Figure 1 for the location and orientation of Cross Section

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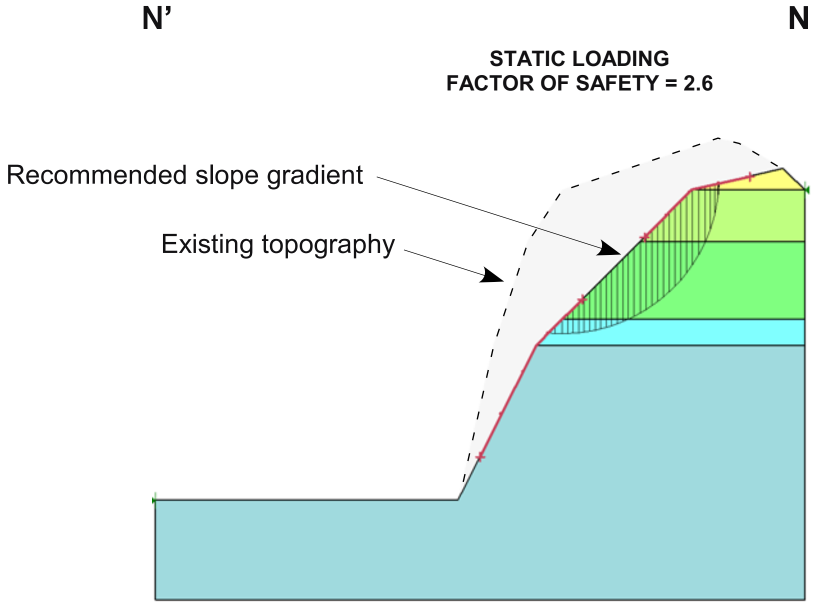
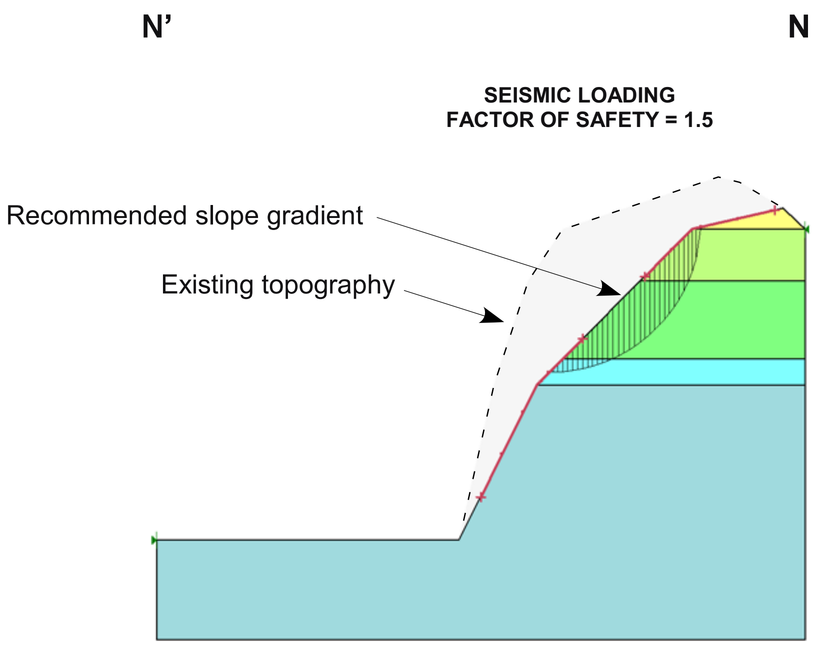
FIGURE 6

EASTERN PORTION OF OSU CASCADES 46-ACRE SITE - BEND, OREGON
SLOPE STABILITY PROFILE N-N'





Fill & Surface Soil	Bend Pumice (Qb)	Desert Spring Tuff (Qds)
Tumalo Tuff (Qtu)	Bend Pumice SM Base Layer	

See Figure 1 for the location and orientation of Cross Section

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FIGURE 7