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December 23, 2020

Mr. Josh Boisvert
Concessions Management Specialist
Mount Rainier National Park
55210 238th Avenue East
Ashford, WA 98304

RE: Geotechnical Engineering Report
Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area
Mount Rainier National Park, Washington
PMIS #2668

Dear Josh,

DJ&A, P.C. (DJ&A) and Terracon Consultants, Inc. (Terracon) are pleased to submit the enclosed Final Geotechnical Engineering Report for the Camp Muir Developed Area in Mount Rainier National Park. This project was completed in accordance with the Statement of Work described in Task Order No. 140P8520F0076 under Contract No. 140P8519D0004.

The enclosed report describes site conditions observed during the August 2020 field investigation, presents findings of geotechnical engineering analyses, and provides geotechnical recommendations for future improvements planned at Camp Muir. The final report also incorporates key points from your team's review comments.

The final report can be downloaded via the project ftp site and is saved in a folder titled "20.12.23 - Final Geotechnical Report for Camp Muir Developed Area"; instructions to access the ftp site follow.

1) Copy/paste the link below into Windows Explorer
   ftp://mail.djanda.com/Camp_Muir
2) Enter Username: CampMuir
3) Enter Password: Rainier@2020

The DJ&A and Terracon team thanks you the opportunity to assist the National Park Service and to be part of this exciting and unique project. Additionally, we thank you for your continued support and coordination throughout the planning, field investigation, and reporting phases. Please let us know if we can be of any further assistance in planning repair and improvements at Camp Muir.

Sincerely,
DJ&A

Jon Pederson, PE
Project Manager

cc: Ms. Liset Gomez, Contracting Officer, National Park Service, Interior Regions 8, 9, 10, & 12
Ms. Jara Woodard, Contract Specialist, National Park Service, Interior Regions 8, 9, 10, & 12
Mr. Matthew Hoffmann, Senior Associate/Geotechnical Engineer, Terracon Consultants, Inc.

Enclosure (electronic version of final report)

Geotechnical Engineering Services Report

Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area
Mount Rainier National Park, WA

PMIS #266786
Title I – Geotechnical Services
Contract No. 140P8519D0004; Task Order No. 140P8520F0076

December 23, 2020
Terracon Project No. C4205024

Prepared for:
DJ&A, P.C.
2000 Maple Street
Missoula, MT 59808
On behalf of
National Park Service
Interior Regions 8, 9, 10, & 12

Prepared by:
Terracon Consultants, Inc.
1392 13th Ave SW
Great Falls, MT 59404

December 23, 2020

DJ&A, P.C.
2000 Maple Street
Missoula, MT 59808

Attn: Mr. Jon Pederson, P.E.
P: (406) 721-4320 Ext. 126
E: Jonathan@djanda.com

Re: Geotechnical Engineering Services Report
Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area
Mount Rainier National Park, WA
PMIS #266876
Terracon Project No. C4205024

Dear Mr. Pederson:

We have completed the Geotechnical Engineering Services for the above referenced project. This study was performed in general accordance with Terracon Proposal No. PC205024 dated June 4, 2020. This report presents the findings from these services in accordance with the intent of the Scope of Work (SOW) provided to Terracon on May 28, 2020, and conference call discussions regarding project objectives for the Camp Muir Development Area geologic and geotechnical exploration and evaluation.

The results of the geotechnical scope of services has focused on the first phase of the geotechnical exploration; specifically, site reconnaissance, surficial exploration, structure condition review, and acquisition of limited subsurface information, along with development of geotechnical engineering recommendations addressing the needs of the project development based on the Phase 1 findings.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

Matthew D. Hoffmann, P.E.
Office Manager

Brian Evans
Senior Staff Geologist

BRIAN J. WILLIAMS
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REGISTERED
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Brian J. Williams, P.E., P.G.
Senior Geotechnical Engineer

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Note: This report was originally delivered in a web-based format. **Orange Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the ***GeoReport*** logo will bring you back to this page. For more interactive features, please view your project online at client.terracon.com.

ATTACHMENTS

EXPLORATION AND TESTING PROCEDURES
PHOTOGRAPHY LOG
SITE LOCATION AND EXPLORATION PLANS
GEOHAZARD AND DEVELOPABLE AREAS
EXPLORATION RESULTS
SUPPORTING INFORMATION

Note: Refer to each individual Attachment for a listing of contents.

Geotechnical Engineering Services Report

Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area

PMIS #266876

Mount Rainier National Park, WA

Terracon Project No. C4205024

December 23, 2020

INTRODUCTION

This report presents the results of limited subsurface exploration, geologic reconnaissance, and geotechnical engineering services performed for the Camp Muir Developed Area in Mount Rainier National Park, WA. These services were provided to achieve specific geotechnical engineering objectives and inform recommendations relative to:

- Subsurface soil and rock conditions
- Regional geologic setting
- Groundwater/Permafrost conditions
- Site preparation and earthwork
- Structural evaluation
- Seismic site classification per International Building Code (IBC) [(1)]
- Foundation design and construction
- Floor slab design and construction
- Geohazards
- Retaining wall considerations
- Conceptual geohazard mitigations
- Recommendations on developable areas

The geotechnical engineering Scope of Services for this project included a site visit to perform geologic reconnaissance and limited geotechnical subsurface explorations on August 17, 18, and 19, 2020. The site visit included the following geotechnical exploration elements:

- Geologic site reconnaissance of the developed area
- Site reconnaissance and review of existing structure performance (including shelters, retaining structures, foundations)
- Limited subsurface exploration using hand tools to excavate a total of two shallow test pits
- Use of Dynamic Cone Penetrometer (DCP) at eight locations for determination of shallow soil penetration resistance for foundation analysis
- Geophysical survey lines/arrays at four locations for acquisition of shear-wave velocities of subsurface profile
- Limited laboratory testing (additional testing in process) for determination of slope material strength properties

Maps showing locations of test pits, DCP's and geophysical lines/arrays are shown on the **Exploration Plan**. Logs of test pits, DCP data summary, and completed laboratory testing are shown as separate graphs in the **Exploration Results** section.

SITE GEOLOGY

The following description of geologic site conditions is derived from our site visit in association with the field exploration and our review of publicly available geologic reports and maps available on the United States Geologic Survey website[(2),(3),(4)], a previous 2002 geotechnical reconnaissance report performed for the National Park Service (NPS) by Pullman Group[(5)], a 2011 engineering report by Western Geotechnical Consultants, Inc.[(6)] completed for NPS, and topographic maps (provided by NPS within the West Toilet Design Set)[(7)].

Present-day Mount Rainier is the direct result of volcanic activity over the past 500,000 years. These eruptions occurred as a direct result of the subduction of the Juan de Fuca plate off the west coast of the North American Continent which was responsible for the volcanism in the region. Mount Rainier is situated upon a base of granodiorite that was emplaced just below the earth's surface approximately 14 to 18 million years ago. Evidence suggests an archaic Mount Rainier once stood at the present location, but as the western edge of North America was uplifted, the overlying volcanic rock was eroded and stripped down to the granodiorite base rock approximately 1 to 2 million years ago. Approximately 500,000 years ago, the present-day Mount Rainier began to form as a series of andesitic lava flows, pyroclastic flows, pumice eruptions, and lahars created the present edifice of the mountain. The most recent lava flow and pumice eruptions are believed to be approximately 2,200 years in age, with a more recent pyroclastic flow occurring about 1,100 years ago.

Glaciation and other geomorphological processes have had a significant role in the shaping of the present-day Mount Rainier. At present, some 29 named glaciers[(8)] occupy the cone of the mountain, and it is believed glaciers have been a present occurrence on the mountain since its formation nearly 500,000 years ago. Within the last 170,000 years, glaciers extended from the precipice of Mount Rainier to distance of up to 63 miles away during the Hayden Creek Period (170,000 to 130,000 years ago) and somewhat less impactful during the Evans Creek Period (22,000 to 15,000 years ago). Numerous more recent glacial advances also occurred in the past 12,000 years including the McNeeley Drift that deposited moraines dated at roughly 11,300 and 9,500 years ago. The most recent 10,000 years have also seen several resurgences of ice up until the mid-1800's. The actions of the glaciers carved large valleys, deposited lateral and terminal moraines, deposited glacial outwash, have broken and plucked bedrock, and have even impacted the path of lava flows over the past 500,000 years. In addition to glacial action, numerous lahars, mudflows, and debris flows helped shape the present-day Mount Rainier as we see it.

The Camp Muir site is situated on an andesite bedrock ridge that is covered with glacial drift, talus, and considerable accumulations of pumice. The surficial pumice is believed to be the residual remnant of the last pumice eruption approximately 2,200 years ago. The pumice accumulations are primarily restricted to the south side of the east-west trending ridge at the project site where it forms very loose and unconsolidated slopes. The site is bound to the north by the Cowlitz Glacier and to the south by the Muir Snow Fields, and the deposits within the vicinity are likely to be those of the Evans Creek Period along with local, recent deposits of moraine from the aforementioned glaciers. Glacial action from the Cowlitz Glacier has significantly broken and fractured the rock on the north side of the ridge. Variations in ice/snow depth year to year severely weather the rock through the abrasive action of the glacier grinding on the rock and the repetitive freeze-thaw cycling, both of which combine to create vast exposures of large, broken and loosely held together slabs of andesite. Some very rocky morainal till also covers the surface where the two icefields (Cowlitz Glacier and Muir Snow Fields) separate, leaving a very rubbly surface of andesite boulders atop the andesite bedrock ridge. The rubbly moraine and broken bedrock tend to ravel down the steep slopes, particularly in the vicinity of the Butler Shelter to the west and the newly constructed restroom to the east. Underlying the pumice deposits, permafrost is encountered at roughly 24 to 36 inches below ground surface and incorporates the pumice and moraine deposits into a frozen mass of ice.

We collected numerous photographs at the time of our field exploration program and representative photos are provided in our **Photography Log**.

PROJECT DESCRIPTION

Camp Muir is currently undergoing a multi-year redevelopment plan including removal of unsightly and condemned buildings and construction of new visitor facilities, which will serve concessions operations and park climbing rangers. This geotechnical assessment has been conducted to assist in determining appropriate locations for redevelopment and construction of new climbing concession facilities and to provide mitigation recommendations to stabilize existing structures and landscape features including the historic ranger station, retaining walls, helipad, and pathways.

During our site visit, we interviewed climbing rangers as well as Mr. Joshua Boisvert (local project representative), regarding the possibility of redevelopment planning with some consideration to the use of temporary structures as well as permanent structures. From a geotechnical standpoint, temporary structures, if found to be of merit (based on anticipated useful life and construction costs), may be considered. Based on our conversations, we understand that NPS desires the aesthetics and architectural style of the high camp structures be considered in redevelopment planning, with knowledge that the existing Client Shelter/Gombu is in stark contrast to the aesthetics of the historic shelters.

Following submittal and review by NPS of our draft geotechnical report, we participated in a call on December 1, 2020 with NPS stakeholders along with DJ&A to present the geotechnical findings, review NPS comments, and collaborate on development of the final report. Comments provided by NPS have been addressed within this report, where applicable, and the discussions held during the conference call have provided Terracon with a more in-depth understanding of NPS needs. As such, sequencing recommendations have been provided based on acceptable risk tolerances and long-term site development planning goals. Of note within the NPS comments was the potential for long-term, or cumulative impacts of glacial and perennial snow thinning overtime and its potential effects on the stability of the Camp Muir Developed Area. The focus of these comments by NPS was primarily on the impacts of climate change and the prediction of impacts on the glaciers/ice field over time. It was noted that this specialty is not within the scope of the geotechnical engineer's services or expertise. If future development of models by ice/glacier engineering experts can establish the loss of ice versus time based on climate change impacts on ice/snow cover, a geotechnical approach to the slope stability could incorporate predictions from climate/ice/glacier engineering experts.

GEOTECHNICAL OVERVIEW

The primary geotechnical considerations for the redevelopment of the Camp Muir site is providing relatively stable bearing support for existing, relocated, or new structures within the developed area that meet the needs for the NPS and climbing concessionaires to provide shelter and amenities. The current developed area includes structures that are situated on various bearing layers with varying risk for differential performance. The stability of existing structures is not at a point of critical need for immediate relocation or underpinning, but consideration has been given to future needs, with the understanding that the area will be redeveloped with shelters moved, and existing temporary toilet facilities on the east side of the camp removed in favor of more permanent toilet structures. We understand NPS desires to maintain the historic appearance for future buildings, with consideration being given to both temporary and permanent structures.

The site is located on the southeastern slope of Mount Rainier, an active volcano located in the Ring of Fire as part of the North American Cordillera. Because of the potential for complete destruction of the site, geohazards discussed within this document are exclusive of the potential for volcanic activity which is not part of the scope of these services. As such, the geohazard evaluations are focused on effects of glaciation, landsliding, rock plucking, rockfall, and potential rock toppling issues within the developed area. Geotechnical considerations are presented for support of new structures on shallow foundations, as well as conceptual mitigation alternatives for existing foundations that are currently undermined at the Historic Guide Shelter (Ranger Station).

Based on site reconnaissance and limited subsurface exploration discussed within the report, there are locations within the Camp Muir development area that are suitable for redevelopment if careful

consideration is given to siting new structures within these areas. In general, the saddle located between the Client Shelter/Gombu and the Helipad provides a suitable, stable platform to host locations of new structures where shallow foundations would bear within the permafrost layer. The Helipad and area east, toward the Historic Men's Comfort Station, is situated on a limited thickness of existing fill overlying shallow bedrock and is also suitable for redevelopment. The stability of these two areas could accommodate various concepts and configurations or structures, especially with the planned removal of the temporary toilet structures east of the Helipad.

The existing conditions and evaluations of current structures is discussed in more detail within this report, but of note the Historic Guide Shelter (Ranger Station) is situated on a portion of glacial moraine mass that is undergoing substantial glacial action resulting in undermining of the foundations. This structure does not currently appear to be in a condition of imminent foundation failure, but the undermining of the foundation will need to be addressed or relocation of the structure should be considered. Conceptual alternatives for support of existing foundations and/or relocation are provided for the Historic Guide Shelter (Ranger Station). Evaluation of the remaining structures, which are in less precarious conditions, are also addressed in this report.

A preliminary geohazard and developable areas map that depicts areas that are suitable for redevelopment, areas that should be avoided, locations of existing structures, and geohazards such as rockfall and slope instabilities is presented in the attachments.

The **General Comments** section provides an understanding of the report limitations.

GEOTECHNICAL CHARACTERIZATION

We have developed a general characterization of the subsurface conditions based upon our review of the geologic setting, our limited subsurface exploration, our understanding of the project, available laboratory data, and analysis of geophysical field data gathered during our field studies. Discussion of the use of DCP and geophysical field data is presented below, with the reduced data shown as separate graphs in the **Exploration Results** section. This characterization, termed GeoModel, forms the basis of our geotechnical evaluation of site development, slope stability, site preparation, and foundation options. As part of our analyses, we identified the following model layers within the subsurface profile.

Model Layer	Layer Name	General Description
1	**Fill**	Fill soils were encountered throughout the site, located beneath or adjacent to the primary structures (Historic Public Shelter, Helipad, Historic Guide Shelter (Ranger Station), East Toilet, and Historic Men's Comfort Station) and at retaining walls to access the Ranger Station, Butler Shelter and Public Shelter area. The fill appears to have been constructed of existing site materials, inclusive of

Model Layer	Layer Name	General Description
		pumice, sand, and boulders/cobbles to level areas for the shelters, and to provide backfill within the retaining walls constructed for the level areas. The fill is likely relatively uncontrolled in nature; that is, the fill was placed with limited compactive effort due to the location and resources available at the time.
2	**Talus**	Loose talus materials were encountered on the south-facing slope of the ridge west of the Client Shelter/Gombu and Butler Shelter. The material was very loose to loose on the surface, with larger talus particles on the order of 18 to 36 inches in maximum particle size. In general, the larger talus overlaid a loose pumice sand material over permafrost (encountered in TP-01 at a depth of 36 inches). Localized slope instability within the talus was noted west of the Butler Shelter. Talus was also noted to the east of the Historic Public Shelter and New Toilet Building, with more slab rock predominating the talus slope.
3	**Pumice**	The saddle located between the Ranger Station and the Helipad and the south side slope of the east-west ridge is generally comprised of pumice within the upper two to three feet. The pumice within the saddle (as observed in TP-02) was of generally uniform size, ¾ to 1 inch diameter, and was very loose to medium dense in consistency based on Dynamic Cone Penetration (DCP) correlations to Standard Penetration Test (SPT) "N" values. The pumice sand/gravel encountered on the south slope was generally very loose to loose, coarse-grained, and appeared unconsolidated overlying a permafrost layer at depths on the order of 24 to 36 inches below existing grade.
4	**Glacial Moraine**	Very rocky morainal till covers the surface where the two icefields (Cowlitz Glacier and Muir Snow Fields) separate, leaving a very rubbly surface of andesite boulders atop the andesite bedrock ridge. The morainal till includes very limited gravel, sand, or silt/clay matrix. The rubbly moraine and broken bedrock tend to ravel down the steep slopes, creating talus slopes, particularly in the vicinity of the Butler Shelter to the west and the newly constructed restroom to the east.
5	**Andesite**	Andesite bedrock was the predominant rock encountered along the east-west ridge line, with prominent exposures north and west of the Historic Guide Shelter (Ranger Station) that have been rubbelized/abraded by glacial action. Andesite was also prevalent on the east portion of the site composing the large resistant outcrop that stands east of the Historic Public Shelter. Based on the shear wave data collected, there appears to be two distinct flow profiles within the upper 100-foot profile. The first flow is

Model Layer	Layer Name	General Description
		encountered approximately 20 feet below existing grade at the saddle and Helipad areas, which is highly fractured and increases in density with depth. A second flow is encountered around 50 feet below existing grade, which is less fractured (based on shear wave data) and of higher density.
6	**Breccia**	A zone of volcanic breccia was mapped to the immediate west of the Historic Men's Comfort Station, in which moderately hard, weathered, fine-grained matrix with large angular andesite/basalt cobble and boulder-sized fragments were observed.

As noted, we performed limited subsurface exploration inclusive of two test pits. The test pits were excavated by hand shoveling and limited picking of surficial soils to shallow depths, approximately two to three feet below existing grades. The test pit excavations were logged by our Geotechnical Engineer based on visual classification using the Unified Soil Classification System (USCS) with observations summarized in the table below as well as individual logs in the **Exploration Results** section:

Test Pit ID	Location	Total Depth (inches)	Material Description
TP-01	South Slope, approximately 30' S & 13' E of SE Corner of Gombu Shelter	38	0" – Large gravel and cobbles up to 18 inches at the surface. 2" – surface poorly-graded sand with gravel, dark brown with some reddish brown/black, very loose to loose, moist (SP) 24" – poorly-graded gravel with sand, brown/black, medium dense to dense, moist to very moist (GP) 36" – permafrost, frozen layer of sand with gravel matrix (SP)
TP-02	Middle of Saddle, approximately 70' E & 20' N	26	0" – pumice, light brown/tan, very loose to

Test Pit ID	Location	Total Depth (inches)	Material Description
	of NE Corner of Gombu Shelter		loose, uniform pumice particle size between ¾ inch and 1 inch observed throughout layer (GP) 24" – permafrost, frozen layer of pumice with sand and limited gravel matrix (GP)

A total of eight Dynamic Cone Penetration (DCP) tests were conducted as part of the field exploration efforts. The DCP tests were conducted in general accordance with the Sower's Cone Method, developed by George F. Sowers and Charles S. Hedges and referenced in Dynamic Cone for Shallow In-Situ Penetration Testing, Vane Shear and Cone Penetration Resistance Testing of In-Situ Soils[9]. In this method, a 15-lb steel mass is dropped from a height of 20 inches where it strikes an anvil to drive a 1.5-inch diameter 45-degree cone into the in-situ soils. The cone is seated into the in-situ soils, approximately 2 inches, and then maintaining a plumb position with the rod assembly the mass is lifted and dropped on the anvil in a free fall. The number of blows to advance the cone in 1-¾-inch depth increments is recorded, and each subsequent 1-¾-inch interval is recorded as desired. The recorded blows to advance the DCP cone for the 1-¾-inch depth interval can be correlated to traditional Standard Penetration Test (SPT) measurements to determine an N-value as if a conventional drill rig and automatic hammer was utilized. The SPT is conducted using an exploration drill rig in a similar fashion where a standard 2-inch outer diameter split-barrel sampling spoon is driven into the ground by a 140-pound automatic hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the Standard Penetration Test "N" resistance value (SPT N-value).

The Sower's cone DCP is utilized in locations where a drill rig and automatic hammer are unable to access an exploration location, such as Camp Muir, to correlate similar blow count measurements to SPT N-values. The SPT N-value is generally utilized to determine the relative density of a soil deposit for use in analyzing the soil for bearing capacity and settlement response under loading. The correlations utilized in our data reduction have been modified from those developed by Sowers and Hedges to account for the lack of available DCP to SPT N-value direct correlations in the geologic setting. We have used two of the correlation curves developed by the authors; a curve for virgin piedmont soils and another for 95 percent compacted soils, and averaged the two to develop correlations for this site. The raw reading, correlation based on the above reference, and relative density of the soil is presented in the **Exploration Results** section for each DCP point performed.

Exploration Point ID	Location	Total Depth (inches)	Average Recorded DCP Value	Average Correlated SPT "N" Value	Remarks
DCP-01	On slope, adjacent TP-01	26	10	9	Refusal in permafrost
DCP-02	On slope at trail access to Camp Muir	63	9	8	Driven to full depth, no refusal (pumice)
DCP-03	On saddle, adjacent TP-02	23	8	7	Refusal in permafrost
DCP-04	Near NE corner of Gombu Shelter	19	10	9	Refusal in permafrost
DCP-05	Below Helipad	19	8	7	Refusal in permafrost
DCP-06	Within west lower retaining wall fill	14	10	8	Refusal in three attempts, all within upper 14 inches
DCP-07	Helipad fill, 6 feet east of stairs	51	16	13	Refusal blow counts, likely cobble/boulder
DCP-08	South of Historic Men's Comfort Station	49	24	18	Medium dense to dense fill zone

Terracon performed refraction microtremor (ReMi) shear-wave profiling and multi-channel analysis of shear wave (MASW) measurements at the site. A total of four seismic survey lines were conducted, as shown on the **Exploration Plan**, throughout the site to develop depth profiles of the measured shear wave velocity (V_S) values. The development of the depth of measured shear wave values was used to assist in delineating the subsurface profile as described above, at depths unobservable by test pit or DCP exploration. The acquired data was reduced and Seismic Line Profiles are attached in the **Exploration Results** section. The shear wave data was reviewed and compared with previously acquired Terracon shear wave profiles for similar velocities to correlate relative density of the subsurface conditions with assumed soil/rock materials.

Based on our findings, the predominant soil profile utilized in our analysis of slope stability and foundation support for new buildings consists of the following generalized layers. All layers are generally considered to be pumice, morainal till, or talus materials overlying bedrock. The upper two to three foot zone is generally very loose to loose sand with gravel (as appears on the south slope below the Client Shelter/Gombu, or pumice over the saddle), with average shear wave velocities on the order of 300 to 550 feet per second (fps). This layer is underlain by an

approximately two to five foot thick zone of medium dense gravel with sand (also includes the permafrost materials based on test pit observations) with shear wave velocities ranging from 600 to 900 fps. Below this layer is another loose to medium dense layer of sand or gravel three to five feet thick with shear wave velocities on the order of 400 fps. Below this loose layer, the density begins to increase at approximately 12 to 16 feet below existing grade within a likely gravel layer prior to what appears to be a highly fractured bedrock layer based on shear wave velocities at approximately 20 to 24 feet below existing grade. The bedrock velocities near the interface are in the range of 1,300 to 1,500 fps and increase to approximately 4,000 to 5,000 fps at depths greater than 50 feet. This generalized profile was used in the development of the slope stability analysis and conceptual micropile foundational alternatives, discuss in the following sections.

EXISTING STRUCTURE EVALUATION

A brief description of each structure observed during the site visit is provided below. Photos of the structure and conditions discussed are provided in the **Photography Log**.

East Toilet

The East Toilet structure is located on the extreme east of the existing developed area. The structure was constructed in the 2014-2015 season using shallow foundations bearing within the fill zone overlying the andesite bedrock in this area of the site. The structure is relatively new, along with the West Toilet (the newest structure at the site), and is constructed with reinforced concrete shallow foundations (assumed as no design or as-built drawings were reviewed), concrete slab-on-grade for the split level (upper level toilets, lower level depository for waste), and a composite concrete/steel pan deck roof structure. A rock façade has been constructed as a veneer around the exterior of the building. No signs of differential movement or adverse performance were observed at this structure during the site visit.

Historic Public Shelter

The Historic Public Shelter is located on the east side of the Camp Muir Developed Area and was erected in memory of John Muir in 1921. The structure is constructed of rock with concrete slab-on-grade and an isolated concrete foundation/stem wall on the north side of the structure. The interior rock has been plastered and exterior mortar joints for the rock have been recently pointed (within the last five years). This structure shows some signs of structural distress at the corners, with shear cracks developing at the rock/mortar joints apparent at all four corners and varying in separation/width. There are also distress cracks propagating from the door and window lintels observed from the interior of the structure. This type of crack is not atypical for rock/brick and mortar construction in which wall movement differs from the middle to edge of walls and results in shear cracking. This result is due to the relatively brittle nature of the façade material, and is typically repaired with pointing of mortar joints to reseal were necessary. The floor slab on the

interior exhibited limited cracking or sloping, and the structure appears to be in good condition for its age and environment.

Historic Men's Comfort Station

The Historic Men's Comfort Station was constructed in or around 1916 of rock with concrete slab-on-grade and currently has a concrete roof. The structure is currently being utilized for storage, and has a limited footprint. The structure appears to have been constructed on a minimum thickness of fill overlying bedrock in the area, with the breccia intrusion located directly north of this structure. The structure shows no signs of distress or differential movement, and the mortar/rock joints are tight with some signs of more recent pointing efforts (within last five years).

Helipad

The helipad is located just east of the center of the ridge saddle at the Camp Muir Developed site. The dry-stack, or rockery retaining wall, constructed to level the area for the helipad has a maximum height of approximately seven feet on the south side, where it intercepts a relatively level walking path perpendicular to the south slope. On the north side, the maximum wall height is on the order of five feet where it intercepts a relatively steep glacial moraine mass above the Cowlitz Glacier. The retaining wall is performing well as currently constructed, and appears to have been constructed utilizing locally available bedrock slabs/boulders for the wall face and backfilled within the approximate 25-foot diameter using available sand/gravel/cobble matrix materials. The wall has a minimal batter, less than 1H:10V (horizontal:vertical) as currently constructed; however, it shows no obvious signs of tilt or deformation that would indicate distress or failure of the as-built system. A Dynamic Cone Penetration (DCP) test was conducted on the west end of the helipad approximately 6 feet east of the access stairs, in which the average correlated SPT N-value of 13 was determined within the upper 4.3 feet of the helipad subsurface. This correlates to a medium dense fill placed within the retained area, consistent with a modest compactive effort. The DCP probe was driven to approximately 4.3 feet below top of pad before encountering refusal blows (>50 blows per 1-¾-inch increment). It is likely that, at that depth, the backfill material was generally cobble size or larger, as it would be in the lowest reaches of the retained mass and near the glacial moraine material elevation on the north edge of the helipad. A geophysical line was also conducted that traversed over the helipad (Line 4) toward the east, in which the shear wave velocity in the upper five feet was found to be consistent with a medium dense material (approximately 1,250 fps).

Retaining Walls South of Historic Guide Shelter (Ranger Station)

On the west side of the developed area, three short dry-stack or rockery retaining walls have been constructed below the Historic Guide Shelter (Ranger Station). The lower retaining wall is approximately six feet tall at maximum height near the radius at the east end of the wall, and tapers to approximately two feet tall near the west toilets. The wall appears to have been

constructed utilizing locally available bedrock slabs/boulders for the wall face and backfilled using available sand/gravel/cobble matrix materials. The lower retaining wall has experienced a localized failure near the radius on the east side of the wall, in which the lower level rocks forming the base of the wall appear to have pushed out. This has resulted in a mobilization of the backfill material, increasing lateral pressures on the wall until a full wall failure occurred. The wall failure has not been repaired, but the rock appears to have been stacked to limit potential for additional failure or loss of backfill materials.

The second level wall is set back to the north from the top of the lower retaining structure by approximately eight to 10 feet, and is approximately six feet tall at maximum height. The wall has also been constructed using locally available bedrock slabs/boulders for the wall face and backfilled using available sand/gravel/cobble matrix materials. The second level wall face is also exhibiting distress near the radius on the east end of the wall, near maximum height, with the backfill material having settled and created a void space. The wall facing rock has a slight bulge and appears to be failing in the same vein as the lower wall section. A DCP test was conducted on the backfill for this level of the retaining wall near the radius of the wall. The DCP testing indicated a correlated average SPT N-value of 8 within the upper one foot of the backfill before refusal was encountered. We attempted three DCP tests in this backfill zone with each resulting in refusal near the one-foot depth. The resistance results correlate to loose fill prior to refusal within the retained area, consistent with limited compactive effort. It is likely backfill material is generally cobble size or larger, with non-uniform particle size.

The upper level wall is located immediately adjacent the Historic Guide Shelter (Ranger Station), and is approximately four to five feet tall at maximum height. This wall, too, appears to have been constructed using locally available bedrock slabs/boulders for the wall face and backfilled using available sand/gravel/cobble matrix materials. The wall facing and backfill at the upper wall appear to be performing well, with no visible indications of the failure mechanism encountered in the lower two levels.

All three dry-stack, rockery walls appear to have been constructed without consideration of proper base/foundation level rock sizing, incorporation of drainage, and the lower two walls have no apparent batter. That is, typical rockery wall construction includes selection of specifically sized base/foundation level rocks to provide for stable starting points for construction and allowance for battering of the wall face as constructed where the mass of the wall face will remain over the base rocks. It is recommended that reconstruction of the lower two levels of retaining walls be conducted in accordance with rockery design and construction guidelines provided within the document Rockery Design and Construction Guidelines, FHWA-CFL/TD-06-006, November 2006. Further support of this design can be provided following NPS determination of material sources, as the design is heavily dependent on material size and wall height.

Historic Guide Shelter (Ranger Station)

The Historic Guide Shelter (Ranger Station), constructed in the 1920's, is the primary structure of concern for the redevelopment plan as discussed with NPS. The Ranger Station is located on the west end of the developed area, situated on a precipice overlooking the Cowlitz Glacier to the north. The glacial action from the Cowlitz Glacier has significantly broken and fractured the rock on the north side of the ridge immediately northwest of the Ranger Station. Variations in ice/snow depth year to year, which have been reported to vary as much as 30 to 50 feet, severely weather the rock through abrasive action of the glacier grinding on the rock and the repetitive freezing and thawing, which create vast exposures of large, broken and loosely held together slabs of andesite. The Ranger Station is situated immediately east of a very broken exposure of andesite slabs on a very rocky morainal till which covers the surface where the two icefields (Cowlitz Glacier and Muir Snow Fields) separate. The rubbly moraine and broken bedrock are continually being attacked/abraded by the Cowlitz Glacier acting from the north to undermine the existing north footing line of the Ranger Station.

There are two distinct concrete footing elements along the north footing line of the Ranger Station, with the most recent, and lower in elevation, placed approximately 10 years ago in an attempt to provide additional support to the structure. The older and upper concrete foundation was apparently placed in the 1970's, and was subsequently undermined. The recently placed concrete at the lower elevation is ineffective in providing support as a majority of the foundation is currently undermined and in air. It appears the weight of the concrete foundation element may be pulling on the Ranger Station rock wall and creating additional distress. As a first move, consideration should be given to removing the latest concrete placed to relieve the structure from the added "dead" weight pulling on the rock. Separation of the upper level of concrete from the wall was also observed, with gaps between the concrete and rock/mortar line on the order of $^1/_8$ to ¼ inches. Cracking of the upper (older generation) footing and rock walls of the Ranger Station were observed on the north wall, with separation of the north jamb of the entrance door from the door frame observed as well. The concrete roof was observed to slope toward the south to drain, and appeared to be in good condition with no cracking or adverse sloping near the north wall. The slab-on-grade was found to be level with minimal cracking observed, all within typical performance expectations for a structure of its age. The superstructure appears to be in good condition in general, with the structure distress localized to the north wall/footing line and near the entrance door on the northeast portion of the building. These distress patterns are consistent with attritional loss of foundation support and differential movement. The structure does not appear to be in impending failure based on the condition of the superstructure and primary foundation elements (again the attempted concrete support at lower elevation appears to be having detrimental effects), but underpinning of the north foundation line or relocation of the structure should be considered if extension of the life of this approximately 100 year-old structure is desired.

Client Shelter/Gombu

The Client Shelter/Gombu is located on the southwest portion of the developed area, immediately south of the Ranger Station. The structure is constructed of wood studs and plywood sheathing, supported by 4x4 timbers bearing on isolated concrete pad footings with cable anchors placed over the top of the structure and embedded below exiting grade near the lower retaining wall (reportedly within permafrost). The pad footings appear to be sitting directly on the unconsolidated talus/pumice slope materials and a large portion of the structure is exposed from beneath. This shelter is utilized by the climbing concessionaires during trips with customers from Paradise to the Summit of Mt. Rainier. It is our understanding that the temporary structure is to be replaced, and relocated. As this structure is not permanent and is programmed for replacement, evaluation of the existing structure was limited to the observations noted above. Construction of a new climbing concessionaire shelter, or relocation of the existing shelter, are considerations provided in the foundation discussion.

West Toilet

The West Toilet was constructed in the 2018-2019 season, with the exception of completion of façade rock materials. The toilet construction, as shown in the design drawings provided for our review by NPS, includes the foundation of the structure steel (Hollow Structural Section beam/column) on isolated reinforced concrete spread footings, or piers. The foundation elements consist of prefabricated BF-36 Forms (high density polyethylene plastic forms) 36 inches in diameter at bearing elevation, tapering to an 18-inch diameter pier (Sonotube) extending to the structural steel connection point. The foundations bear in the permafrost layer at approximately 42 inches below existing grade. It was reported that, during excavation of the foundations, a jackhammer was required to excavate to bearing level within the permafrost layer. The West Toilet is relatively new without the full rock façade constructed, therefore no distress was observed. This foundation alternative appears to provide a reasonable and reliable method for new construction, and has been proven to be able to be constructable with the resources that can be called on for use at the Camp Muir Developed Area.

Butler Shelter

The existing A-frame structure has been impacted by past rockfall, with rocks between two and three feet in diameter reportedly having struck the upslope side of the building. As such, the Butler Shelter has been condemned for use and is currently slated for demolition. It is anticipated that a cooking structure may be considered in the area, or slightly to the east of the Butler Shelter and above the Ranger Station. The Butler Shelter, aside from damage from rock impacts is in relatively good condition with a concrete slab-on-grade that is performing well.

GEOHAZARD MITIGATION

There are several locations at Camp Muir that are potentially impacted by geohazard issues that include the effects of glaciation, rock plucking, rockfall, and potential rock toppling issues. Some of these areas, west-northwest of the current Historic Guide Shelter (Ranger Station) and north of the Historic Public Shelter, could benefit from the installation of rockfall mitigation structures, mesh netting, or potentially rock bolting or micropile installation. While some of these areas, such as the area directly downhill and north of the Historic Guide Shelter (Ranger Station), may be better served by operational controls to limit encroachment of tent camping to some distance from the rock face. There are also rockfall-generating zones adjacent to Camp Muir outside of the Camp Muir Historic District (and, in one instance, encroaching into the Wilderness Boundary) where no action is allowable, dictating that only operational controls are feasible.

The geohazard mitigation measures discussed below are intended to stabilize the subject geohazard issues, using construction methodologies feasible for installation using only helicopter-portable equipment, such as modular drilling equipment. When considering potential mitigation alternatives for the various geohazard issues present at the site, our focus has been to limit mitigation equipment or materials transported to the site to that equipment/material which can be broken down into load sizes capable of being lifted to the site by available KAMAX helicopters, a medium lift helicopter slightly larger than the support ship used during our August 2020 exploration at the site. Transport of larger equipment can be made by larger helicopters, but in all cases, helicopter availability and construction requiring helicopter lift capabilities is often very dependent on firefighting resource requirements in any given year.

In most cases, the potential geohazard mitigation can be made to conform to the appearance of the historic structures at Camp Muir. For several mitigation measures, such as the use of high-strength wire mesh slope stabilization fabric or the use of rock bolts/micropiles, the material can be stained or powder coated to mimic the appearance of the surrounding rock, but these materials will be visible (in the case of wire rope mesh) or may be visible (in the case of stacked rock veneer over gabions or the termination of rock bolts that are buried within rock veneer). In reviewing the proposed geohazard mitigation elements, NPS and other stakeholders will have to decide if the benefits of the various proposed alternatives outweigh the (potentially) adverse visual impact to the site.

The proposed mitigation concepts and methodologies are discussed below, with reference to exhibits included in the **Figures** section of this report. The attached Figures 7 and 8 are maps of the site, adapted from drawings provided in the NPS Camp Muir Rehabilitation Plan 2012, that delineate the historic/wilderness limits and the locations of facilities that are part of the currently preferred restoration alternative. Additional exhibits (Figures 9-16) reference photographs taken by Terracon during our August 2020 site visit, and refer to specific geohazard areas and the potential mitigation alternatives available and feasible for installation at the site.

When reviewing this document, the reader is encouraged to consider the unique issues that steer design and construction efforts for a project such as Camp Muir Redevelopment. First of all, the site is remote, at an elevation of 10,080 feet. Access is only by alpine climbing or via helicopter lift. Delivery of most materials is only practical by helicopter lift, significantly limiting the type and weight of equipment that can be transported to the site. Regrading to facilitate slope safety could be easily accomplished on flat ground by driving a Caterpillar (CAT) 345 excavator with a bucket and thumb to the site and then flattening slopes or by simply reaching and moving potentially unstable boulders. Likewise, potentially unstable rock outcrops next to a highway would either be rock bolted into place or would be scaled or blasted off from their precarious position. None of these methodologies is feasible for the site, given the fact that a CAT 345 is simply too heavy to be lifted by a helicopter in one unit, and that same equipment is not modular, meaning that it cannot be readily broken down into modules that could be lifted by a helicopter. Likewise, no one would ever consider blasting unstable rock from within either the Historic District or especially from within the Wilderness area, so the proposed geohazard mitigation effort has to consider what can be accomplished largely by hand work (like the new Guided Public Sleeping Facility that is to be covered with a rock veneer), and with any machinery deployed to the site being (to a large extent) capable of being self-propelled to install items such as rock bolts or micropiles. The reader should also understand the unique atmospheric and glacial impacts to the site, where sustained high winds can overturn inadequately prepared structures and where the impacts of glaciation can easily scour and rip apart the native bedrock, especially along the margin of the Cowlitz Glacier, rendering the concept of a “100-year design life” meaningless and inadequate.

SITE FEATURES

Figures 7 and 8 are adapted from mapping products included in the “NPS Camp Muir Rehabilitation Plan 2012”. Figure 7 is an aerial image that reflects Camp Muir site conditions in 2002. Of significance on Figure 7 are the Wilderness/Historic District boundaries and the Sensitive Vegetation boundaries. These boundaries have been considered in selecting potential mitigation efforts at each of the geohazard features identified during the 2020 geotechnical project. Figure 8 illustrates the layout of the selected 2012 Rehabilitation Plan design elements, along with reference to historic facilities as well as the locations of the Cowlitz Glacier and the Muir Snowfield relative to the structures at Camp Muir.

GEOHAZARDS

The geohazards at the site include rockfall (located primarily at the west and east ends of Camp Muir), the effects of glaciation, rockfall, rock outcrop plucking and rock toppling issues located at the Camp Muir/Cowlitz Glacier margin, and rockfall and rock toppling near the east end of Camp Muir. The geohazard issues are shown on Figures 9, 10, 11, and 12, with illustration of potential mitigation approaches shown on the intervening Figures in the paragraphs below.

WEST SIDE GEOHAZARDS

Figure 9 shows the layout of the west half of Camp Muir in a photograph taken from the east side of the helipad. Shown on this scene are the Existing Butler Shelter (slated for removal), the Historic Guide Shelter (Ranger Station), the new west side toilets, and the Guided Public Shelter/Gombu. Also shown is the margin between the Camp Muir rock mass and the Cowlitz Glacier. Figure 10 shows the same features as Figure 9, but includes the geohazard issues identified during our August 2020 field work at the site.

ROCKFALL GENERATION ZONE

The identified geohazard issues in this area include a rockfall generation zone on a south to easterly-sloping talus/rock surface that includes sand and gravel-sized material to 6-foot-sized boulders, an enlarged image of which is shown on the attached Figure 11. The rockfall zone reportedly contributed the rockfall that previously impacted the Butler Shelter. Although the rock slope is intermediate in grade (25 to 40 degrees), it has the appearance of being able to occasionally contribute cobble- and boulder-sized material that, if started high enough on the slope, could gain sufficient momentum to injure personnel or damage the proposed new Storage and Cooking Facility, or even potentially result in damage to the Ranger Station.

We have developed two conceptual mitigation alternatives to reduce the potential adverse impacts of rockfall in this area. The first includes the construction of a gabion wall (Figure 12) and the second includes the installation of high-strength steel netting (Figure 13).

The gabion wall concept would include the construction of a gabion wall system using 3-foot by 3-foot gabions two units wide at the base and two units tall at the front face of the wall, with the wall system footprint being along the southerly and easterly-facing slope toe areas, at the same elevation as the bench now occupied by the Butler Shelter, as shown on the attached Figure 12. The gabion units would be wire laced together and would then be filled with selected on-site rock to fill the baskets. (While gabions are usually filled with rounded to semi-rounded gravel- to cobble-sized material, it is our opinion and experience that, for the limited height wall proposed here, angular rock fill will suffice to fill the gabion baskets.) Additional on-site rock (if available from existing on-site sources and if allowed by NPS) may be utilized; or, if on-site rock is not available, limited in quantity, or difficult to attain safely the use of imported materials may be required. These materials would then be dry-stacked in front of and on top of the gabion units, as shown on Figure 12. Although the gabions would have to be flown in, the grading and wall construction can be completed without the need for any powered equipment, and the finished appearance of the wall (except from uphill) would conform to the historic structure appearance of the site. Maintenance of the gabion wall system would have to include removal of accumulated rockfill and disposal of any accumulated rock debris either downhill to the Muir Snowfield or onto the Cowlitz Glacier.

If easy access to the uphill slope is desired, the second mitigation alternative for this geohazard would be installation of steel mesh material on the slope, such as Geobrugg's Spider Mesh or Deltax drape mesh material, as shown on the attached Figure 13. Installation of mesh is easily accommodated by drilling in rock bolt anchors at the top of the slope to anchor the rock retention material, and would likely require multiple drill set-ups with man-portable drilling equipment. (Larger anchor drilling equipment could also be employed, but such equipment would likely require helicopter relocation for each rock bolt anchor installation.) The utility of the drape mesh is that the weight of the mesh alone, when installed on a rock surface, reduces the energy of a rolling boulder by confining the rolling rock between the mesh and the face of the sloping rock surface, and with a surface sloping such as this surface, it is unlikely that any rockfall (within the confines of the contribution area shown on Figure 11) would translate to the toe of the slope.

The biggest issue associated with drape mesh installation is the potentially adverse impacts on any sensitive vegetation within the slope area. A second potentially adverse impact might be the visual appearance of the mesh material. Imagery of the proprietary Geobrugg materials (including the high-strength Spider Mesh and the lower-strength Deltax material) is shown on the attached Figure 13. The example imagery clearly shows that while the strongest material, the Spider Mesh, can be seen from several hundred feet away, the Tecco (medium strength) and the Deltax material (lowest relative strength material), which use progressively smaller wire sizes, can be difficult to see at a distance of more than 25 feet. While the Tecco and Deltax can also be dyed to mimic the site-specific rock color, and are likely strong enough to resist rock displacement forces alone, we would likely recommend (after final design, if this approach is considered as one of the final mitigation methodologies) the use of the Spider Mesh to resist the effects of off-season snow and ice downdrag.

RANGER STATION UNDERMINING

Our observations indicate that the north foundation line of the Historic Guide Shelter (Ranger Station), in its current location, is being actively undermined by the effects of glaciation, rockfall, rock plucking, and the potential for toppling of rock along the margin of the Camp Muir rock mass and the Cowlitz glacier, as shown on Figures 14, 15, and 16. Our observations further indicate that the north foundation wall and the fill supporting those walls (and at least several feet of the west and east walls adjacent to the north foundation wall) are also currently at risk for short-term undermining. Figures 14, 15, and 16 show where previous foundation stabilization efforts included the placement of additional foundation concrete, but where simple and glacial erosion has begun to significantly undermine both the repair sections and parts of the original fill/foundation zones. At this point, it is our opinion that the more recent repair sections are affording little or no protection to the existing original fill and foundation zone, and that the more recent repairs may, in fact, be acting to downdrag part of the original structure. Based on this concern, it is our opinion that the most recent concrete placements should be removed and an Operation and Maintenance (O&M) monitoring program including the installation of crack

monitoring gauges be established at specific locations. Monitoring should be implemented to determine rate of movement (crack separation) of the exiting superstructure and foundation, and action limits should be set as the basis for initiating immediate remedial efforts. Removal of the existing concrete may be accomplished by means of expansive demolition gel materials (as opposed to the use of conventional explosives or jackhammering), which would result in the concrete breaking up and falling onto the glacier below. This can be accomplished by means of drilling a series of 1 to 1-½-inch diameter holes into the existing concrete at spacing between nine and 24 inches (depending on material properties of expansive gel selected) and filling the holes with an expansive demolition gel, such as Bentanomite or Dexpan. Removal of the concrete debris would need to be further evaluated based on necessitate and potential size of debris generated.

We understand that previously proposed repairs for this area may have included the installation of horizontally-drilled rock bolt anchorages installed from the Cowlitz Glacier side of the rock mass. Based on our August 2020 observations, it appears that the foundation undermining has progressed to a point where new repairs utilizing horizontally-oriented rock bolts would require the installation of a thick (two feet plus) reinforced concrete diaphragm wall through which new rock bolts could be installed to maintain the Ranger Station in its current location. The primary drawback with this alternative would likely be the lack of a sufficient bond zone at the downhill end of the subject horizontal (or near-horizontal) rock bolts to provide any support for the diaphragm wall. In addition, drilling and installation of rock bolts strong enough to provide sufficient lateral stability would likely require drilling equipment that would require helicopter assistance for moving between each drill setup, increasing the noise, cost, and visual impacts to the site.

As an alternative approach, we suggest that NPS consider the deconstruction of the current Ranger Station, excavation of the upper fill bench supporting the Ranger Station (from the yellow line down to the red line/arrow shown on Figure 17), and shifting the Ranger Station footprint at least five feet to the south, as indicated by the red arrow on Figure 17. To add to the longevity of the new foundation, we recommend that NPS and the stakeholders consider permeation grouting of the rock/soil mass along the northern foundation line be performed to a depth of at least 20 feet below grade, and then that a micropile A-wall be constructed along and beneath the northern foundation line of the new Ranger Station, as shown conceptually on the attached Figure 17. The permeation grouting will add erosional resistance to the (currently) unconsolidated soil and rock mass, and the installation of the micropile A-wall (at least 25 feet in depth below the new foundation elevation), will provide structural support to bypass the effects of lateral erosion (and future undermining) of the new foundation for considerably longer than just moving the foundation alone. (The type of micropile installed would likely be a gravity-grouted, duplex micropile consisting of an outer steel casing used to assist in resisting lateral bending stress and an inner steel reinforcing bar, such as a Grade 75 #14 or larger bolt to develop anchorage within the fractured rock mass at depth.) Both the vertical and the battered micropiles can be installed with

a single type of drill rig, and depending upon availability, that drill rig might be track-mounted so that it runs under its own power and does not require helicopter relocation between micropile locations.

During our conference all on December 1, 2020 with NPS, it was discussed that it may be necessary to leave the Ranger Station in its current location due to its historic value and existing permits issued for redevelopment. An alternative to allow the Ranger Station to remain in place would include the installation of a micropile A-wall, similar to discussed above, permeation grouting, and further armoring (likely with a concrete/shotcrete material) of the slope below the Ranger Station that is impacted by the Cowlitz Glacier to provide additional protection. The primary concern to this approach is the ability to provide a feasible and constructable design for connection of the micropile underpinning system to the Ranger Station foundation elements (which may or may not be reinforced). This would require further coordination between the geotechnical and structural design disciplines to provide an underpinning design.

EAST SIDE GEOHAZARDS

On the east side of Camp Muir, our team identified two geohazard areas, both involving the large, arete-like outcrop east to north of the Historic Public Shelter and the new toilets at the east end of Camp Muir.

Based on the Wilderness Boundary shown on Figure 7, the rock outcrop above the denoted tents on Figure 18 is likely situated within the designated Wilderness Area, so no rock slope stabilization efforts can be installed or performed. The only means to protect the Public-At-Large is to institute operational controls such as limiting how close climbers can pitch their tents to the adjacent rock face. In this case, the minimum offset should be AT LEAST 50 feet between the rock face and the area where tents can be located. Further to the west (toward the Ranger Station), the minimum offset distance between the tent locations and the rock face should be AT LEAST equal to the height of the rock slope. If the snow is sloping downward as shown at the right of the Figure 18, the minimum offset distance between the rock face and the tent locations should be AT LEAST 50 feet between the rock face and any tents.

Further south on the same outcrop, the rock outcrop northeast of the new toilets exhibits two different primary joint sets, each of which tends to develop its own rockfall failure mode and fall direction. In general, the rock above the dashed yellow line exhibits a dominant joint set that plunges north, or toward the Cowlitz Glacier. Observations indicate that rock within this joint set group tends to crack away and topple downward to the south (right on Figure 19), with the detached rock blocks then tending to slide downhill on a relatively flat joint face. Our experience indicates that most of the rockfall from this joint group will tend to develop little kinetic energy. In addition, most of the rockfall-generating slope is east of the Historic Public Shelter, so the Historic Public Shelter should not be impacted by rockfall from the upper joint group. The new toilets could be adversely impacted from rockfall above this slope (the upper joint set). Rockfall

generated from the upper joint set can be mitigated by installation of a barrier wall such as a two-course gabion wall shown on the attached Figure 12 that is filled with on-site rock and which is then veneered by the same local rock dry-stacked to maintain a historic appearance. Rock outcropping from the lower joint set tends to plunge to the south toward the Muir Snowfield. Unlike rock generated from the upper joint group, rockfall from the lower joint group will tend to fall or roll vertically downward until an elevation about five feet below the floor of the Historic Public Shelter, where dislodged rock will follow the steep till slope down to the Cowlitz Glacier. Rockfall from the lower joint set can be disregarded as long as the proposed operational controls (offset distances between the rock face and any tents discussed previously) are adopted.

EARTHWORK CONSIDERATION

The primary consideration for redevelopment of the Camp Muir area will be the type and size of equipment available to move materials, dig foundation excavation, or construct retaining walls. It is likely that small tracked equipment such as mini-excavators (such as CAT 305 Series, less than 15,000 pounds) or skidsteers (such as CAT 289 or 299 Series, less than 15,000 pounds) will be required to move the large cobble/boulder rocks required for retaining wall construction and level areas to be used in future construction (such as near the East Toilet area). Mobilization of equipment this heavy will likely require a heavy lift helicopter such as a Boeing Vertol CH-47 Chinook, the availability of which may be very limited during the summer fire season. It appears that an ample supply of large size material will be available on the extreme west and east portions of the site for use in retaining wall construction for wall rock, if permitted by NPS. Backfill materials may need to be imported to the site, along with compaction equipment. If hand operated equipment is utilized, backfill should be conducted in lifts not exceeding four inches in thickness; if self-propelled equipment can be mobilized, lift thickness should not exceed nine inches.

For the site, earthwork will need to be conducted in the warmer and drier months of the year which provides a small window of opportunity. Planning of construction sequencing will therefore be very important. Preparation of foundations subgrade retaining wall earthwork considerations will be discussed in subsequent sections.

SHALLOW FOUNDATIONS

Based on the limited subsurface exploration conducted using DCP, Test Pits, and geophysical testing, it appears that shallow foundations can be considered for use at the site to support new structures or relocated structures on new foundations. Further evaluation of design loads and estimated settlement will be required to determine, based on structure size and loading, the anticipated settlement (total and differential) for the site. However, it is assumed that, in the areas designated as green in the Geohazard and Developable Areas diagram in this report, shallow foundations will bear within the permafrost layer encountered in each test pit at a minimum depth

of 3.5 feet below existing grade. Where existing fill or bedrock are likely to be encountered, east of the helipad, allowable bearing capacity will be far in excess of the likely loading conditions from the structures. Therefore, preparation of the rock or cobble fill zones will be important.

Shallow Foundation Design Parameters

Item	Description
Maximum Allowable Bearing pressure	2,500 psf (foundations bearing within permafrost) 3,500 psf (foundation bearing on upper fracture bedrock)
Minimum Foundation Dimensions	Columns: 36 inches Continuous: 18 inches
Minimum Embedment below Finished Grade	42 inches minimum or at least 18 inches into permafrost, whichever is deeper

Design Parameters - Uplift Loads

Uplift resistance of spread footings can be developed from the effective weight of the footing and the overlying soils. The effective weight of the soil prism defined by diagonal planes extending up from the top of the perimeter of the foundation to the ground surface at an angle of 20 degrees from the vertical can be included in uplift resistance. The maximum allowable uplift capacity should be taken as a sum of the effective weight of soil plus the dead weight of the foundation, divided by an appropriate factor of safety. A maximum total unit weight of 100 pcf should be used for the backfill.

Foundation Construction Considerations

The footing excavations should be evaluated under the direction of the Geotechnical Engineer or a representative on-site that can communicate findings to the Geotechnical Engineer. The base of all foundation excavations should be free of water and loose material, prior to placing concrete. Concrete should be placed soon after excavating to reduce bearing soil disturbance. Care should be taken to prevent wetting or drying of the bearing materials during construction. Excessively wet or dry material or any loose/disturbed material in the bottom of the footing excavations should be removed before foundation concrete is placed. The excavations into permafrost should be insulated between exposure and placement of concrete to limit potential for thawing of the bearing surface.

If unsuitable bearing soils are encountered at the base of the planned footing excavation, the excavation should be extended deeper to suitable soils, and the footings could bear directly on these soils at the lower level or on lean concrete backfill placed in the excavations.

HISTORIC GUIDE SHELTER (RANGER STATION) UNDERPINNING CONCEPTUAL OPTION

The use of underpinning at the Historic Guide Shelter (Ranger Station) using a system of micropiles may be considered, if the structure is to remain in its current location. This form of deep foundation is recommended in lieu of a driven pile or drilled shaft system due to installation characteristics that are more positive and better suited to the subsurface conditions. Micropiles can be cored or advanced with downhole hammer equipment in the boulder/cobble moraine till deposits, and through the less competent, fractured upper bedrock layers to reach the bond zone and achieve relatively large axial capacity with small-diameter drilling. In addition, drilling equipment for micropiles is much less space-and noise-intrusive than conventional pile driving or shaft drilling rigs; such equipment is also more maneuverable. From a mobilization standpoint, micropile drilling equipment is probably the only deep foundation equipment that can be helicopter lifted to the project site. At least one specialty contractor in Washington has modular micropile drilling equipment designed to be transported in modules light enough to be transported by AS350 Astar helicopters, whereas pile driving or drilled shaft rigs are much larger and would likely not be able to be mobilized to the site.

Micropile Selection

There are a variety of micropile sizes, types, and installation methods which could be designed and employed for the project. The foundation design recommendations contained herein are based on a Type B, composite micropile consisting of a casing with a centralized reinforcing bar installed as defined by the Federal Highway Administration in the Micropile Design and Construction Reference Manual, FHWA NHI-05-039, December 2005. This method would include the following basic installation steps:

- Drillhole advanced with casing to the design depth in the andesite bond zone.
- Casing is tremie-filled with grout (neat cement) and the reinforcing bar centralized for the full length within the casing.
- Withdraw casing to a typical depth (plunge length) below the rock-till interface as the grout column is pressurized to establish bond.

For purposes of this report, the use of a composite micropile would be the approach recommended for underpinning to provide axial and lateral capacity. A composite micropile consists of a larger diameter steel pipe casing (such as a 7-inch diameter, 0.408-inch wall pipe meeting specific steel grade for the design) with a centralized steel all-thread bar (such as No. 14 Grade 75 Williams all-thread). The grout would consist of a neat cement with Type I-II cement at water:cement ratio not exceeding 0.45, attain a minimum 3-day compressive strength of 2,000 psi, and a 28-day compressive strength of at least 4,000 psi. Actual required loading conditions to be supported would need to be determined to facilitate a micropile design for underpinning.

SEISMIC CONSIDERATIONS

The seismic design requirements for buildings and other structures are based on Seismic Design Category. Site Classification is required to determine the Seismic Design Category for a structure. The Site Classification is based on the upper 100 feet of the site profile defined by a weighted average value of either shear wave velocity, standard penetration resistance, or undrained shear strength in accordance with Section 20.4 of ASCE 7 and the International Building Code (IBC). Based on the soil/bedrock properties encountered at the site and as described on the exploration logs and results, and results of our limited geophysical testing it is our professional opinion that the **Seismic Site Classification is C**. Specific seismic activity is being monitored at the site by the University of Washington and data for recent seismic events in the vicinity at the Pacific Northwest Seismic Network link, https://pnsn.org/seismogram/current/rcm, for seismogram data acquired at the Camp Muir location A Peak Ground Acceleration (PGA) coefficient of 0.48g is expected for a 2425 year (two percent in 50 years) interval based on United Stated Geological Survey (USGS) Unified Hazard Tool. A site-specific seismic analysis was not part of this scope of work, but Terracon can provide if requested in a subsequent phase of work.

FLOOR SLABS

Design parameters for floor slabs assume the upper 12 inches of the subgrade is scarified and compacted. Based on location and logistics, a procedural approach is recommended for compaction, in which a specific number of passes of a piece of compaction equipment flown to the site is conducted prior to placement of subsequent lifts. Based on what equipment can be mobilized to the site, we can further develop an appropriate procedural specification for use in slab subgrade support preparation. Specific attention should be given to positive drainage away from the structure and positive drainage of the aggregate base beneath the floor slab.

Floor Slab Design Parameters

Item	Description
Floor Slab Support	Properly prepared subgrade and a minimum of 6 inches of crushed base course
Estimated Modulus of Subgrade Reaction	125 pounds per square inch per inch (psi/in) for point loads

RETAINING WALLS

Rockery Wall Systems

A rockery is a retaining or protection structure that consists of stacked rocks without mortar, concrete, or steel reinforcement. Although the rocks are stacked in an "interlocking" pattern, there are no mechanical connections made between the individual rocks. Rather, these structures rely on the weight, size, shape, and interface friction of the rock elements to provide overall stability. We recommend that rockeries be considered, as defined by the Federal Highway Administration in the Rockery Design and Construction Guidelines, FHWA-CFL/TD-06-006, November 2006, as these systems will provide the desired grade separation at the redevelopment site along with internal stability that is not currently exhibited by the dry-stack rock walls at present. Design focuses on the use of rocks generally greater than 18 inches and rock weights generally greater than 200 pounds which provide stability through mass of the rocks and inter-rock friction.

Consideration given to rockeries will require some additional NPS input on specific maximum height of wall to assist in providing design parameters. Rockeries typically are specified by their height, base width (function of height), face batter (function of height) and individual rock weight and/or size (see Figure 1). It is likely that sufficient rock by mass and size can be attained from the talus slopes along the east and west sides of the site (if sufficient quantities are available to be safely mined from existing sources, if not off-site materials may need to be flown to site to facilitate rockery construction); however, loading and transport equipment will be needed which may be difficult to mobilize such large equipment (as noted in the **Earthwork** section) to the site.

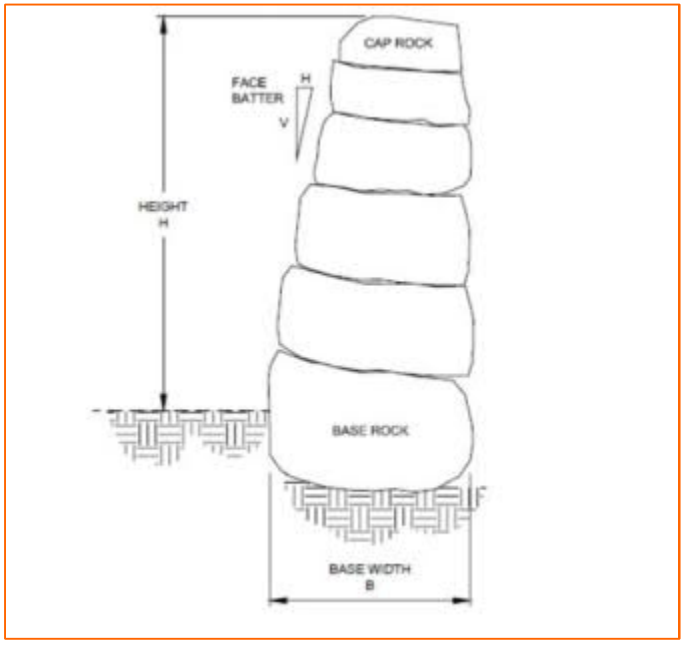


Figure 1: Graphic from FHWA Diagram showing definitions of height (H), base width (B), face batter, and relative rock size

Based on observed retaining wall failures, the inclusion of drainage behind the rockery face should be considered in the design, using open-graded drainage aggregate and separation fabric with a slotted drain pipe graded to a positive outlet. This will relieve the potential hydrostatic pressure from building up behind the wall face. Backfill materials behind the retaining walls generally consist of relatively free-draining cobble, gravel, and sand sized particles; however, walls in excess of four feet should consider additional drainage elements to provide for more rapid drawdown of potential water in the backfill zones.

SLOPE STABILITY

Mechanics of Stability

Slope stability analyses take into consideration material strength, presence and orientation of weak layers, water (piezometric) pressures, surcharge loads, and the slope geometry. Generally, these analyses are performed using the theory of Limiting Equilibrium, where the forces and moments driving instability are compared to the forces and moments resisting instability. Mathematical computations are performed using computer-assisted simulations to calculate a Factor of Safety (FS) against failure. Minor changes to slope geometry, surface water flow, and/or elevated groundwater levels could result in slope instability. Reasonable FS values are dependent upon the confidence in the parameters utilized in the analyses performed, among other factors related to the project itself.

Geometric Analysis Results

Preliminary slope stability analyses were performed for the cross-section geometry corresponding to Geophysical Survey Line 03, shown on the **Exploration Plan**. The cross-section geometry used in the analysis for the slope was interpolated from the topographic site plan contained within the West Toilet Design Drawing Set, provided by the NPS with the sheet titled Site Plan Sub Sheet No. C dated June 29, 2018 (the interpolation was consistent with our field measurements of the slope at Line 03 when measuring geophone profile). Preliminary stability analyses were conducted using the computer program GEOSTASE – Slope Stability Analysis Software by Gregory Geotechnical Software.

Slope Material Properties

Based on the results of our field investigation (test pits, DCP tests, and geophysical survey lines), currently completed laboratory testing, and geotechnical analysis, we have developed a generalized subsurface profile for our analysis of the south slope below the Client Shelter/Gombu (shown as Line 03 on the **Exploration Plan**). Laboratory testing for remolded consolidated-drained direct shear was completed on a sample of gravel with sand from Test Pit TP-01, located along the slope obtained from a depth of 26 inches below the slope surface. The test was performed in general accordance with ASTM D3080 – Direct Shear Test of Soils Under Consolidated Drained Conditions. In the remolding process, we packed the gravel with sand material into the direct shear apparatus in a manner that we believe simulated the in-situ conditions encountered on the site, in that the slope was generally loose near surface, as we did not have the ability to obtain in-situ samples for determination of in-place density. As such, the remolding process for the pumice sand/gravel obtained dry unit weight values on the order of 55 to 60 pounds per cubic foot (pcf), moisture contents in the range of 23 to 25 percent, resulting in a shear envelope with a cohesion intercept of 204 pounds per square foot (psf) and friction angle of 46 degrees. This result appears reasonable as the sand and gravel sized particles are very angular in nature (and as observed under magnification), and our experience with natural pumice indicates the light unit weight is consistent with the material properties; and, the angle of repose of the slope is near 45 degrees in the location of Test Pit TP-01.

For the materials below which the test pits were terminated, the shear wave profiles were reviewed to determine relative density, as discussed above. Based on the shear wave profile for Seismic Line 03, we assigned five distinct subsurface layers. The surficial sand profile was developed based on the results of the laboratory testing results discussed above. The parameters developed for the underlying gravel with sand layer were determined based on a combination of the DCP results from DCP-01, visual observation of the layer in TP-01, and the shear wave velocity profile obtained on Line 03. The velocity within the loose, surficial sand was on the order of 440 fps, with an increase to 1263 fps in the gravel with sand layer. It should be noted that the shear wave velocity was consistent within the gravel with sand layer and into the permafrost layer

encountered in TP-01. As such, we have modeled the permafrost layer to be consistent with layers encountered above the bedrock. That is, where the shear wave velocity varied the parameters developed for the stability analyses are based on the soil matrix conditions, and the permafrost is not inherently modeled as adding or detracting from those material properties. The permafrost layer impacts, if melting occurs, are modeled as a water level encountered at the permafrost layer and allowing the layer it is encountered within (i.e. gravel with sand) and overlying layers to become saturated as it potentially melts. The water surface is modeled at the depth of permafrost for the majority of the slope, with the water seeping at the end of our Line 03, where the Muir Snowfield was encountered. For the remaining layers, the shear wave velocities and correlations as discussed above were used to develop appropriate parameter for use in the stability analyses. Material properties used in the analyses are shown below:

Soil Model No.	Material	Moist Unit Weight (pcf)	Cohesion (psf)	Friction Angle (degrees)
1	Surficial Sand – Pumice (SP)	75	200	45
2	Poorly Graded Gravel w/ Sand (GP)	110	0	36
3	Poorly Graded Sand (SP)	75	100	40
4	Poorly Graded Gravel (GP)	115	0	38
5	Upper Fractured Bedrock / Andesite	140	2500	0

Preliminary Slope Stability Analyses Results

The stability of the slope was modeled with several conditions to develop preliminary Factors of Safety with regards to slope instabilities. Our stability analyses included the evaluation of the existing site slope in all cases based on topography as discussed above. Static analysis of stability for the slope below the Gombu Shelter (Figure 1), static analysis with water layer modeled at permafrost elevation (Figure 2), static analysis with water layer and structural load of 100 psf at the Gombu Shelter (Figure 3), seismic analysis of the of the slope with the addition of a 0.48g horizontal acceleration modeling the design earthquake (Figure 4), seismic analysis with the addition of ½ the above horizontal acceleration and water layer (Figure 5), and seismic analysis with the addition of 0.48g horizontal acceleration and 100 psf structural loading from the Gombu Shelter (Figure 6)

The calculated FS for the critical surfaces identified in each section is shown in the table below. The typically accepted minimum FS for static and seismic slope stability supporting improvements are 1.5 and 1.1, respectively. For all cases except the seismic and water condition, the minimum FS is achieved. The scenario in which seismic and water are combined resulted in the FS of the critical failure surface of 1.099. This value is acceptable for stability of the slope, based on the

fact that the scenario analyzed is very unlikely to occur; that is, the probability of an earthquake event (causing the maximum PGA for a 2,475-year return interval) occurring at the same time that permafrost melt is resulting in saturated subgrade conditions is low. The value also rounds to 1.1, or an acceptable FS. Slope stability results are included in the **Figures** section of this report.

Condition	Figure No.	Minimum Calculated Factor-of-Safety for Slopes
Static	1	1.9
Static and Water	2	1.4
Static, Water, and Building Load	3	1.4
Seismic	4	1.3
Seismic and Water	5	1.1
Seismic and Building Load	6	1.3

Surficial Slope Instability

A previous landslide was observed on the south facing talus slope to the west of the Butler Shelter, in which a recent landslide mass and trough was clearly evident. The surficial slope instability appeared to impact the upper five feet of the subsurface profile, and our stability analyses performed as discussed above are taking into consideration the mode and shape of failure to ensure subsurface parameters used in the model are reasonable. Slope surficial instabilities were not observed immediately below any of the existing structures on south slope. Based on visual reconnaissance, the slope appears relatively stable with uniform slope geometry. The slope material is very loose and unconsolidated, but stable in its current environment.

SEQUENCING CONSIDERATIONS

As discussed in the December 1, 2020 conference call with NPS, guidance is desired regarding sequencing of mitigation recommendations, based on relative geotechnical importance, to inform NPS decision-making for the project. As such, we are providing our recommendations regarding prioritization of hazards, from a geotechnical perspective, for the Camp Muir Developed Site. The following presents our recommendations for major priorities based on the findings and our discussion with NPS on overall objectives for the project:

1. Remove the most recent concrete placements from the north footing line of the Historic Guide Shelter (Ranger Station) to reduce the dead load at the location of foundation undermining. This action, in our opinion, would be the most effective means of improving conditions that are within our control (we cannot impact the glacier movement) to reduce

current rates of movement/cracking within the structure. Along with this action, it is recommended that more formalized monitoring of the existing cracks along the north side of the Ranger Station be implemented. The use of several crack monitors along with a documented reading schedule for recording crack spacing should be developed and implemented by Operations and Maintenance (O&M).

2. Demolition of existing Gombu Structure and construction of new Guided Public Sleeping Facility within a zone delineated as suitable for existing or new construction. This recommendation assumes that a monitoring program of the Ranger Station is implemented by O&M and crack movements are tolerable.
3. Address the West Side rockfall/geohazards above the existing Butler Shelter and Ranger Station. Addressing these hazards and limiting potential for future rockfall from these areas will provide more developable space for the next phase of the Camp Muir Redevelopment plan and protect the Ranger Station from rockfall.
4. If a monitoring program is implemented by O&M, underpinning of the Ranger Station could be delayed to a later phase, as the structure is not in imminent threat of foundation failure in its current condition. The monitoring program will allow for a real-time indication of movement in excess of a set action limit in which underpinning would become a critical path item. Provided an adequate monitoring program is implemented, the underpinning of the Ranger Station would be a lower priority to allow for determination of final location and design of the underpinning system.

GENERAL COMMENTS

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Natural variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be notified immediately so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence or collaboration through this system are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and

are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client, and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly impact excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety, and cost estimating including, excavation support, and dewatering requirements/design are the responsibility of others. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

FIGURES

Contents:

Stability Analyses (Figures 1-6)
Site Aerial Image 2002 (Figure 7)
Camp Muir Rehabilitation Plan Selected Alternative (Figure 8)
Geohazard Rockfall Evaluation West Side of Site (Figures 9-13)
Historic Guide Shelter Foundation Evaluation (Figures 14 -17)
Geohazard Rockfall Evaluation East Side of Site (Figures 18-19)

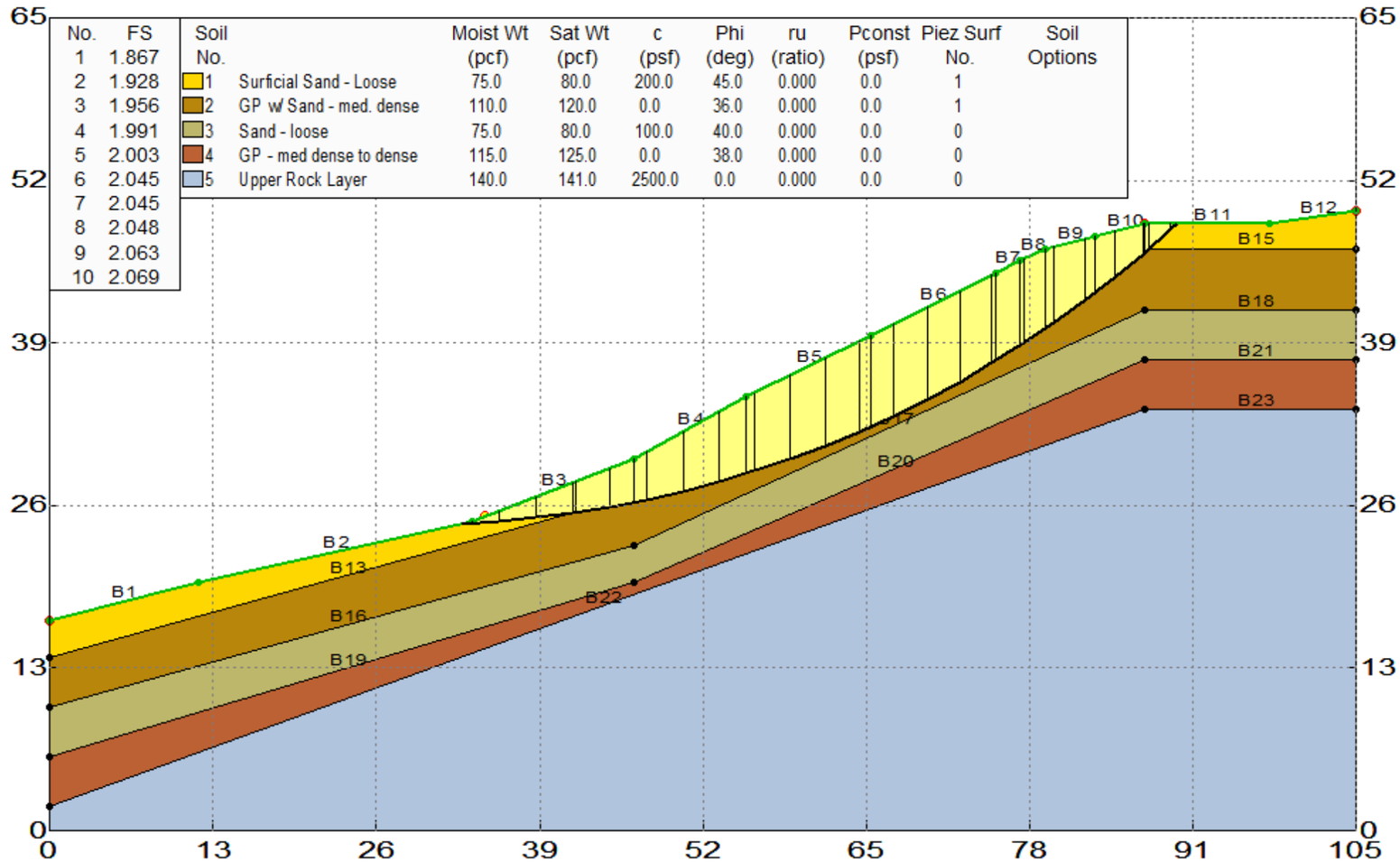
Camp Muir Developed Area
South Slope - Below Gombu Shelter - Static

Terracon Consultants, Inc. - MDH

\Slope Below Gombu Shelter_lower.gsd

No.	FS
1	1.867
2	1.928
3	1.956
4	1.991
5	2.003
6	2.045
7	2.045
8	2.048
9	2.063
10	2.069

Soil No.		Moist Wt (pcf)	Sat Wt (pcf)	c (psf)	Phi (deg)	ru (ratio)	Pconst (psf)	Piez Surf No.	Soil Options
1	Surficial Sand - Loose	75.0	80.0	200.0	45.0	0.000	0.0	1	
2	GP w/ Sand - med. dense	110.0	120.0	0.0	36.0	0.000	0.0	1	
3	Sand - loose	75.0	80.0	100.0	40.0	0.000	0.0	0	
4	GP - med dense to dense	115.0	125.0	0.0	38.0	0.000	0.0	0	
5	Upper Rock Layer	140.0	141.0	2500.0	0.0	0.000	0.0	0	



GEOSTASE FS = 1.867

Simplified Bishop Method

GEOSTASE Slope Stability Analysis

GEOSTASE® by GREGORY GEOTECHNICAL SOFTWARE

Project Manager: MDH	Project No. C4205024	Terracon	Static Condition
Drawn by: MDH	Scale: NA	1392 13th Ave. SW, Great Falls, MT	Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area
Checked by: BJW	File Name:	PH. (406) 453-5400 FAX. (406) 761-6655	Figure 1
Approved by: BJW	Date: 12/23/2020		

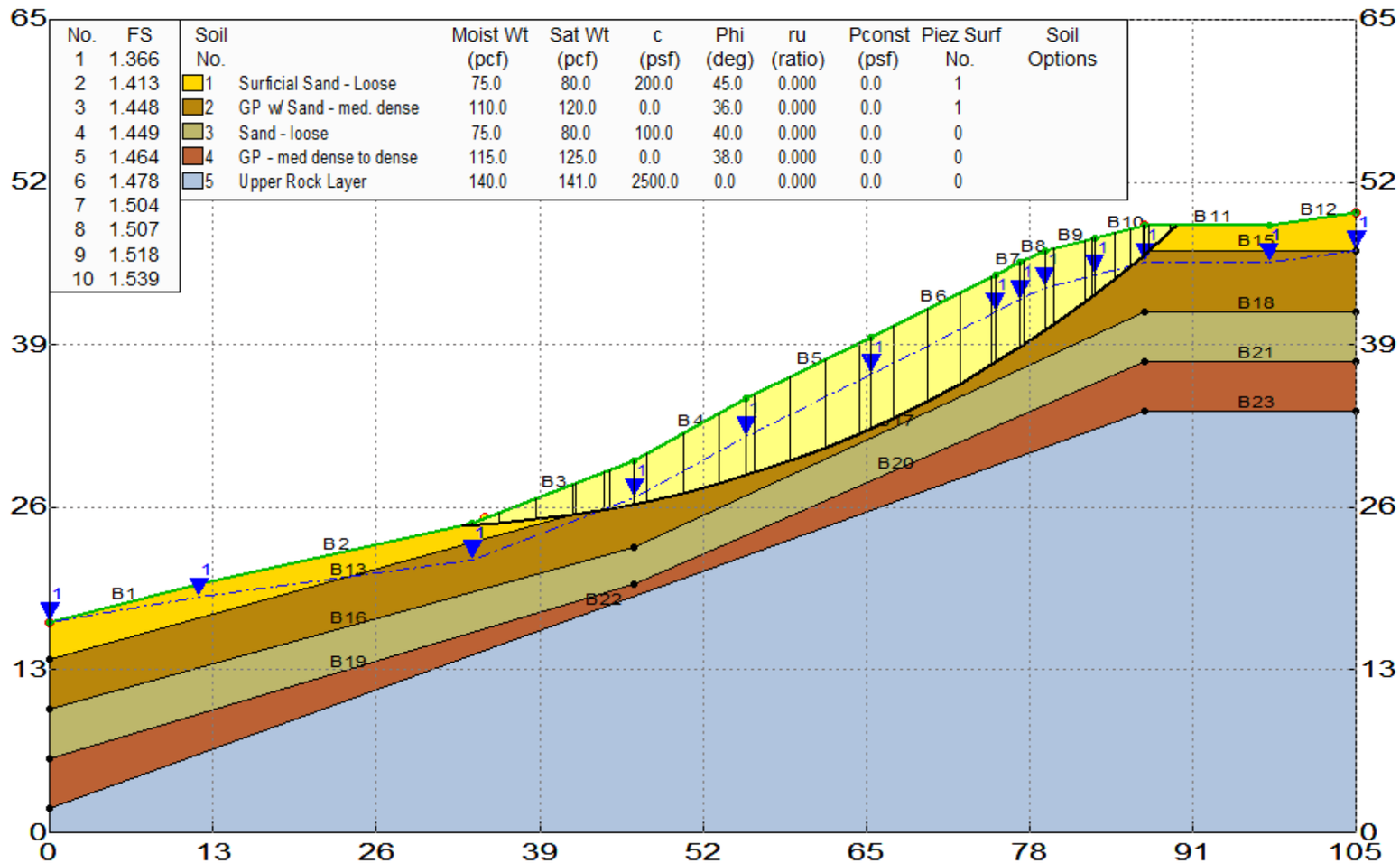
Camp Muir Developed Area
South Slope - Below Gombu Shelter - Static w/ Water

Terracon Consultants, Inc. - MDH

\Slope Below Gombu Shelter_lower.gsd

No.	FS
1	1.366
2	1.413
3	1.448
4	1.449
5	1.464
6	1.478
7	1.504
8	1.507
9	1.518
10	1.539

Soil No.		Moist Wt (pcf)	Sat Wt (pcf)	c (psf)	Phi (deg)	ru (ratio)	Pconst (psf)	Piez Surf No.	Soil Options
1	Surficial Sand - Loose	75.0	80.0	200.0	45.0	0.000	0.0	1	
2	GP w/ Sand - med. dense	110.0	120.0	0.0	36.0	0.000	0.0	1	
3	Sand - loose	75.0	80.0	100.0	40.0	0.000	0.0	0	
4	GP - med dense to dense	115.0	125.0	0.0	38.0	0.000	0.0	0	
5	Upper Rock Layer	140.0	141.0	2500.0	0.0	0.000	0.0	0	



GEOSTASE FS = 1.366

Simplified Bishop Method

GEOSTASE Slope Stability Analysis

GEOSTASE® by GREGORY GEOTECHNICAL SOFTWARE

Project Manager: MDH	Project No. C4205024	Terracon	Static with Water Condition
Drawn by: MDH	Scale: NA	1392 13th Ave. SW, Great Falls, MT	Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area
Checked by: BJW	File Name:	PH. (406) 453-5400 FAX. (406) 761-6655	Figure 2
Approved by: BJW	Date: 12/23/2020		

Camp Muir Developed Area

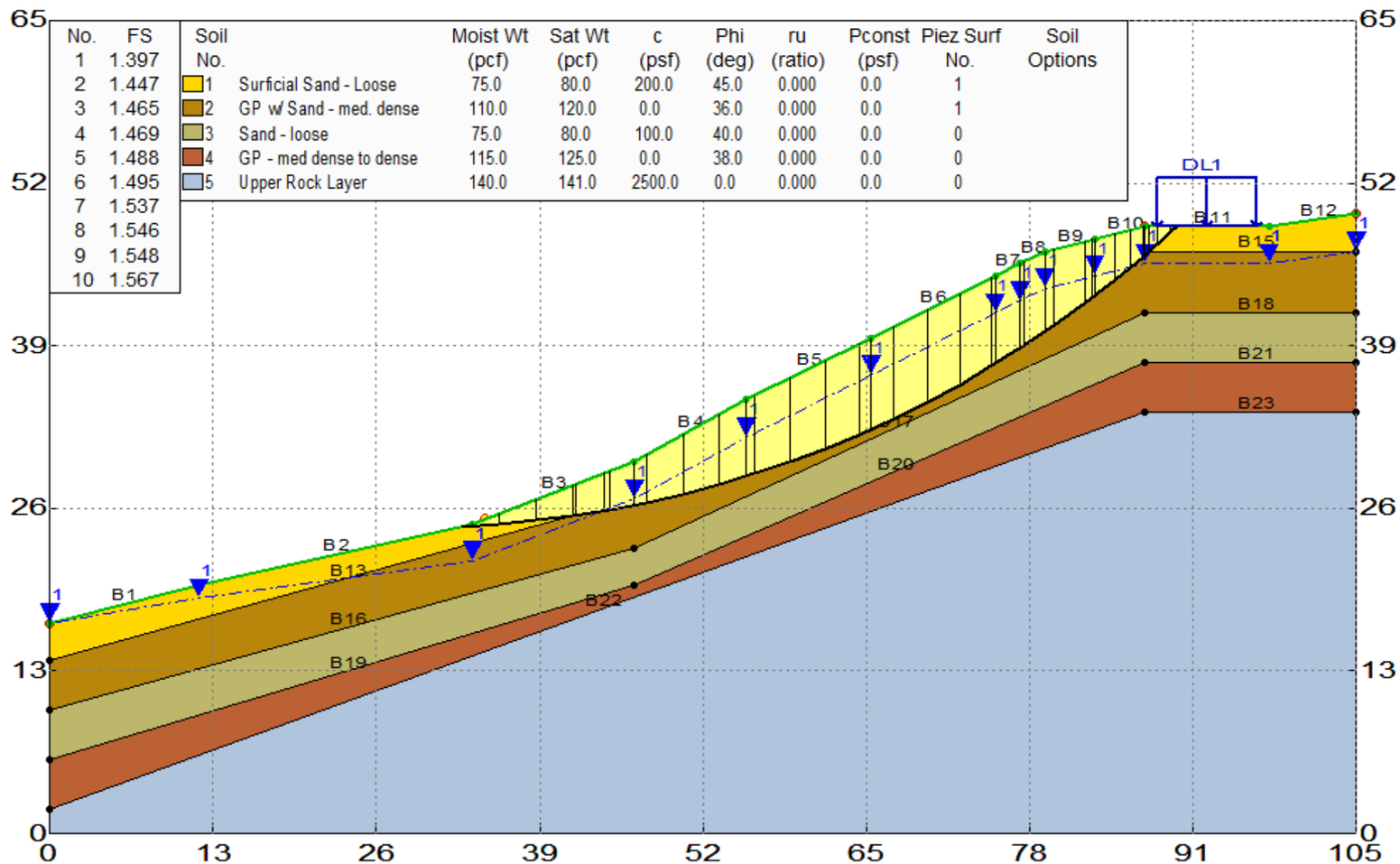
South Slope - Below Gombu Shelter - Static w/ Water and Building Load

Terracon Consultants, Inc. - MDH

\Slope Below Gombu Shelter_lower.gsd

No.	FS
1	1.397
2	1.447
3	1.465
4	1.469
5	1.488
6	1.495
7	1.537
8	1.546
9	1.548
10	1.567

Soil No.		Moist Wt (pcf)	Sat Wt (pcf)	c (psf)	Phi (deg)	ru (ratio)	Pconst (psf)	Piez Surf No.	Soil Options
1	Surficial Sand - Loose	75.0	80.0	200.0	45.0	0.000	0.0	1	
2	GP w/ Sand - med. dense	110.0	120.0	0.0	36.0	0.000	0.0	1	
3	Sand - loose	75.0	80.0	100.0	40.0	0.000	0.0	0	
4	GP - med dense to dense	115.0	125.0	0.0	38.0	0.000	0.0	0	
5	Upper Rock Layer	140.0	141.0	2500.0	0.0	0.000	0.0	0	



GEOSTASE FS = 1.397

Simplified Bishop Method

GEOSTASE Slope Stability Analysis

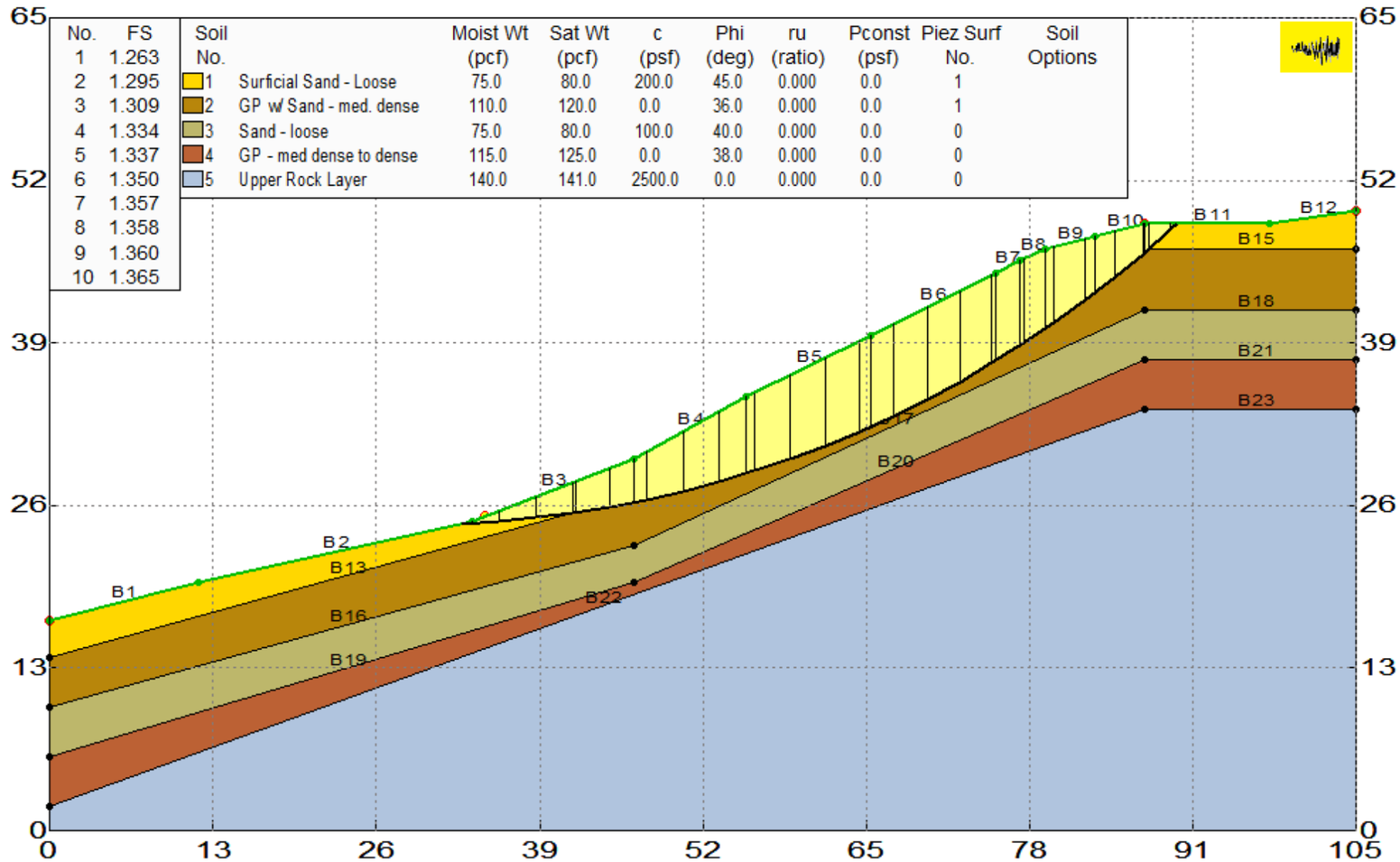
GEOSTASE® by GREGORY GEOTECHNICAL SOFTWARE

Project Manager: MDH	Project No. C4205024	Terracon	Static with Water and Building Condition
Drawn by: MDH	Scale: NA	1392 13th Ave. SW, Great Falls, MT	Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area
Checked by: BJW	File Name:	PH. (406) 453-5400 FAX. (406) 761-6655	Figure 3
Approved by: BJW	Date: 12/23/2020		

Camp Muir Developed Area
South Slope - Below Gombu Shelter - Seismic

Terracon Consultants, Inc. - MDH

\Slope Below Gombu Shelter_lower.gsd



No.	FS
1	1.263
2	1.295
3	1.309
4	1.334
5	1.337
6	1.350
7	1.357
8	1.358
9	1.360
10	1.365

Soil No.		Moist Wt (pcf)	Sat Wt (pcf)	c (psf)	Phi (deg)	ru (ratio)	Pconst (psf)	Piez Surf No.	Soil Options
1	Surficial Sand - Loose	75.0	80.0	200.0	45.0	0.000	0.0	1	
2	GP w/ Sand - med. dense	110.0	120.0	0.0	36.0	0.000	0.0	1	
3	Sand - loose	75.0	80.0	100.0	40.0	0.000	0.0	0	
4	GP - med dense to dense	115.0	125.0	0.0	38.0	0.000	0.0	0	
5	Upper Rock Layer	140.0	141.0	2500.0	0.0	0.000	0.0	0	

GEOSTASE FS = 1.263

Simplified Bishop Method

kh = 0.18000

GEOSTASE® by GREGORY GEOTECHNICAL SOFTWARE

GEOSTASE Slope Stability Analysis

Project Manager: MDH	Project No. C4205024	Terracon	Seismic Condition
Drawn by: MDH	Scale: NA	1392 13th Ave. SW, Great Falls, MT PH. (406) 453-5400 FAX. (406) 761-6655	Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area
Checked by: BJW	File Name:		Figure 4
Approved by: BJW	Date: 12/23/2020		

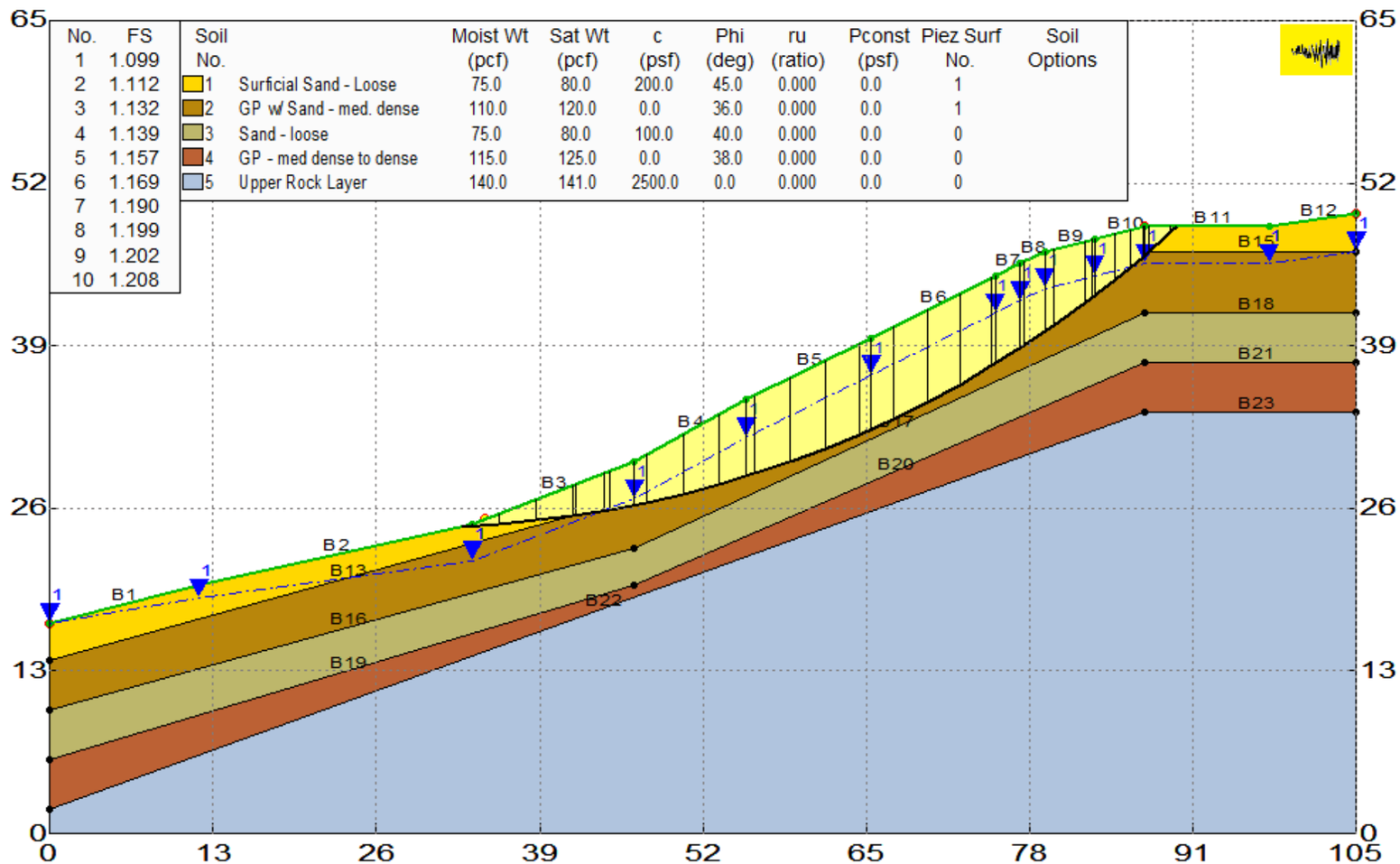
Camp Muir Developed Area
South Slope - Below Gombu Shelter - Seismic w/ Water

Terracon Consultants, Inc. - MDH

\Slope Below Gombu Shelter_lower.gsd

No.	FS
1	1.099
2	1.112
3	1.132
4	1.139
5	1.157
6	1.169
7	1.190
8	1.199
9	1.202
10	1.208

Soil No.		Moist Wt (pcf)	Sat Wt (pcf)	c (psf)	Phi (deg)	ru (ratio)	Pconst (psf)	Piez Surf No.	Soil Options
1	Surficial Sand - Loose	75.0	80.0	200.0	45.0	0.000	0.0	1	
2	GP w/ Sand - med. dense	110.0	120.0	0.0	36.0	0.000	0.0	1	
3	Sand - loose	75.0	80.0	100.0	40.0	0.000	0.0	0	
4	GP - med dense to dense	115.0	125.0	0.0	38.0	0.000	0.0	0	
5	Upper Rock Layer	140.0	141.0	2500.0	0.0	0.000	0.0	0	



GEOSTASE FS = 1.099

Simplified Bishop Method

kh = 0.09000

GEOSTASE® by GREGORY GEOTECHNICAL SOFTWARE

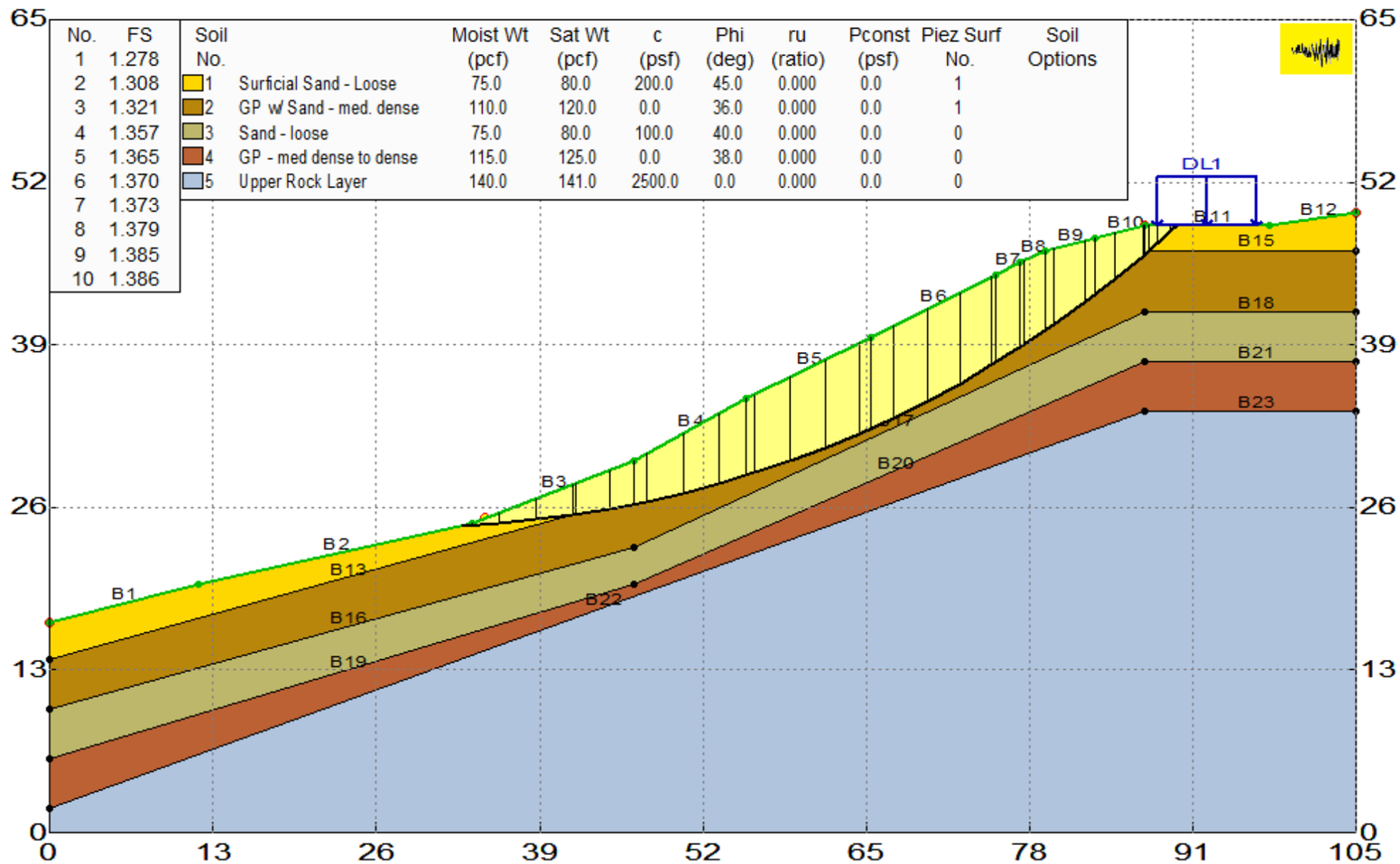
GEOSTASE Slope Stability Analysis

Project Manager: MDH	Project No. C4205024	Terracon	Seismic with Water Condition
Drawn by: MDH	Scale: NA	1392 13th Ave. SW, Great Falls, MT	Figure
Checked by: BJW	File Name:	PH. (406) 453-5400 FAX. (406) 761-6655	**Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area**
Approved by: BJW	Date: 12/23/2020		5

Camp Muir Developed Area
South Slope - Below Gombu Shelter - Seismic w/ Building

Terracon Consultants, Inc. - MDH

\Slope Below Gombu Shelter_lower.gsd



No.	FS
1	1.278
2	1.308
3	1.321
4	1.357
5	1.365
6	1.370
7	1.373
8	1.379
9	1.385
10	1.386

Soil No.		Moist Wt (pcf)	Sat Wt (pcf)	c (psf)	Phi (deg)	ru (ratio)	Pconst (psf)	Piez Surf No.	Soil Options
1	Surficial Sand - Loose	75.0	80.0	200.0	45.0	0.000	0.0	1	
2	GP w/ Sand - med. dense	110.0	120.0	0.0	36.0	0.000	0.0	1	
3	Sand - loose	75.0	80.0	100.0	40.0	0.000	0.0	0	
4	GP - med dense to dense	115.0	125.0	0.0	38.0	0.000	0.0	0	
5	Upper Rock Layer	140.0	141.0	2500.0	0.0	0.000	0.0	0	

GEOSTASE FS = 1.278

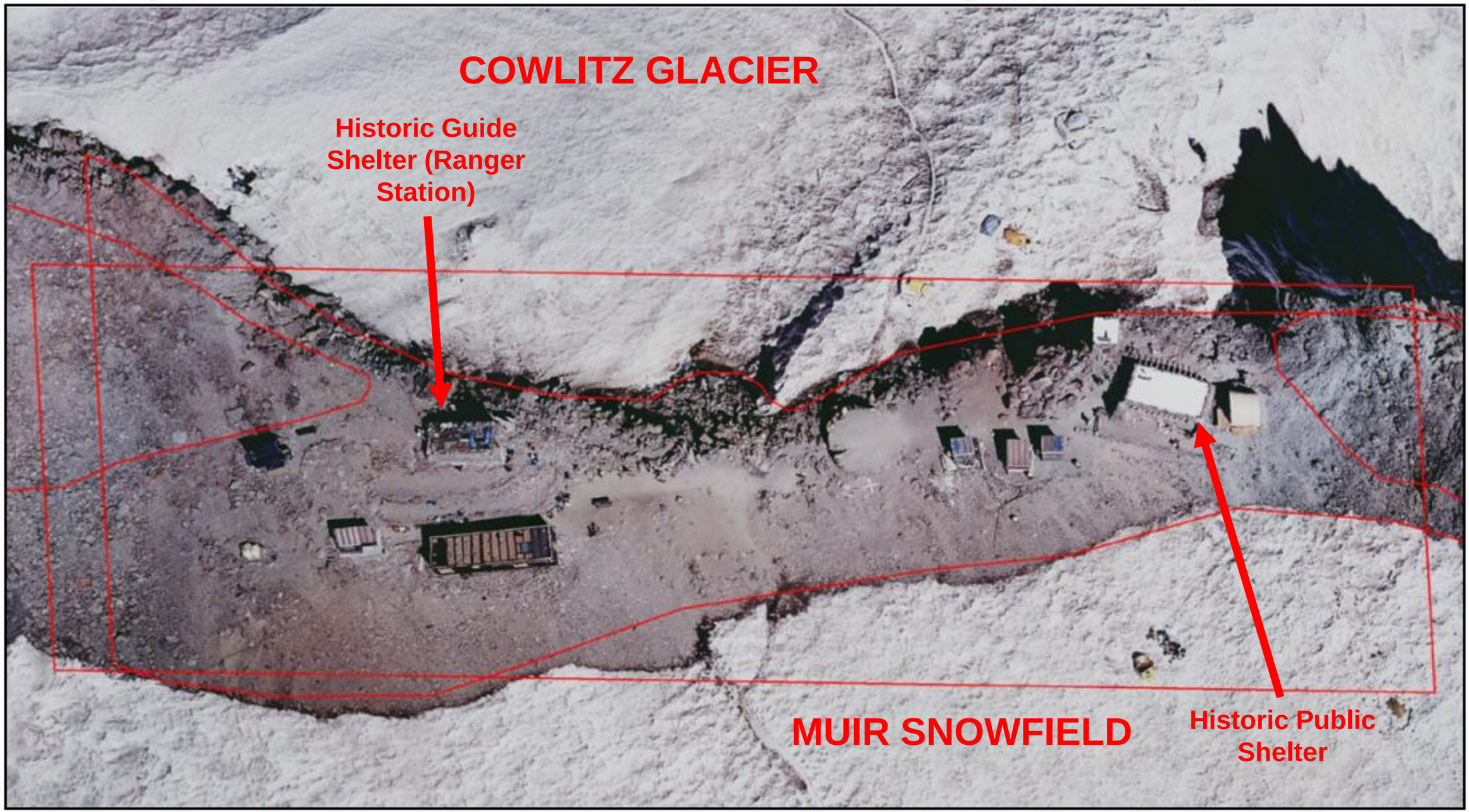
Simplified Bishop Method

kh = 0.18000

GEOSTASE® by GREGORY GEOTECHNICAL SOFTWARE

GEOSTASE Slope Stability Analysis

Project Manager: MDH	Project No. C4205024	Terracon	Seismic with Building Condition
Drawn by: MDH	Scale: NA	1392 13th Ave. SW, Great Falls, MT	Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area
Checked by: BJW	File Name:	PH. (406) 453-5400 FAX. (406) 761-6655	Figure 6
Approved by: BJW	Date: 12/23/2020		



N

The above image is reproduced from the "NPS Camp Muir Rehabilitation Plan 2012". The Red rectangular outline denotes the wilderness boundary, and the irregular outline Denotes the Camp Muir historic district and sensitive vegetation boundaries.

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Project Manager: BJW	Project No. C4205024
Drawn by: BJW	Scale: N.T.S.
Checked by: MDH	File Name:
Approved by: MDH	Date: 12/23/2020

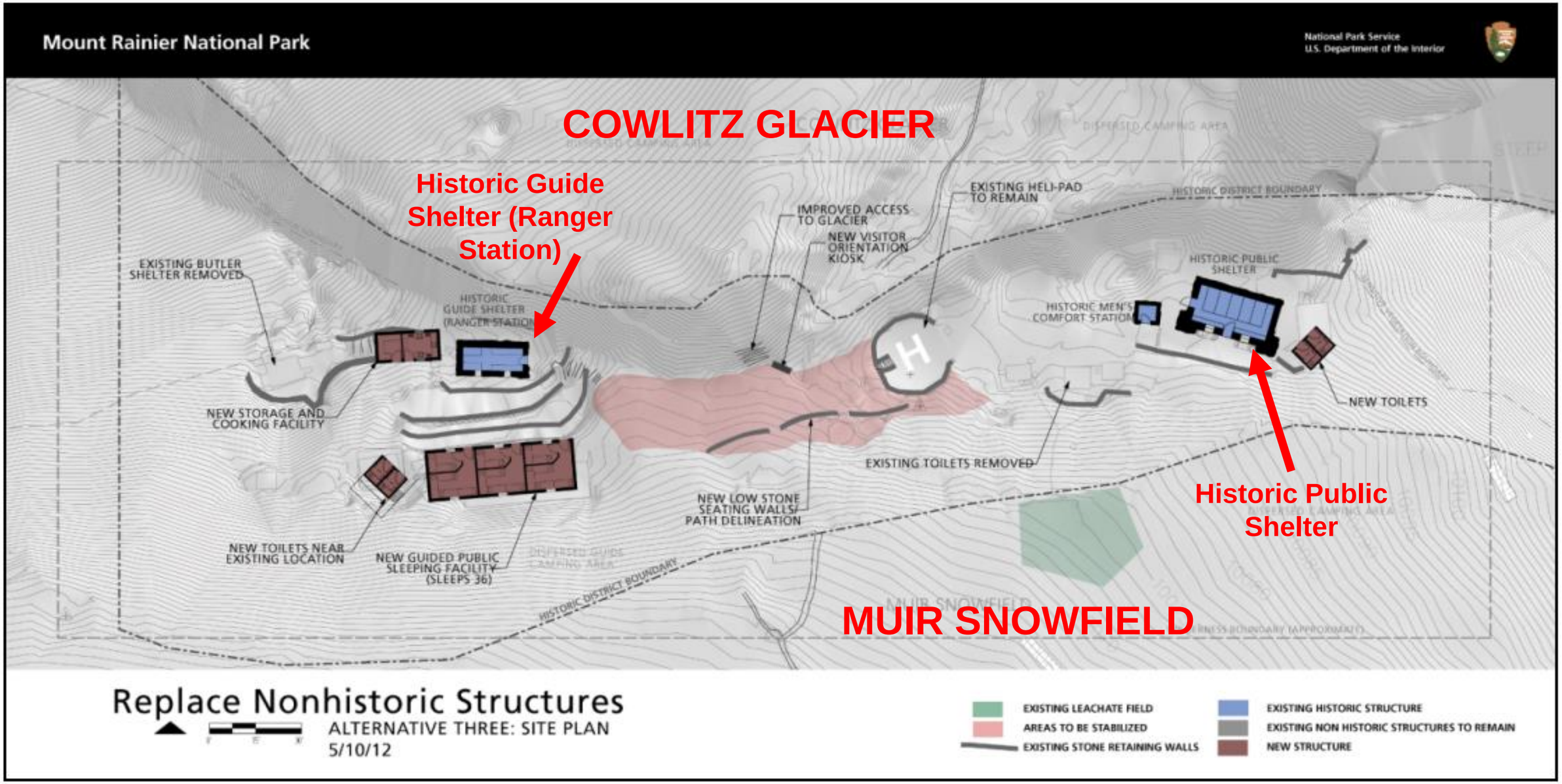
Terracon

1392 13th Ave. SW, Great Falls, MT
PH. (406) 453-5400 FAX. (406) 761-6655

Aerial View of Camp Muir, 2002

Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area

Figure 7

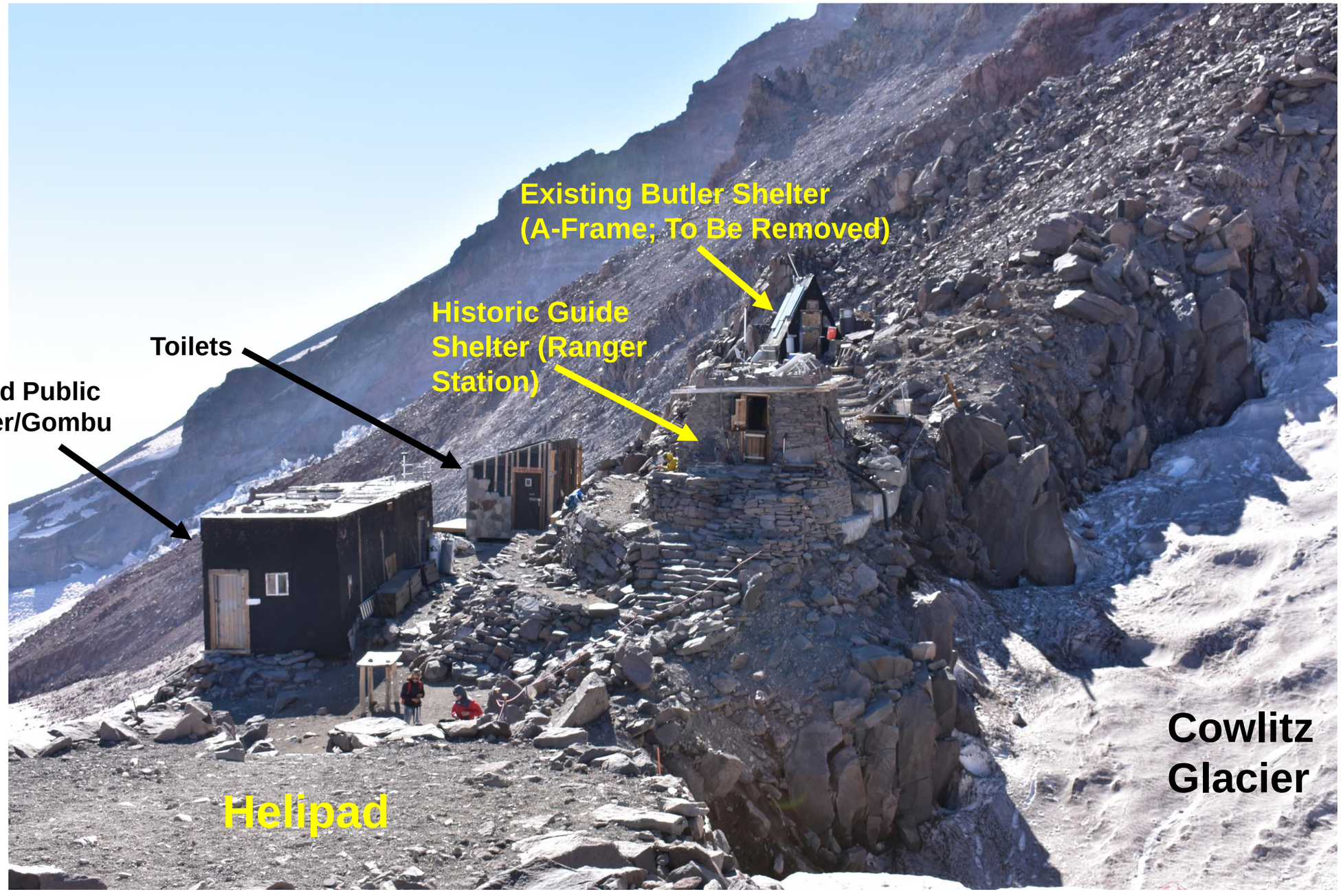


N

The above image is reproduced from the "NPS Camp Muir Rehabilitation Plan 2012".

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Project Manager: BJW	Project No. C4205024	Terracon	Camp Muir Rehabilitation Plan Selected Alternative
Drawn by: BJW	Scale: N.T.S.	1392 13th Ave. SW, Great Falls, MT	Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area
Checked by: MDH	File Name:	PH. (406) 453-5400 FAX. (406) 761-6655	Figure 8
Approved by: MDH	Date: 12/23/2020		



N

The above image taken by Terracon 17 August 2020.

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Project Manager: BJW	Project No. C4205024	Terracon	Camp Muir, looking west over helipad to Historic Guide Shelter (Ranger Station)
Drawn by: BJW	Scale: N.T.S.	1392 13th Ave. SW, Great Falls, MT	Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area
Checked by: MDH	File Name:	PH. (406) 453-5400 FAX. (406) 761-6655	Figure 9
Approved by: MDH	Date: 12/23/2020		

N

The above image taken by Terracon 17 August 2020.

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

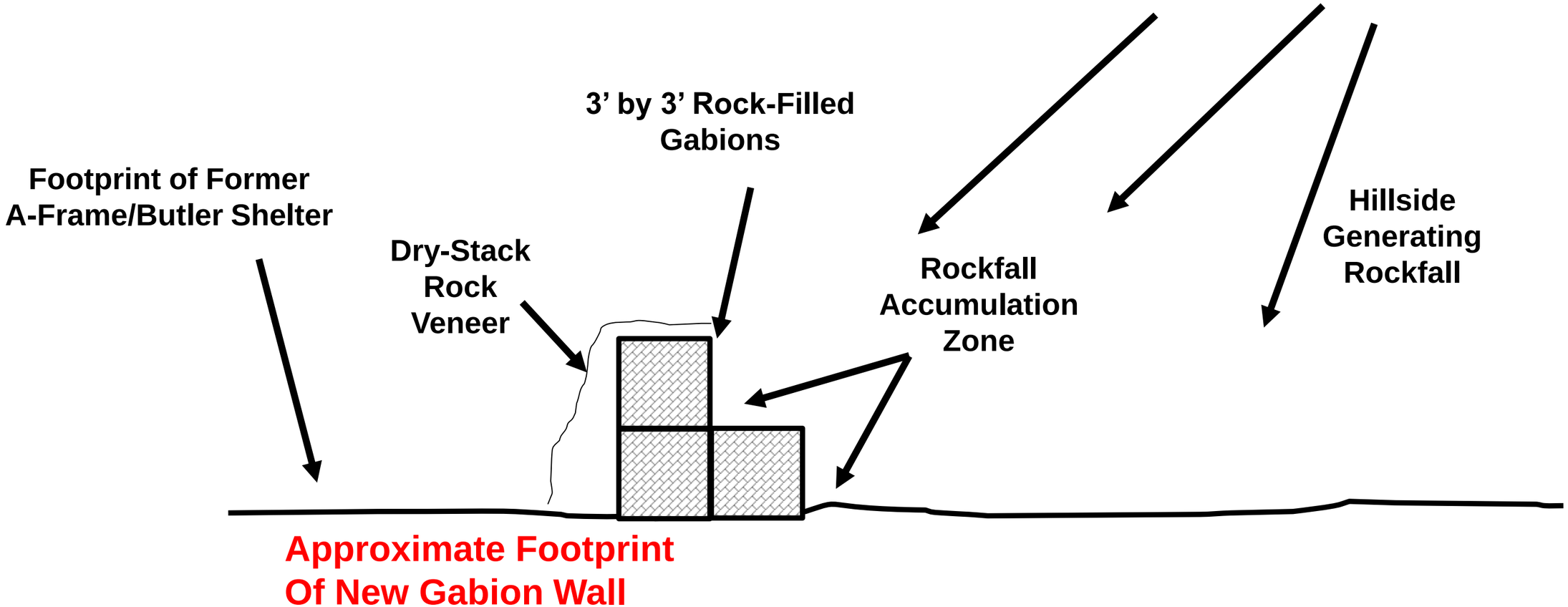
Project Manager: BJW	Project No. C4205024	Terracon	Camp Muir, looking west from helipad; Geohazard Issues
Drawn by: BJW	Scale: N.T.S.	1392 13th Ave. SW., Great Falls, MT	Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area
Checked by: MDH	File Name:	PH. (406) 453-5400 FAX. (406) 761-6655	Figure 10
Approved by: MDH	Date: 12/23/2020		

N

The above image taken by Terracon 17 August 2020.

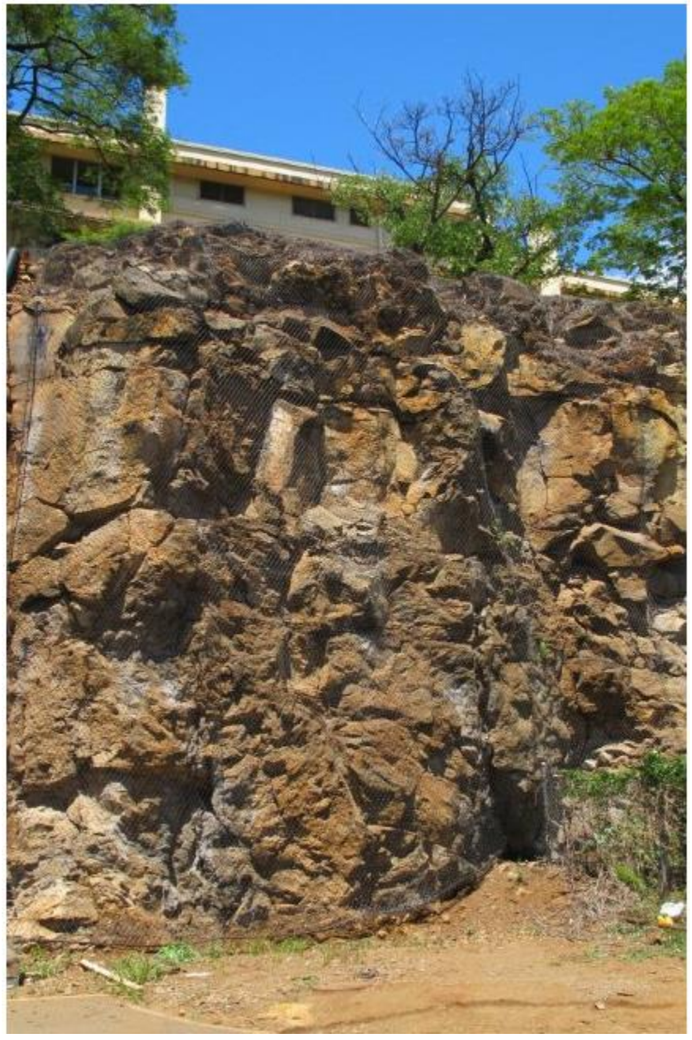
DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Project Manager: BJW	Project No. C4205024	Terracon	Camp Muir, looking west from helipad; Upper Rockfall Generation Site
Drawn by: BJW	Scale: N.T.S.	1392 13th Ave. SW, Great Falls, MT	Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area
Checked by: MDH	File Name:	PH. (406) 453-5400 FAX. (406) 761-6655	Figure 11
Approved by: MDH	Date: 12/23/2020		

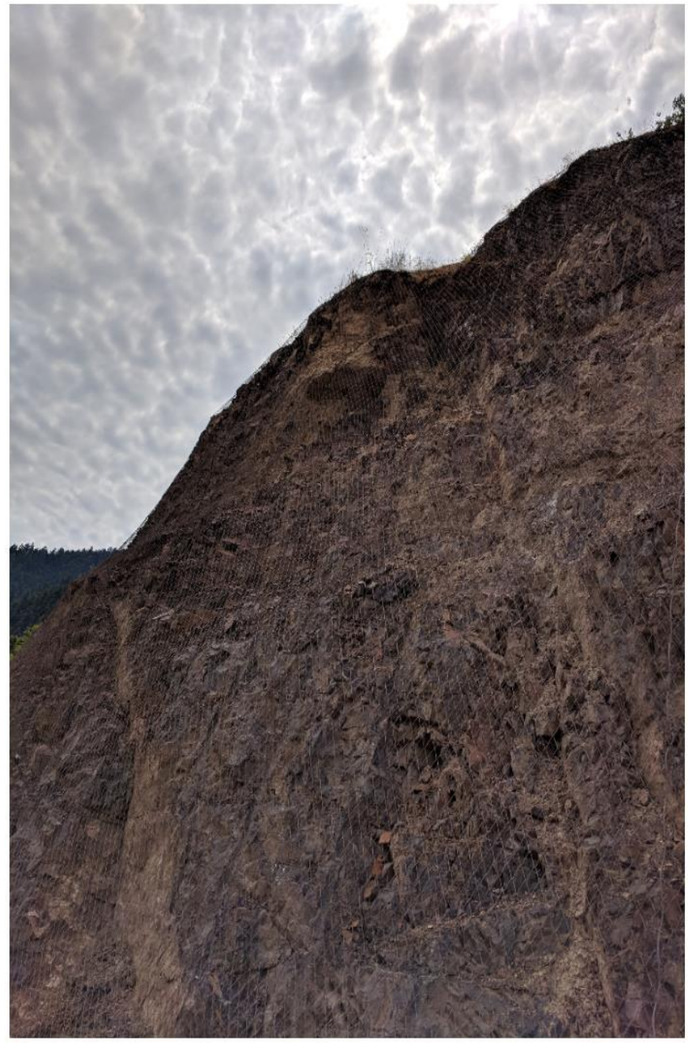


With this rockfall mitigation alternative, a gabion rockfall accumulation wall would be constructed at the upper reaches of the western-most rockfall generation zone potentially impacting the new storage and Cooking Facility and the Historic Guide Shelter (Ranger Station).

Project Manager: BJW	Project No. C4205024	Terracon	Camp Muir, looking west from helipad; Upper Rockfall Mitigation Alternative 1
Drawn by: BJW	Scale: N.T.S.	1392 13th Ave. SW, Great Falls, MT	Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area
Checked by: MDH	File Name:	PH. (406) 453-5400 FAX. (406) 761-6655	Figure 12
Approved by: MDH	Date: 12/23/2020		



Deltax Mesh from 25 Feet



Dyed Tecco Mesh from 50 Feet

Spider Mesh from 100 Feet +

Project Manager: BJW	Project No. C4205024	Terracon 1392 13th Ave. SW, Great Falls, MT PH. (406) 453-5400 FAX. (406) 761-6655	Example imagery of the visual appearance rockfall drape mesh stabilization fabric	Figure
Drawn by: BJW	Scale: N.T.S.		Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area	13
Checked by: MDH	File Name:			
Approved by: MDH	Date: 12/23/2020			

N

The above image taken by Terracon 17 August 2020

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Project Manager: BJW	Project No. C4205024
Drawn by: BJW	Scale: N.T.S.
Checked by: MDH	File Name:
Approved by: MDH	Date: 12/23/2020

Terracon

1392 13th Ave. SW, Great Falls, MT

PH. (406) 453-5400 FAX. (406) 761-6655

Camp Muir, looking south from the Cowlitz Glacier to the undermined Ranger Station foundation

Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area

Figure

14

N

The above image taken by Terracon 17 August 2020

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Project Manager: BJW	Project No. C4205024
Drawn by: BJW	Scale: N.T.S.
Checked by: MDH	File Name:
Approved by: MDH	Date: 12/23/2020

Terracon

1392 13th Ave. SW, Great Falls, MT

PH. (406) 453-5400 FAX. (406) 761-6655

Camp Muir, looking south from the Cowlitz Glacier at the effects of glaciation impacting undermining of the Ranger Station

Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area

Figure

15

N

The above image taken by Terracon 17 August 2020

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Project Manager: BJW	Project No. C4205024
Drawn by: BJW	Scale: N.T.S.
Checked by: MDH	File Name:
Approved by: MDH	Date: 12/23/2020

Terracon

1392 13th Ave. SW, Great Falls, MT
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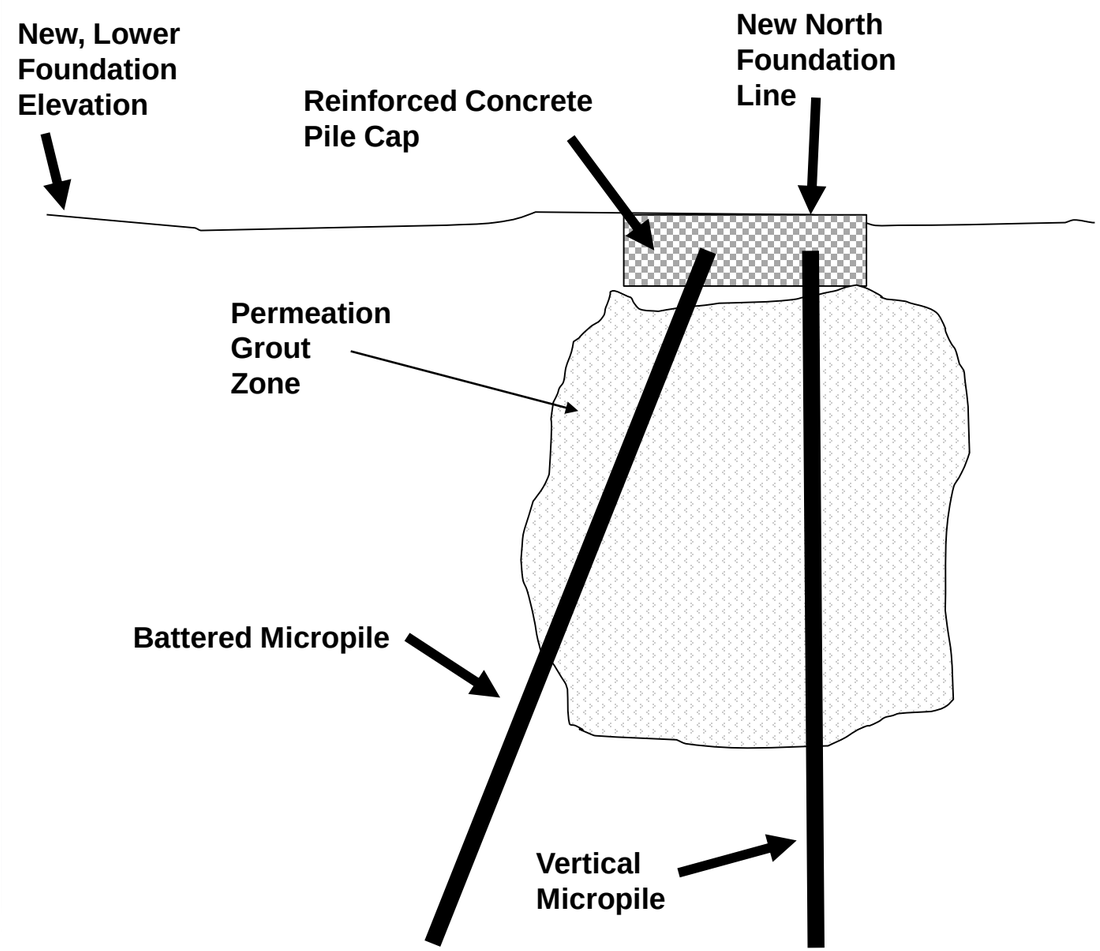
Camp Muir, looking south to the Ranger Station from the Cowlitz Glacier to undermined foundation

Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area

Figure 16

New North Wall Foundation Design Concept



The previous three exhibits indicate the degree of undermining and erosion beneath the previous foundation repairs made at the Ranger Station's north foundation wall. Little, if any, of the previous foundation patches (the light gray colored concrete indicated by the yellow arrows) remains in contact with the foundation subgrade, and our observations indicate that part of the original foundation is again being undermined, limiting the current foundation life.

N

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

The above image taken by Terracon 17 August 2020

Project Manager: BJW	Project No. C4205024	Terracon	Camp Muir, looking west to the Ranger Station from the helipad
Drawn by: BJW	Scale: N.T.S.	1392 13th Ave. SW, Great Falls, MT	Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area
Checked by: MDH	File Name:	PH. (406) 453-5400 FAX. (406) 761-6655	Figure 17
Approved by: MDH	Date: 12/23/2020		

The above image taken by Terracon 17 August 2020

N

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Project Manager: BJW	Project No. C4205024	Terracon	Camp Muir, looking east from Cowlitz Glacier; rockfall and potential toppling issues
Drawn by: BJW	Scale: N.T.S.	1392 13th Ave. SW, Great Falls, MT	Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area
Checked by: MDH	File Name:	PH. (406) 453-5400 FAX. (406) 761-6655	Figure 18
Approved by: MDH	Date: 12/23/2020		

N

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

The above images taken by Terracon 17 August 2020

Project Manager: BJW	Project No. C4205024
Drawn by: BJW	Scale: N.T.S.
Checked by: MDH	File Name:
Approved by: MDH	Date: 12/23/2020

Terracon

1392 13th Ave. SW, Great Falls, MT

PH. (406) 453-5400 FAX. (406) 761-6655

Camp Muir, looking east from Historic Men's Comfort Station; potential rock toppling/sliding

Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area

Figure 19

REFERENCES

(1) 2018 International Building Code (IBC); Chapter 18 – Soils and Foundations
(2) United States Geologic Survey (USGS) website reviewed;
   a. "Mount Rainier National Park Geology Detail Description and Maps"
   b. "Mount Rainier, Glaciers Help to Shape Mount Rainier"
(3) The Geologic Story of Mount Rainier; Dwight R. Crandell; Geologic Survey Bulletin 1292 (1969)
(4) Geologic Map and Section of Mount Rainier National Park Washington; Richard S. Fiske, C.A. Hopson, and Aaron C. Waters, USGS Map I-432 (1964)
(5) Camp Muir Geologic Reconnaissance Visit Summary Report, Geotechnical; The Pullman Group, (June 2002)
(6) Preliminary Stability Analyses Report for Gombu Shelter; Western Geotechnical Consultants, Inc., (October 23, 2011)
(7) Design Drawing Set for Camp Muir West Toilet; (June 29, 2018)
(8) Change in Glacial Extent at Mount Rainier National Park from 1896 to 2015; Scott R. Beason, National Park Services (June 2017)
(9) Dynamic Cone for Shallow In-Situ Penetration Testing, Vane Shear and Cone Penetration Resistance Testing of In-Situ Soils; George F. Sowers and Charles S. Hedges, ASTM STP 399 (1966), pg. 29

ATTACHMENTS

PHOTOGRAPHY LOG

View of Camp Muir from west above Ranger Station

View south of talus slope from above Butler Shelter

View of slope below Butler Shelter

View of new West Toilet structure on slope

View of south slope, below Gombu Shelter, talus in foreground and pumice/sand matrix below

View of south slope below Butler, West Toilet, and Gombu from Muir Snowfield

View of south slope, looking west from below Public Shelter

View of north face of Ranger Station

View looking west middle of saddle, surveying in geophysical array

View of east slope above East Toilet, Public Shelter in foreground

View of Public Shelter from below

View of north face of Ranger Station

View looking west from Helipad area, note the north ridge exposure to Cowlitz Glacier

Wider view from above

Bedrock ridge north side of Ranger Station looking west

View of broken rock west of Ranger Station, results of glacial action

Breccia outcrop north and west of Historic Men's Comfort Station

View of Historic Men's Comfort Station and Public Shelter

View of north side of Ranger Station, undermined concrete

View of north side of Ranger Station, undermined concrete from below

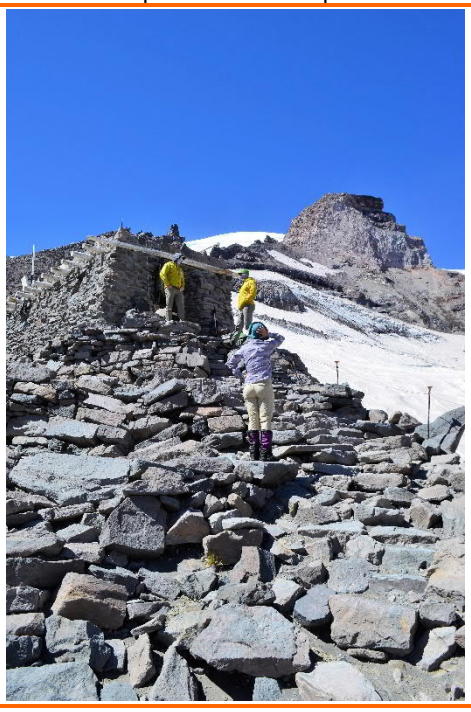
View of north side of Ranger Station, undermined concrete loss of contact with moraine till

View of previous concrete repair loss of contact

Separation of repair concrete from rock at Ranger Station	Southwest corner of Ranger Station, no cracking
Entrance to Ranger Station, jamb separation	Closeup of jamb separation at Ranger Station

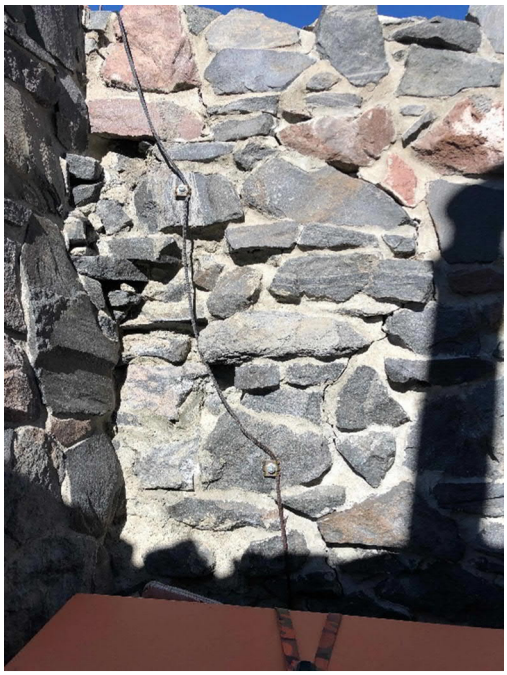
Close up of rock/lintel separation

Northeast corner of Ranger Station



View of Ranger Station and retaining walls

View of Public Shelter lintel

	
View of west pilaster of Public Shelter, shear crack	View behind east pilaster at Public Shelter, shear crack
Northwest corner of Public Shelter, shear crack	View of south wall interior at Public Shelter, door lintel cracking

Test Pit (TP-01)	Frozen matrix from base of TP-01
Test Pit (TP-02), uniform pumice	Base of TP-02

SITE LOCATION AND EXPLORATION PLANS

Contents:

Site Location Plan
Exploration Plan with Project Overlay
Geohazards and Developable Area Map

Note: All attachments are one page unless noted above.

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

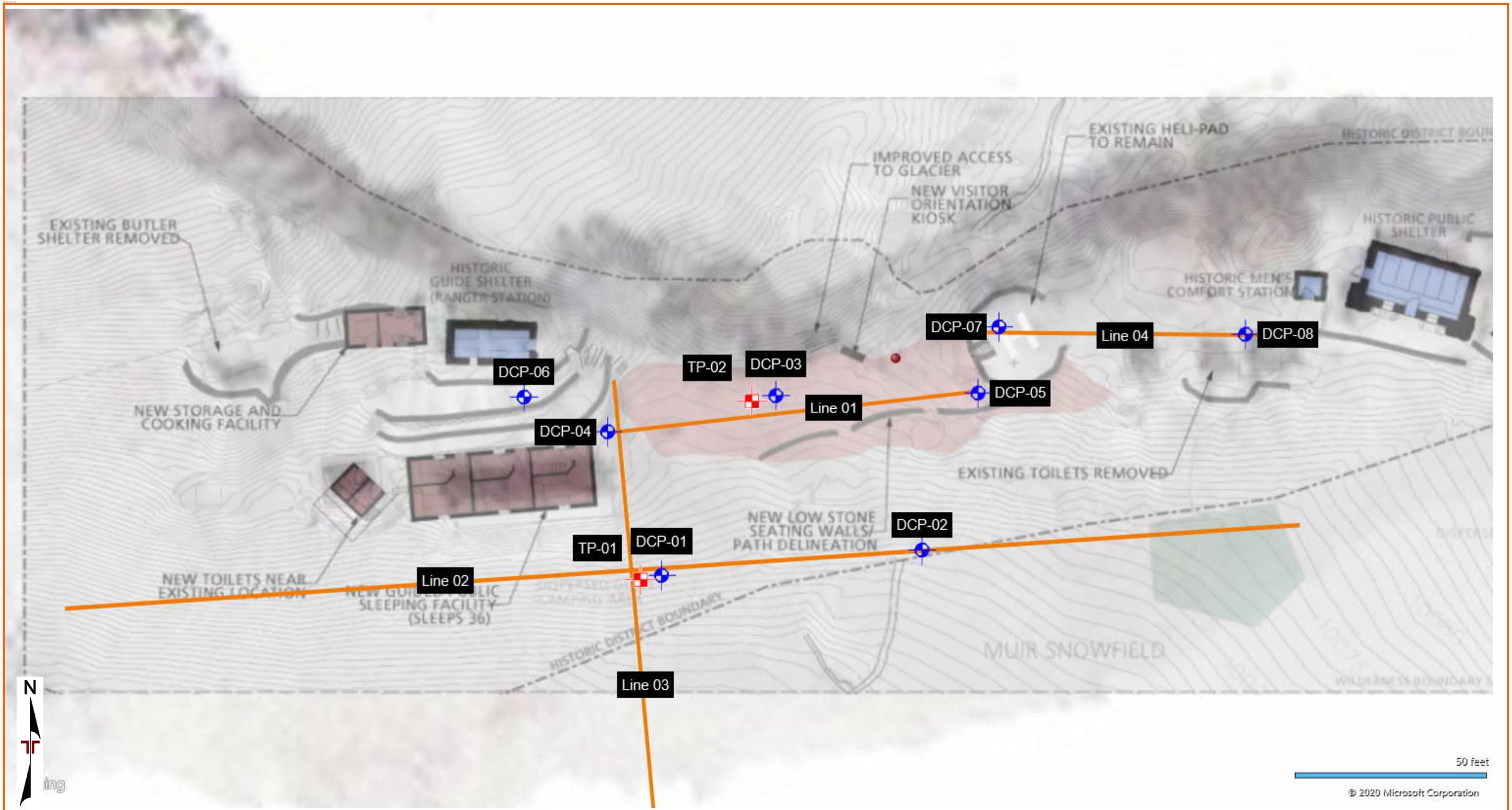


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

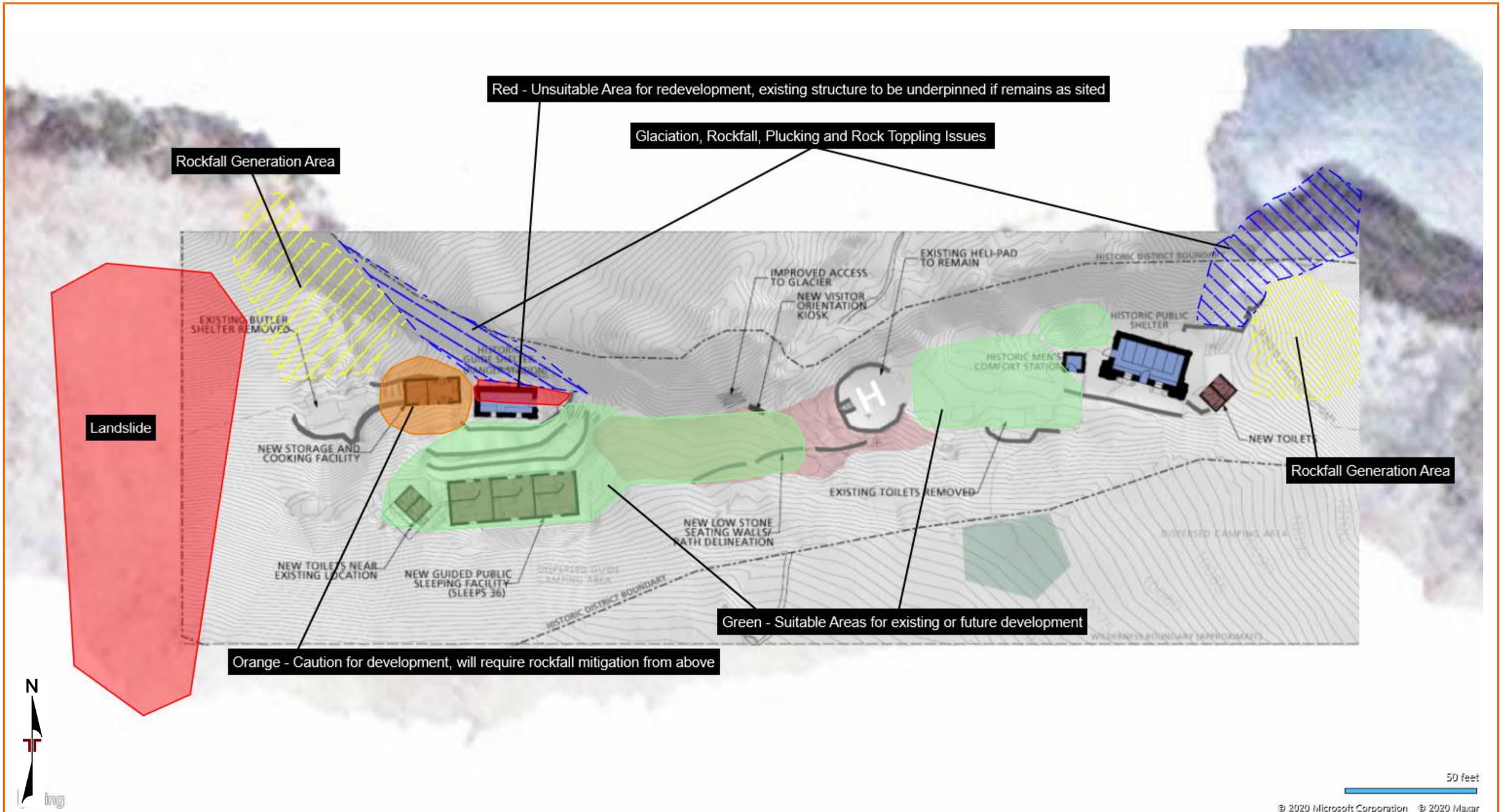


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

EXPLORATION RESULTS

Contents:

Test Pit Logs (TP-01 and TP-02)
DCP Summary Sheets (11 pages)
Shear-Wave Velocity Profiles (4 pages)
Grain Size Distribution
Direct Shear

Note: All attachments are one page unless noted above.

TEST PIT LOG NO. TP-01

PROJECT: Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area

CLIENT: DJ&A PC Missoula, MT

SITE: Mount Rainier National Park Paradise, WA

LOCATION See Exploration Plan

Latitude: 46.8353° Longitude: -121.7328°

Approximate Surface Elev.: 10067 (Ft.) +/-

DEPTH	ELEVATION (Ft.)	Description
0.2	10067+/-	**SURFACE COBBLES (TALUS)**
2.0	10065+/-	**POORLY-GRADED SAND WITH GRAVEL**, dark brown/black, very loose to loose, moist, pumice sand, angular sand particles and gravel inclusions, trace cobbles, some reddish tan sand sized particles (SP)
3.0	10064+/-	**POORLY-GRADED GRAVEL WITH SAND**, brown/black, medium dense to dense, moist to very moist, angular to subangular gravels, coarse-grained pumice sand (GP)
3.2	10064+/-	**PERMAFROST, POORLY-GRADED SAND WITH GRAVEL MATRIX**, dark brown/black, frozen (dense), angular to subangular gravels, coarse-grained pumice sand (SP)

Test Pit Refusal at 3.2 Feet

GRAPHIC LOG	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (tsf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS: LL-PL-PI
	1									
	2									
	3									

Stratification lines are approximate. In-situ, the transition may be gradual.

Advancement Method:
Hand Excavated

Abandonment Method:
Hand excavated test pit backfilled with spoils after completion

Elevations were interpolated from a topographic site plan.

Notes:

WATER LEVEL OBSERVATIONS

none

encountered

Terracon
1392 13th Ave SW
Great Falls, MT

Test Pit Started: 08-18-2020

Test Pit Completed: 08-18-2020

Excavator:

Operator:

Project No.: C4205024

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL C4205024 TITLE I - EVALUAT.GPJ TERRACON_DATATEMPLATE.GDT 10/8/20

TEST PIT LOG NO. TP-02

PROJECT: Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area

CLIENT: DJ&A PC Missoula, MT

SITE: Mount Rainier National Park Paradise, WA

LOCATION See Exploration Plan

Latitude: 46.8355° Longitude: -121.7326°

Approximate Surface Elev.: 10080 (Ft.) +/-

GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	STRENGTH TEST: TEST TYPE	STRENGTH TEST: COMPRESSIVE STRENGTH (tsf)	STRENGTH TEST: STRAIN (%)	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS: LL-PL-PI
	PUMICE, tan, very loose to loose, moist, maximum particle size approximately 1-inch, fairly uniform particle size from 1/2 to 1-inch throughout depth of test pit, angular (GP)		1									
	2.0	10078+/-	2									
	2.1 **PERMAFROST, PUMICE MATRIX**, tan to brown, frozen (dense), angular sand and gravel matrix (GP)	10078+/-										
	Test Pit Refusal at 2.1 Feet											

Stratification lines are approximate. In-situ, the transition may be gradual.

Advancement Method:
Hand Excavated

Abandonment Method:
Hand excavated test pit backfilled with spoils after completion

Elevations were interpolated from a topographic site plan.

Notes:

WATER LEVEL OBSERVATIONS

none

encountered

Terracon
1392 13th Ave SW
Great Falls, MT

Test Pit Started: 08-18-2020

Test Pit Completed: 08-18-2020

Excavator:

Operator:

Project No.: C4205024

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GEO SMART LOG-NO WELL C4205024 TITLE I - EVALUAT.GPJ TERRACON_DATATEMPLATE.GDT 10/8/20

Dynamic Cone Penetration (DCP-01)

ASTM STP 399

Terracon

1392 13th Ave SW, Great Falls, Montana 59404 (406) 453-5400

Project: Evaluate Geotechnical and Stuctural Stability of Camp Muir Developed Area
Job No: C4205024
Date: 8/17/2020

Exploration Point: DCP-01 Tested By: MDH/JP
Location: South slope below Gombu Shelter
Approximately 30' S & 13' E of SE Corner of Gombu (See Exploration Plan)

*Correlations based on Sowers and Hedges published values (ASTM STP 399)

Curve A Virgin Piedmont Soils
SPT "N" Value = 0.4611*DCP + 2.5994

Curve B 95% compacted
SPT "N" value = DCP

DCP READING Blows per increment (1-3/4")	Depth inches	Depth feet	SPT "N" Value A	SPT "N" Value B	Average "N" Value	SPT Correlated Relative Density
2	1.75	0.1	4	2	3	very loose
2	3.5	0.3	4	2	3	very loose
2	5.25	0.4	4	2	3	very loose
2	7	0.6	4	2	3	very loose
3	8.75	0.7	4	3	3	very dense
5	10.5	0.9	5	5	5	loose
3	12.25	1.0	4	3	3	very loose
2	14	1.2	4	2	3	very loose
5	15.75	1.3	5	5	5	loose
6	17.5	1.5	5	6	6	loose
8	19.25	1.6	6	8	7	loose
8	21	1.8	6	8	7	loose
12	22.75	1.9	8	12	10	medium dense
42	24.5	2.0	22	42	32	dense
50	26.25	2.2	26	50	38	dense / frozen

*refusal in permafrost

Average DCP Penetration Reading 10
Average SPT "N" Value 9

Dynamic Cone Penetration (DCP-02)

ASTM STP 399

Terracon

1392 13th Ave SW, Great Falls, Montana 59404 (406) 453-5400

Project: Evaluate Geotechnical and Stuctural Stability of Camp Muir Developed Area

Job No: C4205024

Date: 8/17/2020

Exploration Point: DCP-02 Tested By: MDH/JP

Location: South slope on trail access to Camp Muir Site

See Exploration Plan

*Correlations based on Sowers and Hedges published values (ASTM STP 399)

Curve A Virgin Piedmont Soils

SPT "N" Value = 0.4611*DCP + 2.5994

Curve B 95% compacted

SPT "N" value = DCP

DCP READING Blows per increment (1-3/4")	Depth inches	Depth feet	SPT "N" Value A	SPT "N" Value B	Average "N" Value	SPT Correlated Relative Density
14	1.75	0.15	9	14	12	medium dense
10	3.5	0.29	7	10	9	loose
9	5.25	0.44	7	9	8	loose
9	7	0.58	7	9	8	loose
12	8.75	0.73	8	12	10	medium dense
11	10.5	0.88	8	11	9	loose
8	12.25	1.02	6	8	7	loose
7	14	1.17	6	7	6	loose
7	15.75	1.31	6	7	6	loose
7	17.5	1.46	6	7	6	loose
7	19.25	1.60	6	7	6	loose
7	21	1.75	6	7	6	loose
9	22.75	1.90	7	9	8	loose
6	24.5	2.04	5	6	6	loose
8	26.25	2.19	6	8	7	loose
6	28	2.33	5	6	6	loose
6	29.75	2.48	5	6	6	loose

Dynamic Cone Penetration (DCP-02)

ASTM STP 399

Terracon

1392 13th Ave SW, Great Falls, Montana 59404 (406) 453-5400

Project: Evaluate Geotechnical and Stuctural Stability of Camp Muir Developed Area

Job No: C4205024

Date: 8/17/2020

Drill Hole: DCP-02 Tested By: MDH/JP

Location: South slope on trail access to Camp Muir Site

See Exploration Plan

*Correlations based on Sowers and Hedges published values (ASTM STP 399)

Curve A Virgin Piedmont Soils

SPT "N" Value = 0.4611*DCP + 2.5994

Curve B 95% compacted

SPT "N" value = DCP

Continued

DCP READING Blows per increment (1-3/4")	Depth inches	Depth feet	SPT "N" Value A	SPT "N" Value B	Average "N" Value	SPT Correlated Relative Density
6	31.5	2.63	5	6	6	loose
8	33.25	2.77	6	8	7	loose
8	35	2.92	6	8	7	loose
9	36.75	3.06	7	9	8	loose
9	38.5	3.21	7	9	8	loose
9	40.25	3.35	7	9	8	loose
9	42	3.50	7	9	8	loose
7	43.75	3.65	6	7	6	loose
7	45.5	3.79	6	7	6	loose
5	47.25	3.94	5	5	5	loose
7	49	4.08	6	7	6	loose
6	50.75	4.23	5	6	6	loose
8	52.5	4.38	6	8	7	loose
7	54.25	4.52	6	7	6	loose
8	56	4.67	6	8	7	loose
12	57.75	4.81	8	12	10	medium dense
12	59.5	4.96	8	12	10	medium dense
18	61.25	5.10	11	18	14	medium dense
25	63	5.25	14	25	20	medium dense

* no refusal, drove all rod available

Average DCP Penetration Reading 9

Average SPT "N" Value 8

Dynamic Cone Penetration (DCP-03)

ASTM STP 399

Terracon

1392 13th Ave SW, Great Falls, Montana 59404 (406) 453-5400

Project: Evaluate Geotechnical and Stuctural Stability of Camp Muir Developed Area
Job No: C4205024
Date: 8/17/2020

Exploration Point: DCP-03 Tested By: MDH/JP
Location: Middle of saddle
See Exploration Plan

*Correlations based on Sowers and Hedges published values (ASTM STP 399)

Curve A Virgin Piedmont Soils
SPT "N" Value = 0.4611*DCP + 2.5994

Curve B 95% compacted
SPT "N" value = DCP

DCP READING Blows per increment (1-3/4")	Depth inches	Depth feet	SPT "N" Value A	SPT "N" Value B	Average "N" Value	SPT Correlated Relative Density
5	1.75	0.15	5	5	5	loose
5	3.5	0.29	5	5	5	loose
3	5.25	0.44	4	3	3	very loose
3	7	0.58	4	3	3	very loose
5	8.75	0.73	5	5	5	loose
3	10.5	0.88	4	3	3	very loose
4	12.25	1.02	4	4	4	loose
8	14	1.17	6	8	7	loose
3	15.75	1.31	4	3	3	very loose
6	17.5	1.46	5	6	6	loose
6	19.25	1.60	5	6	6	loose
6	21	1.75	5	6	6	loose
50	22.75	1.90	26	50	38	dense / frozen

*refusal in permafrost

Average DCP Penetration Reading 8
Average SPT "N" Value 7

Dynamic Cone Penetration (DCP-04)

ASTM STP 399

Terracon

1392 13th Ave SW, Great Falls, Montana 59404 (406) 453-5400

Project: Evaluate Geotechnical and Stuctural Stability of Camp Muir Developed Area
Job No: C4205024
Date: 8/17/2020

Exploration Point: DCP-04 Tested By: MDH/JP
Location: West end of saddle, 5'N & 5' E of NE corner of Gombu Shelter
See Exploration Plan

*Correlations based on Sowers and Hedges published values (ASTM STP 399)

Curve A Virgin Piedmont Soils
SPT "N" Value = 0.4611*DCP + 2.5994

Curve B 95% compacted
SPT "N" value = DCP

DCP READING Blows per increment (1-3/4")	Depth inches	Depth feet	SPT "N" Value A	SPT "N" Value B	Average "N" Value	SPT Correlated Relative Density
7	1.75	0.15	6	7	6	loose
5	3.5	0.29	5	5	5	loose
6	5.25	0.44	5	6	6	loose
5	7	0.58	5	5	5	loose
5	8.75	0.73	5	5	5	loose
5	10.5	0.88	5	5	5	loose
7	12.25	1.02	6	7	6	loose
5	14	1.17	5	5	5	loose
5	15.75	1.31	5	5	5	loose
15	17.5	1.46	10	15	12	medium dense
50	19.25	1.60	26	50	38	dense / frozen

*refusal in permafrost

Average DCP Penetration Reading 10
Average SPT "N" Value 9

Dynamic Cone Penetration (DCP-05)

ASTM STP 399

Terracon

1392 13th Ave SW, Great Falls, Montana 59404 (406) 453-5400

Project: Evaluate Geotechnical and Stuctural Stability of Camp Muir Developed Area
Job No: C4205024
Date: 8/17/2020

Exploration Point: DCP-05 Tested By: MDH/JP
Location: East end of saddle, 6' S & 2'E of base of helipad stairs
See Exploration Plan

*Correlations based on Sowers and Hedges published values (ASTM STP 399)

Curve A Virgin Piedmont Soils
SPT "N" Value = 0.4611*DCP + 2.5994

Curve B 95% compacted
SPT "N" value = DCP

DCP READING Blows per increment (1-3/4")	Depth inches	Depth feet	SPT "N" Value A	SPT "N" Value B	Average "N" Value	SPT Correlated Relative Density
2	1.75	0.15	4	2	3	very loose
4	3.5	0.29	4	4	4	loose
3	5.25	0.44	4	3	3	very loose
4	7	0.58	4	4	4	loose
2	8.75	0.73	4	2	3	very loose
2	10.5	0.88	4	2	3	very loose
4	12.25	1.02	4	4	4	loose
4	14	1.17	4	4	4	loose
4	15.75	1.31	4	4	4	loose
12	17.5	1.46	8	12	10	medium dense
50	19.25	1.60	26	50	38	dense / frozen

*refusal in permafrost

Average DCP Penetration Reading 8
Average SPT "N" Value 7

Dynamic Cone Penetration (DCP-06)

ASTM STP 399

Terracon

1392 13th Ave SW, Great Falls, Montana 59404 (406) 453-5400

Project: Evaluate Geotechnical and Stuctural Stability of Camp Muir Developed Area
Job No: C4205024
Date: 8/17/2020

Exploration Point: DCP-06 Tested By: MDH/JP
Location: Middle retaining wall south of Ranger Station
See Exploration Plan

*Correlations based on Sowers and Hedges published values (ASTM STP 399)

Curve A Virgin Piedmont Soils
SPT "N" Value = 0.4611*DCP + 2.5994

Curve B 95% compacted
SPT "N" value = DCP

DCP READING Blows per increment (1-3/4")	Depth inches	Depth feet	SPT "N" Value A	SPT "N" Value B	Average "N" Value	SPT Correlated Relative Density
4	1.75	0.15	4	4	4	loose
5	3.5	0.29	5	5	5	loose
8	5.25	0.44	6	8	7	loose
9	7	0.58	7	9	8	loose
8	8.75	0.73	6	8	7	loose
14	10.5	0.88	9	14	12	medium dense
17	12.25	1.02	10	17	14	medium dense
Refusal	14	1.17				

*Refusal on cobble/boulder within backfill zone

Average DCP Penetration Reading 9
Average SPT "N" Value 8

Dynamic Cone Penetration (DCP-07)

ASTM STP 399

Terracon

1392 13th Ave SW, Great Falls, Montana 59404 (406) 453-5400

Project: Evaluate Geotechnical and Stuctural Stability of Camp Muir Developed Area
Job No: C4205024
Date: 8/17/2020

Exploration Point: DCP-07 Tested By: MDH/JP
Location: Helipad Backfill, 6' E of top of stairs
See Exploration Plan

*Correlations based on Sowers and Hedges published values (ASTM STP 399)

Curve A Virgin Piedmont Soils
SPT "N" Value = 0.4611*DCP + 2.5994

Curve B 95% compacted
SPT "N" value = DCP

DCP READING Blows per increment (1-3/4")	Depth inches	Depth feet	SPT "N" Value A	SPT "N" Value B	Average "N" Value	SPT Correlated Relative Density
9	1.75	0.15	7	9	8	loose
7	3.5	0.29	6	7	6	loose
9	5.25	0.44	7	9	8	loose
9	7	0.58	7	9	8	loose
8	8.75	0.73	6	8	7	loose
11	10.5	0.88	8	11	9	loose
8	12.25	1.02	6	8	7	loose
11	14	1.17	8	11	9	loose
7	15.75	1.31	6	7	6	loose
9	17.5	1.46	7	9	8	loose
16	19.25	1.60	10	16	13	medium dense
11	21	1.75	8	11	9	loose
18	22.75	1.90	11	18	14	medium dense
12	24.5	2.04	8	12	10	medium dense
9	26.25	2.19	7	9	8	loose
11	28	2.33	8	11	9	loose
18	29.75	2.48	11	18	14	medium dense

Dynamic Cone Penetration (DCP-07)

ASTM STP 399

Terracon

1392 13th Ave SW, Great Falls, Montana 59404 (406) 453-5400

Project: Evaluate Geotechnical and Stuctural Stability of Camp Muir Developed Area
Job No: C4205024
Date: 8/17/2020

Drill Hole: DCP-07 Tested By: MDH/JP
Location: Helipad Backfill, 6' E of top of stairs
See Exploration Plan

*Correlations based on Sowers and Hedges published values (ASTM STP 399)

Curve A Virgin Piedmont Soils
SPT "N" Value = 0.4611*DCP + 2.5994

Curve B 95% compacted
SPT "N" value = DCP

Continued

DCP READING Blows per increment (1-3/4")	Depth inches	Depth feet	SPT "N" Value A	SPT "N" Value B	Average "N" Value	SPT Correlated Relative Density
12	31.5	2.63	8	12	10	medium dense
15	33.25	2.77	10	15	12	medium dense
16	35	2.92	10	16	13	medium dense
19	36.75	3.06	11	19	15	medium dense
19	38.5	3.21	11	19	15	medium dense
16	40.25	3.35	10	16	13	medium dense
28	42	3.50	16	28	22	medium dense
24	43.75	3.65	14	24	19	medium dense
18	45.5	3.79	11	18	14	medium dense
25	47.25	3.94	14	25	20	medium dense
28	49	4.08	16	28	22	medium dense
50	50.75	4.23	26	50	38	dense

*Refusal on cobble/boulder in backfill zone, near base of backfill zone

Average DCP Penetration Reading 16
Average SPT "N" Value 13

Dynamic Cone Penetration (DCP-08)

ASTM STP 399

Terracon

1392 13th Ave SW, Great Falls, Montana 59404 (406) 453-5400

Project: Evaluate Geotechnical and Stuctural Stability of Camp Muir Developed Area

Job No: C4205024

Date: 8/17/2020

Exploration Point: DCP-08 Tested By: MDH/BE

Location: East end of Seismic Line 4, near temporary toilets

See Exploration Plan

*Correlations based on Sowers and Hedges published values (ASTM STP 399)

Curve A Virgin Piedmont Soils

SPT "N" Value = 0.4611*DCP + 2.5994

Curve B 95% compacted

SPT "N" value = DCP

DCP READING Blows per increment (1-3/4")	Depth inches	Depth feet	SPT "N" Value A	SPT "N" Value B	Average "N" Value	SPT Correlated Relative Density
16	1.75	0.15	10	16	13	medium dense
11	3.5	0.29	8	11	9	loose
20	5.25	0.44	12	20	16	medium dense
21	7	0.58	12	21	17	medium dense
17	8.75	0.73	10	17	14	medium dense
16	10.5	0.88	10	16	13	medium dense
22	12.25	1.02	13	22	17	medium dense
37	14	1.17	20	37	28	medium dense
24	15.75	1.31	14	24	19	medium dense
26	17.5	1.46	15	26	20	medium dense
28	19.25	1.60	16	28	22	medium dense
24	21	1.75	14	24	19	medium dense
23	22.75	1.90	13	23	18	medium dense
30	24.5	2.04	16	30	23	medium dense
42	26.25	2.19	22	42	32	dense
19	28	2.33	11	19	15	medium dense
23	29.75	2.48	13	23	18	medium dense

Dynamic Cone Penetration (DCP-08)

ASTM STP 399

Terracon

1392 13th Ave SW, Great Falls, Montana 59404 (406) 453-5400

Project: Evaluate Geotechnical and Stuctural Stability of Camp Muir Developed Area
Job No: C4205024
Date: 8/17/2020

Drill Hole: DCP-08 Tested By: MDH/BE
Location: East end of Seismic Line 4, near temporary toilets
See Exploration Plan

*Correlations based on Sowers and Hedges published values (ASTM STP 399)

Curve A Virgin Piedmont Soils
SPT "N" Value = 0.4611*DCP + 2.5994

Curve B 95% compacted
SPT "N" value = DCP

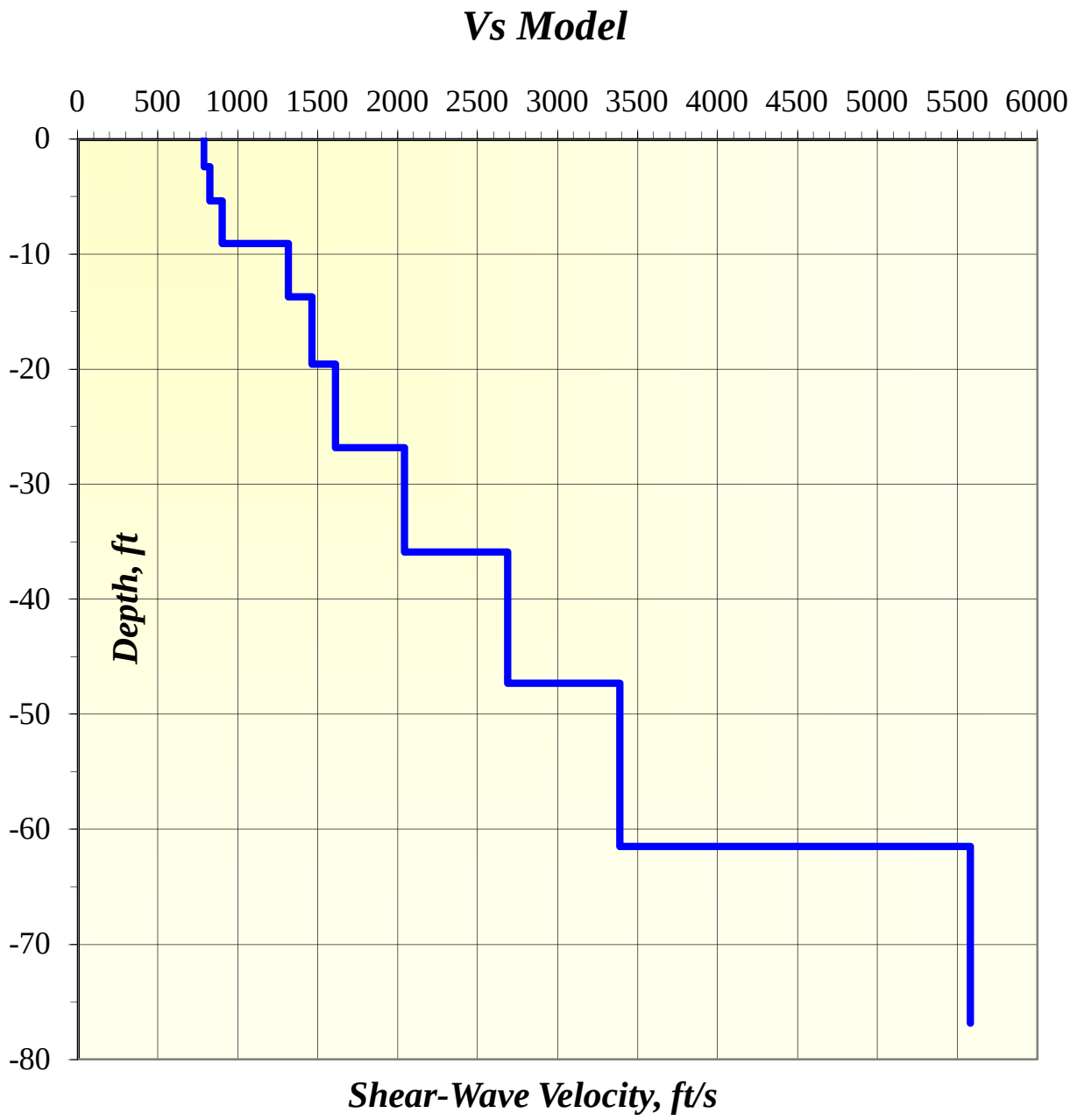
Continued

DCP READING Blows per increment (1-3/4")	Depth inches	Depth feet	SPT "N" Value A	SPT "N" Value B	Average "N" Value	SPT Correlated Relative Density
26	31.5	2.63	15	26	20	medium dense
20	33.25	2.77	12	20	16	medium dense
18	35	2.92	11	18	14	medium dense
18	36.75	3.06	11	18	14	medium dense
32	38.5	3.21	17	32	25	medium dense
18	40.25	3.35	11	18	14	medium dense
22	42	3.50	13	22	17	medium dense
15	43.75	3.65	10	15	12	medium dense
21	45.5	3.79	12	21	17	medium dense
20	47.25	3.94	12	20	16	medium dense
50	49	4.08	26	50	38	dense

*Refusal on cobble/boulder/bedrock

Average DCP Penetration Reading 24
Average SPT "N" Value 18

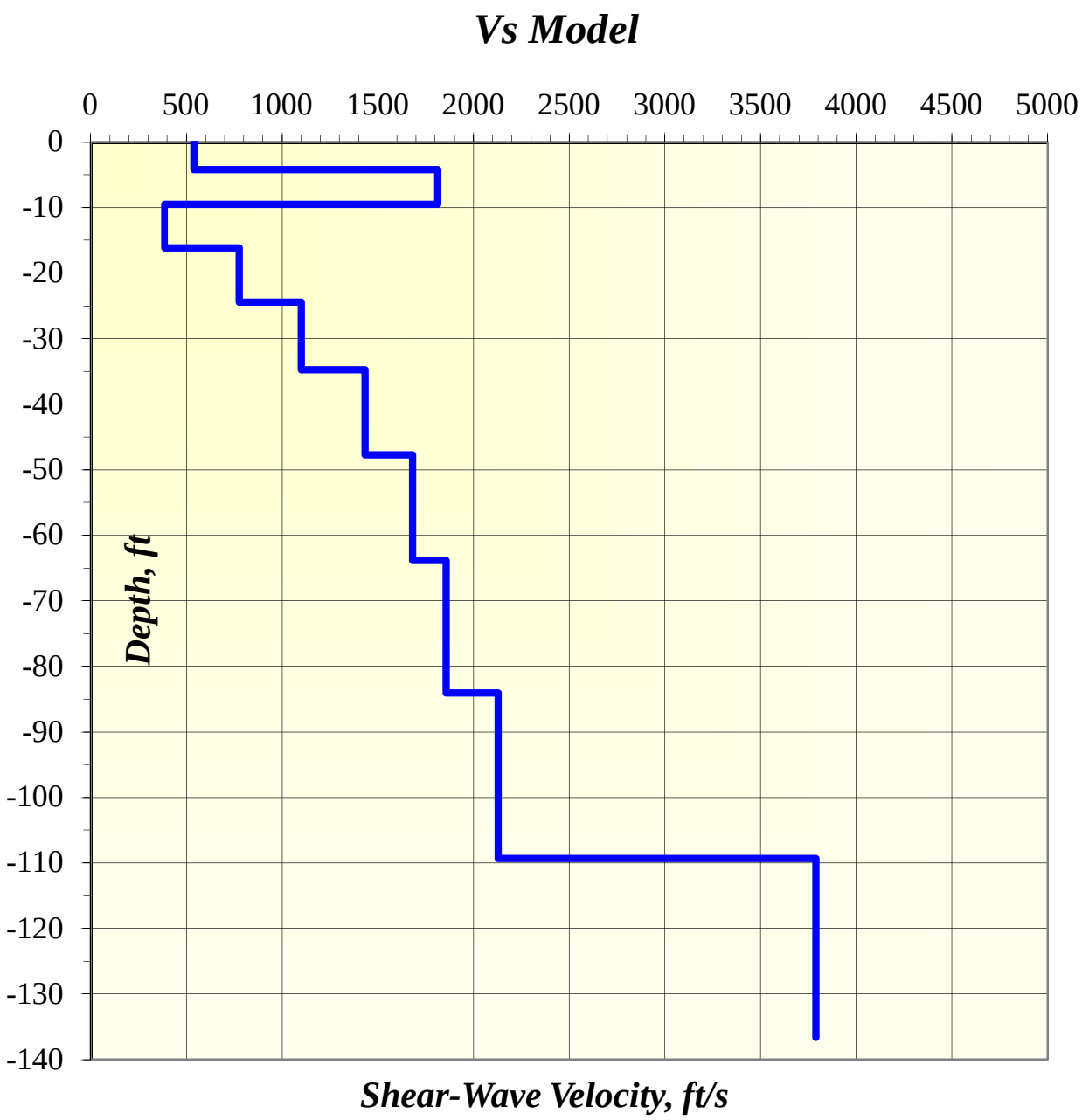
Line 1 - Seismic Profile



Average Shear Wave Velocity to 100 ft (rounded) = 2,605 ft/s

Project Manager: MDH	Project No. C4205024	Terracon Consulting Engineers & Scientists, 1392 13th Ave SW, Great Falls, MT 59404-3155	Shear Wave Profile
Drawn by: RMK	Scale: N.T.S.		Camp Muir Developed Area, Mount Rainier National Park, WA
Checked by: BJW	File Name: Saddle2.xlsx		
Approved by: BJW	Date: August 2020		

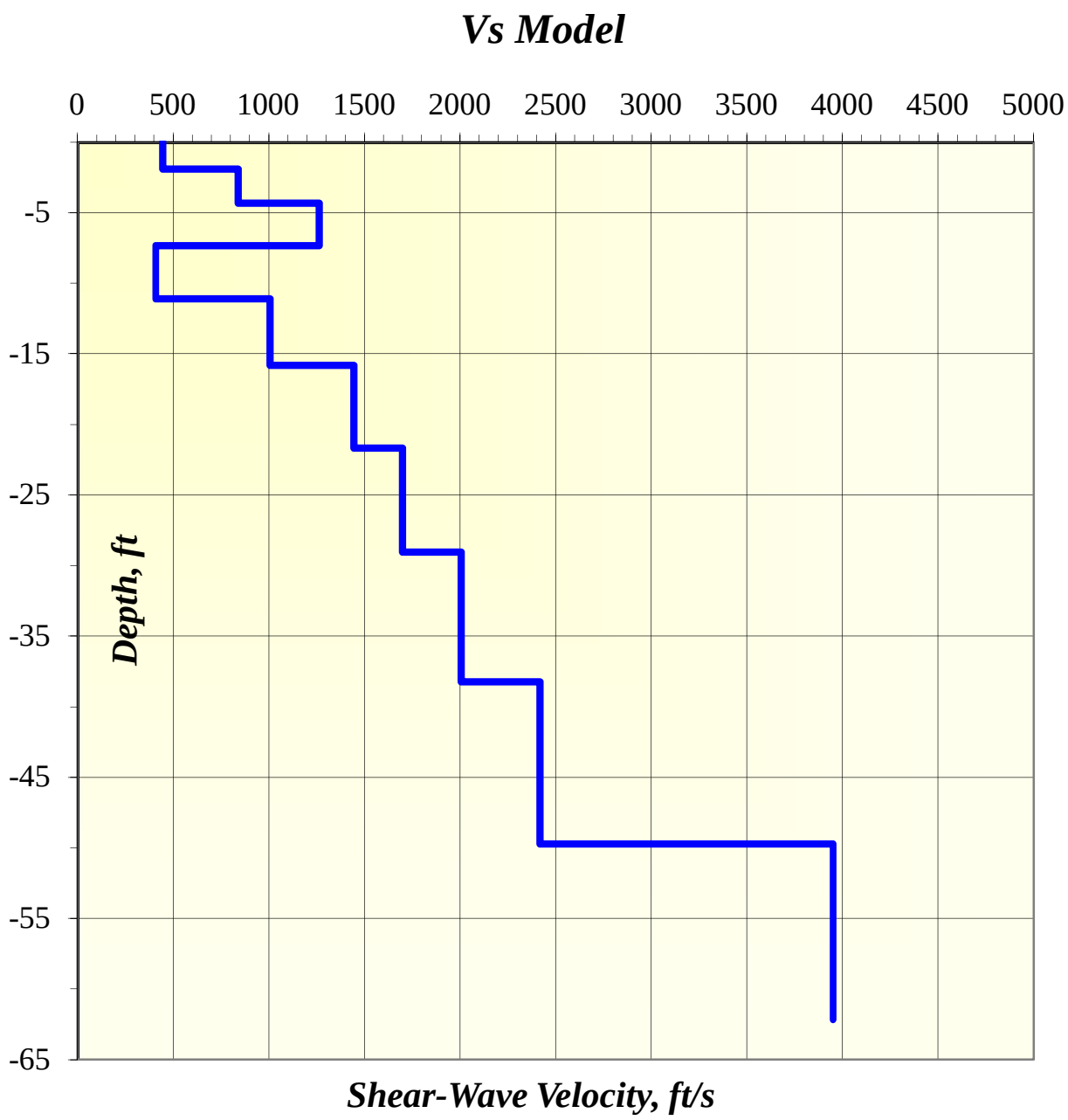
Line 2 - Seismic Profile



Average Shear Wave Velocity to 100 ft (rounded) = 1,040 ft/s

Project Manager: MDH	Project No. C4205024	Terracon Consulting Engineers & Scientists 1392 13th Ave SW Great Falls, MT 59404-3155	Shear Wave Profile
Drawn by: RMK	Scale: N.T.S.		Camp Muir Developed Area Mount Rainier National Park, WA
Checked by: BJW	File Name: GombuParallel.xlsx		
Approved by: BJW	Date: August 2020		

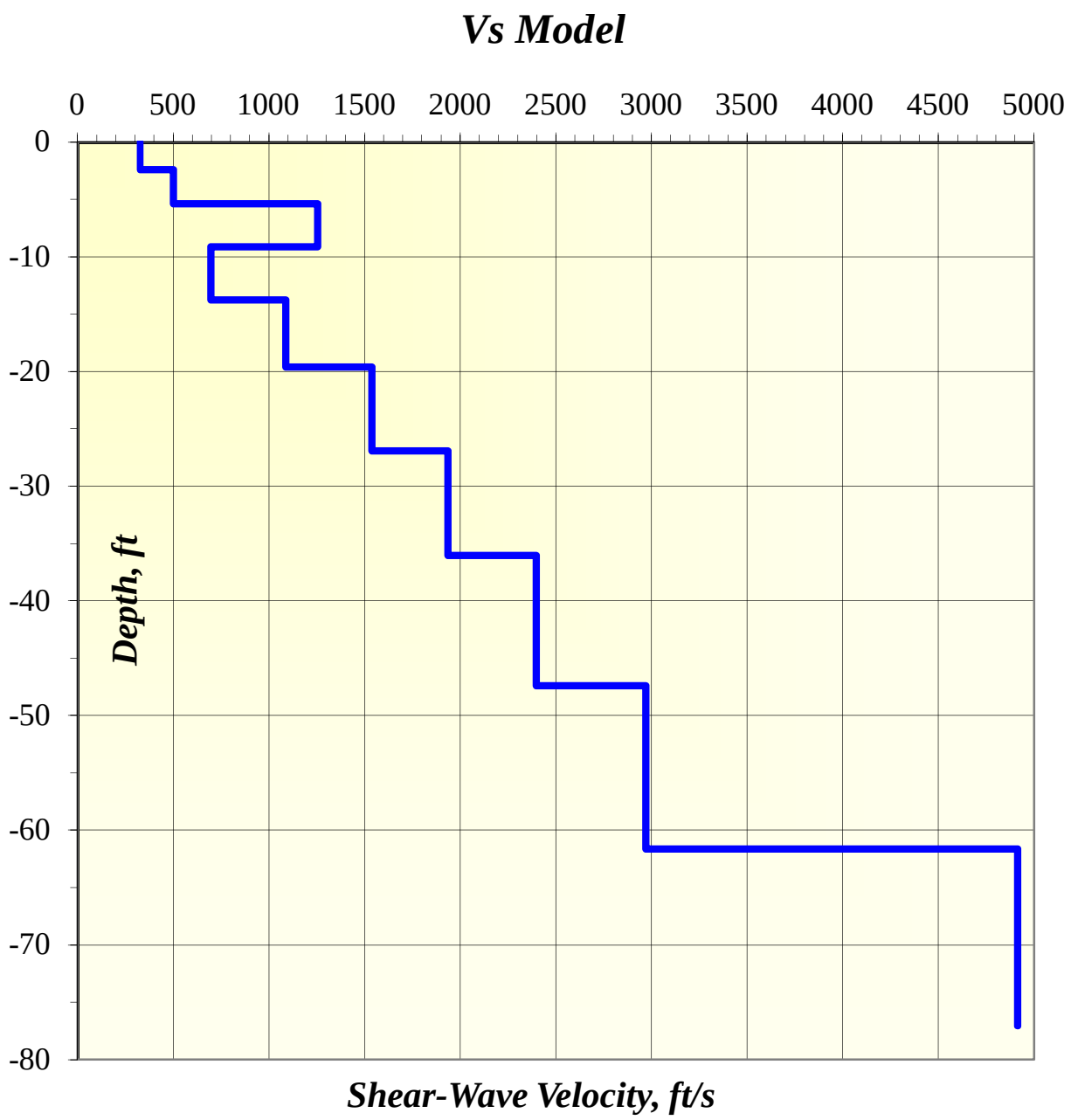
Line 3 - Seismic Profile



Average Shear Wave Velocity to 100 ft (rounded) = 2,260 ft/s

Project Manager: MDH	Project No. C4205024	Terracon Consulting Engineers & Scientists 1392 13th Ave SW Great Falls, MT 59404-3155	Shear Wave Profile
Drawn by: RMK	Scale: N.T.S.		Camp Muir Developed Area Mount Rainier National Park, WA
Checked by: BJW	File Name: GombuPerp.xls		
Approved by: BJW	Date: X August 2020		

Line 4 - Seismic Profile

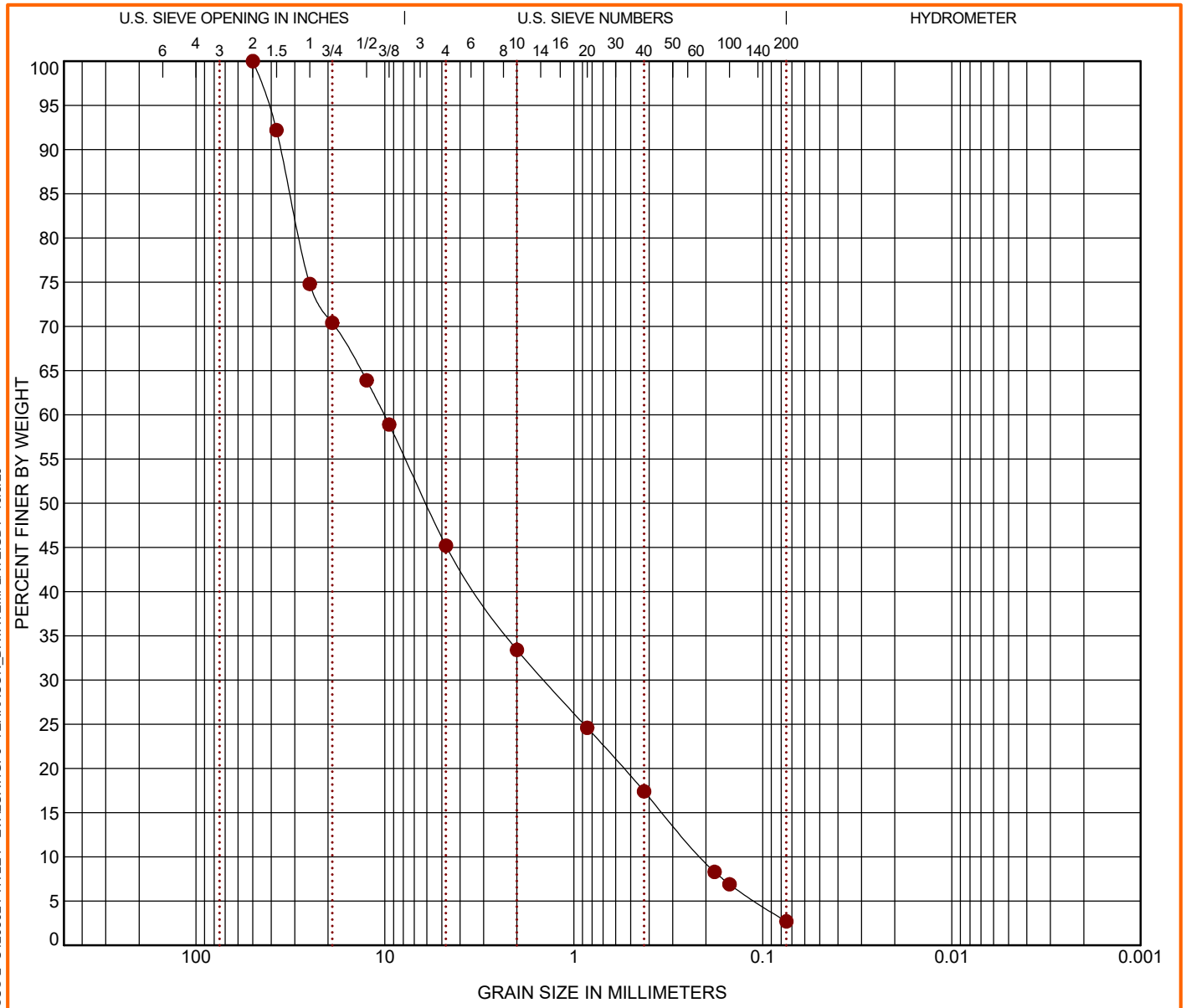


Average Shear Wave Velocity to 100 ft (rounded) = 1,980 ft/s

Project Manager: MDH	Project No. C4205024	Terracon Consulting Engineers & Scientists	Shear Wave Profile
Drawn by: RMK	Scale: N.T.S.	1392 13th Ave SW	Camp Muir Developed Area
Checked by: BJW	File Name: HelipadEast.xls	Great Falls, MT 59404-3155	Mount Rainier National Park, WA
Approved by: BJW	Date: August 2020		

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring ID	Depth	USCS Classification	WC (%)	LL	PL	PI	Cc	Cu
TP-01	2.2 - 2.4	POORLY GRADED GRAVEL with SAND (GP)					0.97	47.75

Boring ID	Depth	D_{100}	D_{60}	D_{30}	D_{10}	%Cobbles	%Gravel	%Sand	%Silt	%Fines	%Clay
TP-01	2.2 - 2.4	50	10.091	1.437	0.211	0.0	54.8	42.5		2.7	

PROJECT: Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area

SITE: Mount Rainier National Park
Paradise, WA

Terracon
1392 13th Ave SW
Great Falls, MT

PROJECT NUMBER: C4205024

CLIENT: DJ&A PC
Missoula, MT

LABORATORY TESTS ARE NOT VALID IF SEPARATED FROM ORIGINAL REPORT. GRAIN SIZE: USCS-2 C4205024 TITLE I - EVALUAT.GPJ TERRACON_DATATEMPLATE.GDT 10/8/20

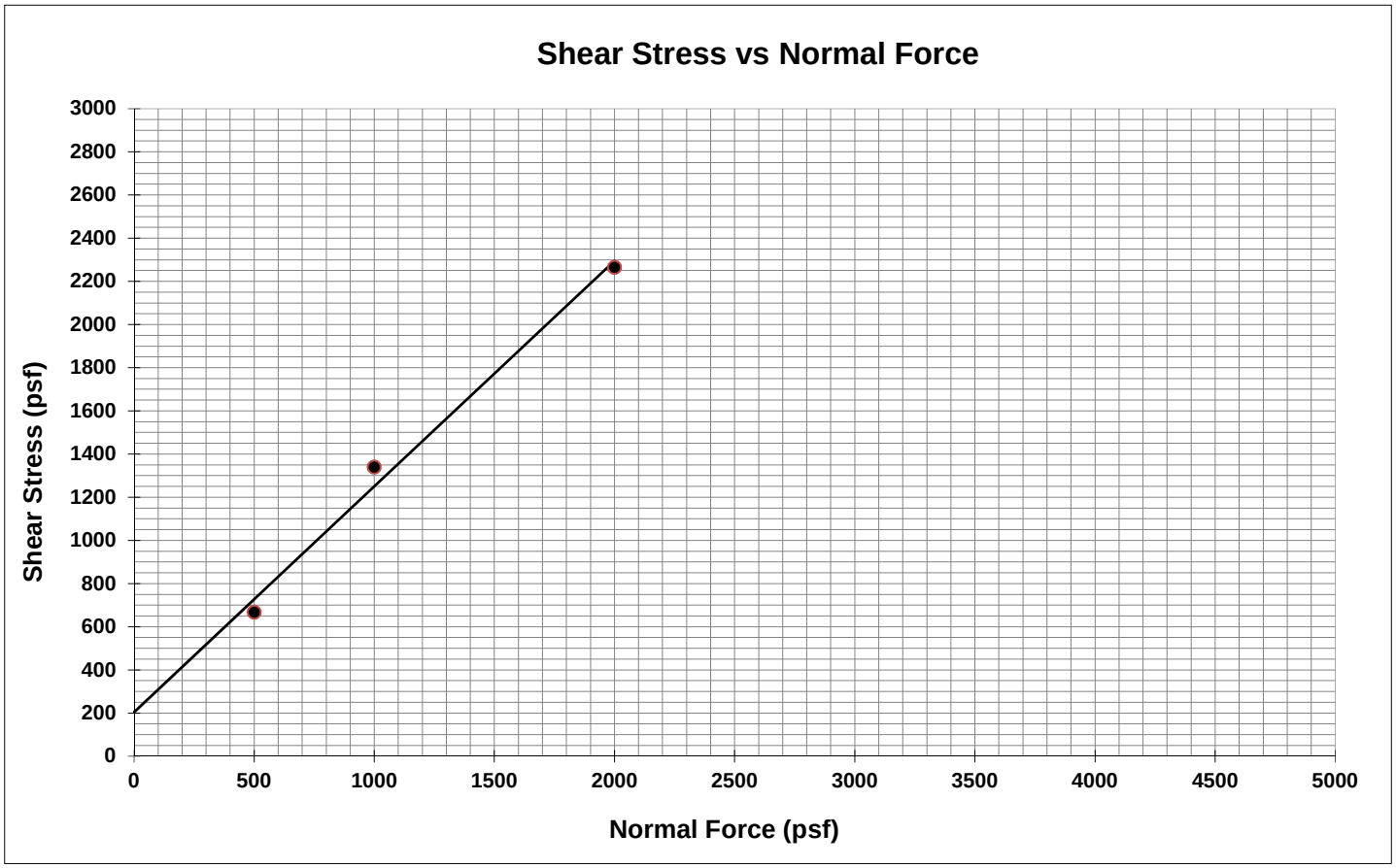
Direct Shear Test
Remolded-Consolidated Drained
ASTM D3080

Drill Hole: TP-2 Sample Depth: 2.2-2.5 Tested By: BE
Classification: GP with Sand - Pumice Reviewed By: MDH

Test Data

Load	Wet Unit Wt. (pcf)	Dry Unit Wt. (pcf)	Test MC%	After Test MC%	Shear Strength (psf)	% Compaction Obtained
500	68.18	55.32	23.2	46.9	667.31	N/A
1000	71.63	57.00	25.7	31.4	1340.02	N/A
2000	74.78	59.76	25.1	55.6	2264.99	N/A
Averages	**71.5**	**57.4**	**24.7**	**44.6**		

Shear Rate (inches/min): 0.00052



Cohesion: 204.83 ***Angle Phi:*** 46.3
Maximum Density ***Optimum MC%*** *25.15*

PROJECT:	Evaluate Geotechnical and Structural Stability of Camp Muir Developed Area	Terracon	PROJECT NUMBER: C4205024
SITE:	Mount Rainier National Park, WA	1392 13th Ave. SW Great Falls, MT	CLIENT: DJ&A, P.C. on behalf of National Park Service Interior Regions 8, 9, 10, & 12

SUPPORTING INFORMATION

Contents:

Unified Soil Classification System
Description of Rock Properties

Note: All attachments are one page unless noted above.

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]					Soil Classification	
					Group Symbol	Group Name [B]
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	Cu ≥ 4 and 1 ≤ Cc ≤ 3 [E]		GW	Well-graded gravel [F]
			Cu < 4 and/or [Cc<1 or Cc>3.0] [E]		GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH		GM	Silty gravel [F, G, H]
			Fines classify as CL or CH		GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	Cu ≥ 6 and 1 ≤ Cc ≤ 3 [E]		SW	Well-graded sand [I]
			Cu < 6 and/or [Cc<1 or Cc>3.0] [E]		SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH		SM	Silty sand [G, H, I]
			Fines classify as CL or CH		SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots on or above "A"		CL	Lean clay [K, L, M]
			PI < 4 or plots below "A" line [J]		ML	Silt [K, L, M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried	< 0.75	OL	Organic clay [K, L, M, N]
						Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line		CH	Fat clay [K, L, M]
			PI plots below "A" line		MH	Elastic Silt [K, L, M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried	< 0.75	OH	Organic clay [K, L, M, P]
						Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor				PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve.

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ≥ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.

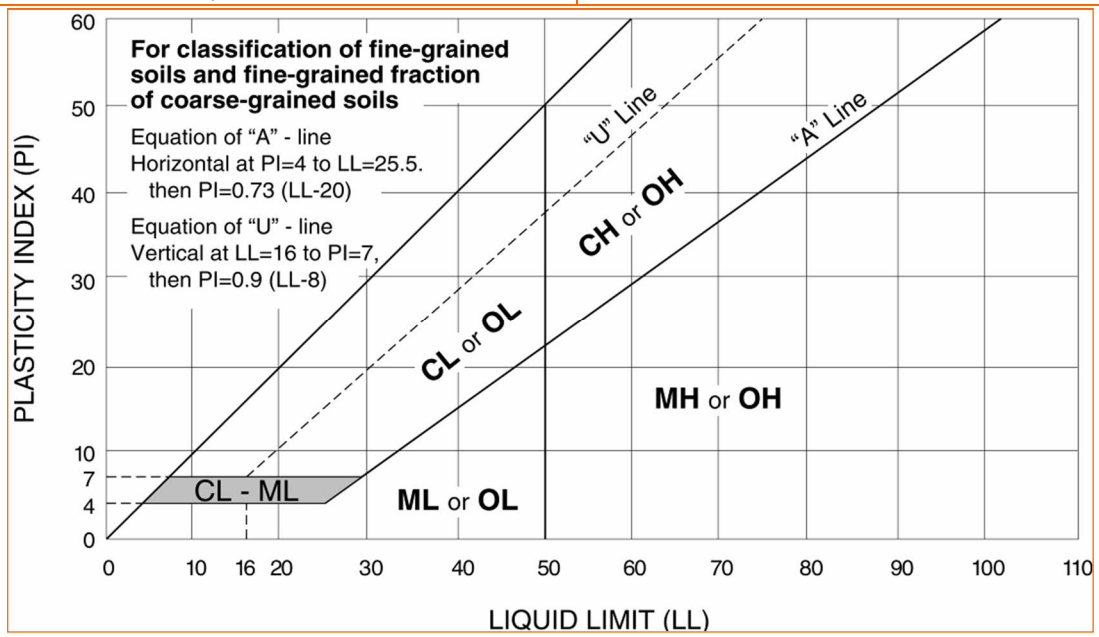
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] PI ≥ 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.



DESCRIPTION OF ROCK PROPERTIES

WEATHERING	
Fresh	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
Very slight	Rock generally fresh, joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
Slight	Rock generally fresh, joints stained, and discoloration extends into rock up to 1 in. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.
Moderate	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some show clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.
Moderately severe	All rock except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick.
Severe	All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
Very severe	All rock except quartz discolored or stained. Rock "fabric" discernible, but mass effectively reduced to "soil" with only fragments of strong rock remaining.
Complete	Rock reduced to "soil". Rock "fabric" no discernible or discernible only in small, scattered locations. Quartz may be present as dikes or stringers.

HARDNESS (for engineering description of rock – not to be confused with Moh's scale for minerals)	
Very hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows of geologist's pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
Moderately hard	Can be scratched with knife or pick. Gouges or grooves to ¼ in. deep can be excavated by hard blow of point of a geologist's pick. Hand specimens can be detached by moderate blow.
Medium	Can be grooved or gouged 1/16 in. deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1-in. maximum size by hard blows of the point of a geologist's pick.
Soft	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.
Very soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces 1-in. or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

Joint, Bedding, and Foliation Spacing in Rock [1]

Spacing	Joints	Bedding/Foliation
Less than 2 in.	Very close	Very thin
2 in. – 1 ft.	Close	Thin
1 ft. – 3 ft.	Moderately close	Medium
3 ft. – 10 ft.	Wide	Thick
More than 10 ft.	Very wide	Very thick

1. Spacing refers to the distance normal to the planes, of the described feature, which are parallel to each other or nearly so.

Rock Quality Designator (RQD) [1]

RQD, as a percentage	Diagnostic description
Exceeding 90	Excellent
90 – 75	Good
75 – 50	Fair
50 – 25	Poor
Less than 25	Very poor

1. RQD (given as a percentage) = length of core in pieces 4 inches and longer / length of run

Joint Openness Descriptors

Openness	Descriptor
No Visible Separation	Tight
Less than 1/32 in.	Slightly Open
1/32 to 1/8 in.	Moderately Open
1/8 to 3/8 in.	Open
3/8 in. to 0.1 ft.	Moderately Wide
Greater than 0.1 ft.	Wide

References: American Society of Civil Engineers. Manuals and Reports on Engineering Practice - No. 56. Subsurface Investigation for Design and Construction of Foundations of Buildings. New York: American Society of Civil Engineers, 1976. U.S. Department of the Interior, Bureau of Reclamation, Engineering Geology Field Manual.