APPENDIX A

U.S. Department
of Transportation
**Federal Highway
Administration**

MEMORANDUM

Western Federal Lands Highway Division
610 E. Fifth Street
Vancouver, WA 98661

DATE: March 23, 2020 In Reply Refer to: HFL-19

TO: Brandon Stokes
Project Manager

FROM: Douglas A. Anderson, Engineering Geologist
Nicholas Farny, Engineering Geologist
Evan Garich, Geotechnical Engineer
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Mike Baron, Construction Operations Engineer
Tyler Yeoman, Design Engineer

DAA. FOR ALL AUTHORS 3/23/2020

SUBJECT: **REVISED Geotechnical Memorandum 03-20**
Preliminary Pretty Rocks Landslide Bridge Feasibility and Constructability
Pretty Rocks Landslide Investigation
AK NPS DENA 10(45)

INTRODUCTION

At the request of the Park, we have evaluated the feasibility and constructability of the bridging option following the installation of four additional test borings (September 2019), laboratory testing completed in November 2019, and measurement and acquisition of subsurface borehole instrumentation data (September 2019 to January 2020) (Figure 1).

PRELIMINARY FOUNDATION STABILITY ANALYSES AND FEASIBILITY

To evaluate the feasibility of foundation locations to bridge the active, ice-rich Pretty Rocks Landslide, we determined the approximate bridge loads and analyzed the rock mass strength of the bedrock material at potential abutment locations. Rock mass strength is a function of two things, 1) the "intact rock" strength and, 2) the strength along the interface of existing fractures, or "discontinuities" in the rock. The analyses evaluated both of these strengths to determine the feasibility of the approximate bridge foundation locations (Figure 2).

The "intact rock" strength analysis is developed from breaking rock cores from the borings to determine if the rhyolite bedrock at the east abutment location and the basalt rock located at the west abutment is strong enough to withstand the loads of the bridge. These "intact rock" strengths were then used to evaluate the stability of the slopes where the foundations would likely be placed. The results of this part of the analysis shows the "intact rock" strength of the rhyolite and basalt are adequate to withstand the loads of the bridge.

The "rock discontinuity" strength analysis utilizes existing rock outcrop mapping of rock joints and fractures as well as the measurement of rock joints and fractures in the test borings during the drilling operation. We measure the direction the rock is dipping and describe the conditions of each discontinuity to help us determine if there are problematic (adverse) rock structures that may be unstable for the proposed bridge foundation locations.

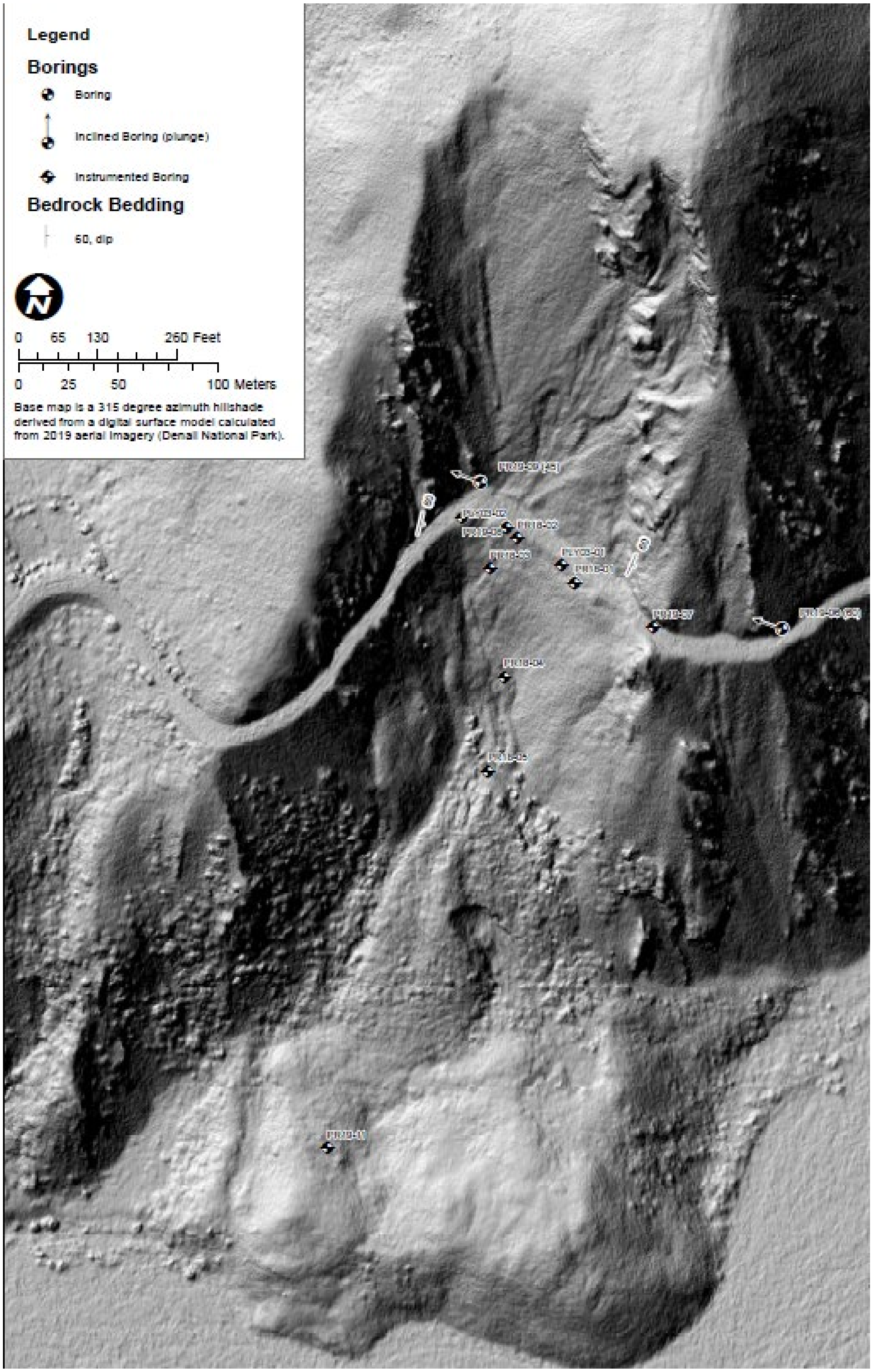

Figure 1. Test boring locations for Pretty Rocks Landslide Investigation.

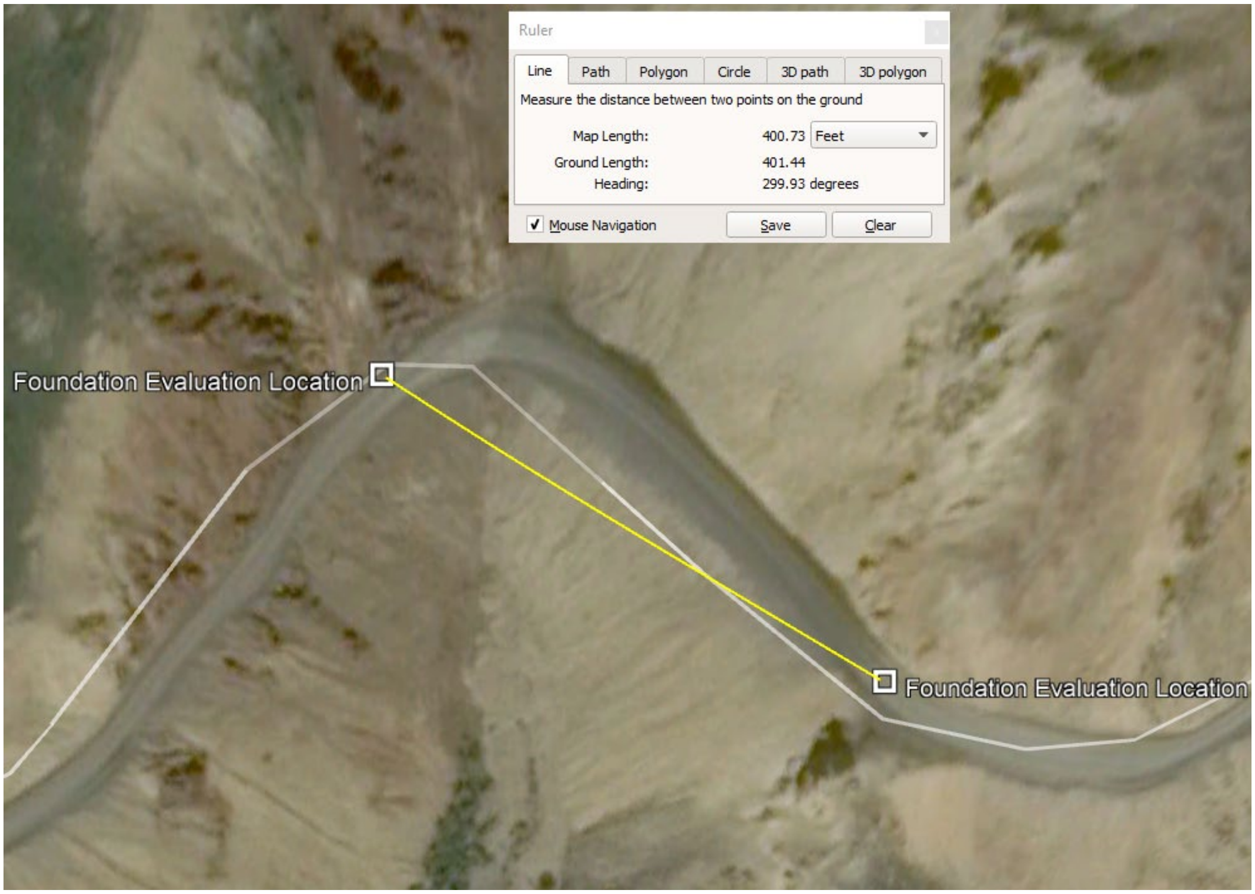

Figure 2. Approximate location of bridge foundations evaluated for feasibility of bridging the landslide.

In the case of the rhyolitic bedrock at the approximate east abutment location, there is an adverse rock discontinuity that dips out of the slope toward the landslide at approximately 66 degrees from horizontal and controls the stability of the proposed east abutment location (Figure 3). To meet the industry standard for factor of safety for the stability of a critical slope, such as at a bridge abutment, a factor of safety of 1.50 is required. The factor of safety is simply applying all the forces that drive the slope to be unstable (weight of slope materials, groundwater, bridge loads, etc.) divided by all the resisting forces (rock mass strength, slope reinforcement loads, etc.).

As shown in Figure 4, we do not meet this factor of safety, so deep foundation elements and/or rock reinforcement of the foundation area will likely be required to achieve a factor of safety of 1.50. We believe this can be achieved in this abutment location.

In the case of the basalt bedrock at the approximate west abutment location, there are two adverse rock structures but one of the rock discontinuities controls the stability of the slope. The low angle, 38 degree dipping rock discontinuity shown in Figure 5 displays the rock discontinuity of greatest concern for the basalt bedrock proposed foundation location. This rock discontinuity is likely a large contributor to the existing Pretty Rocks Landslide failure, and must be mitigated for the west abutment area. As shown in Figure 6, this location also does not meet the factor of safety of 1.50. *The initial evaluation suggests that this foundation location will likely require a complex and iterative mitigation strategy that may involve a combination of significant rock excavation, deep*

Figure 3. Photograph of the rhyolite bedrock outcrop near the east bridge abutment location. Red arrows show the direction of the controlling discontinuity.

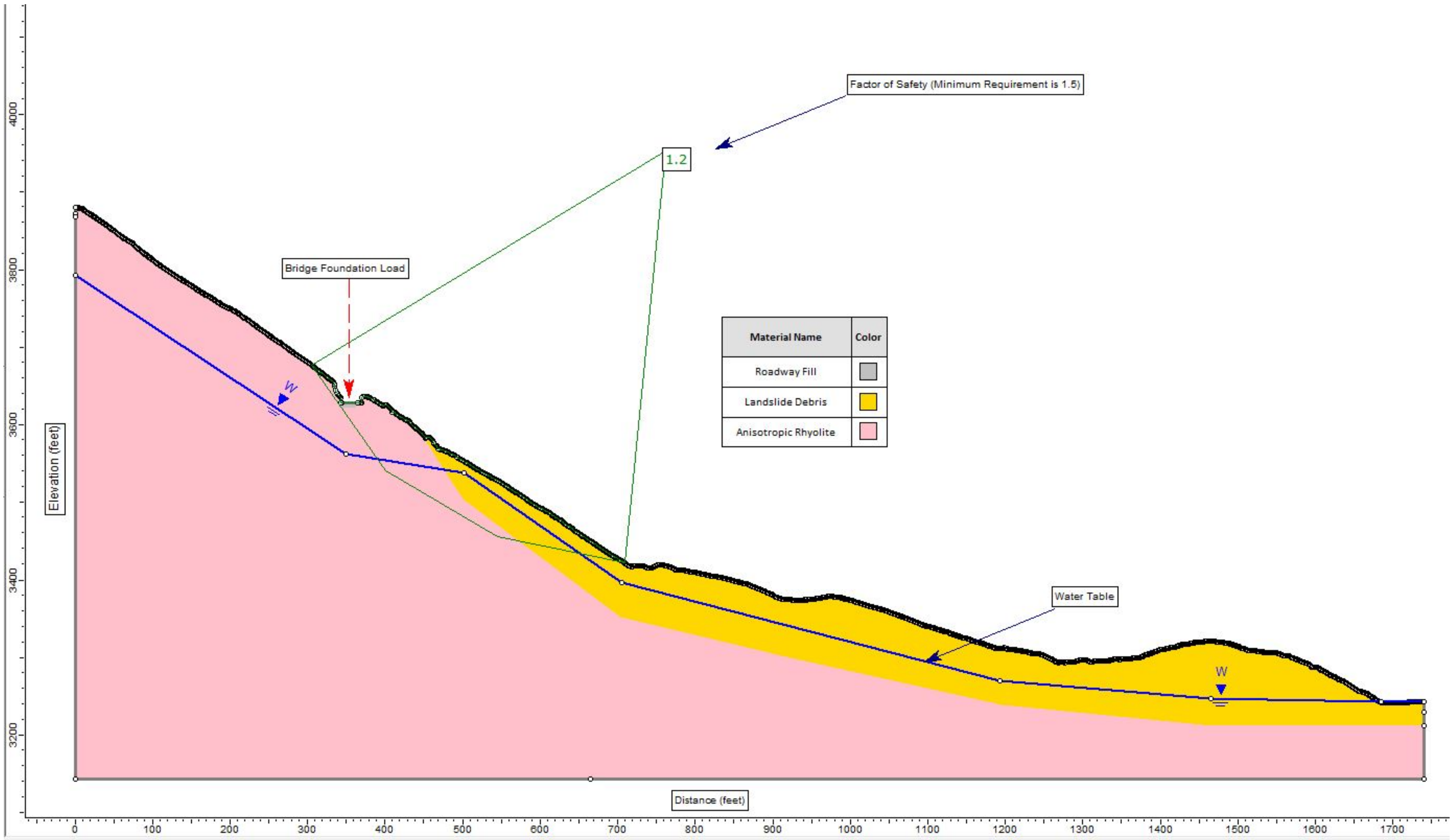

Figure 4. Slope stability analysis of east abutment location with the adverse, steeply dipping rhyolitic rock discontinuity and approximate bridge loads.

Figure 5. Photograph of the basalt bedrock outcrop at the approximate location of the west bridge abutment. Red arrows show the direction of the controlling discontinuity.

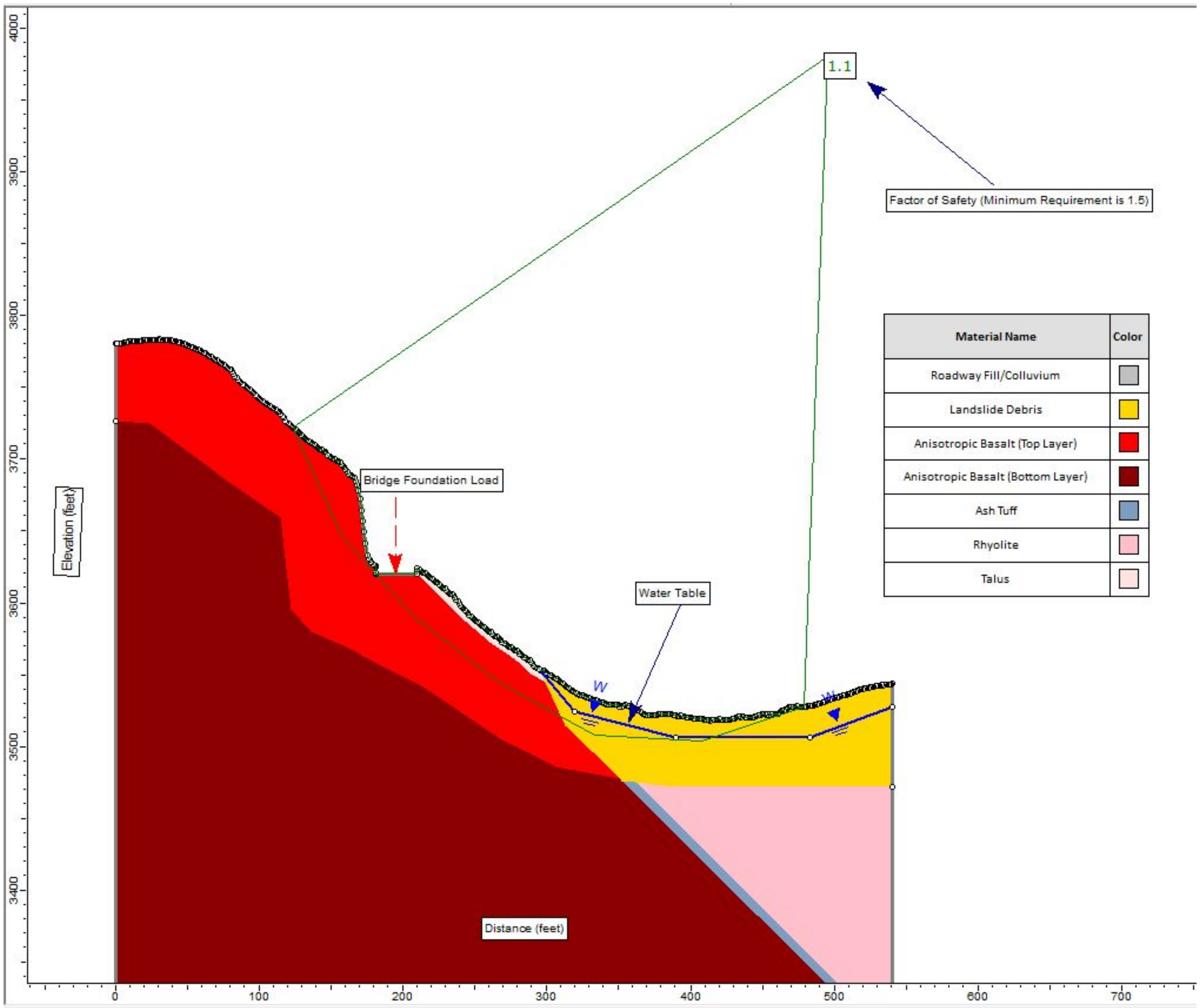

Figure 6. Slope stability analyses of the west abutment location with the adverse, low angle dipping basaltic rock discontinuity and approximate bridge loads.

Figure 14: Storage of New Bridge Girders in Local Storage Yard

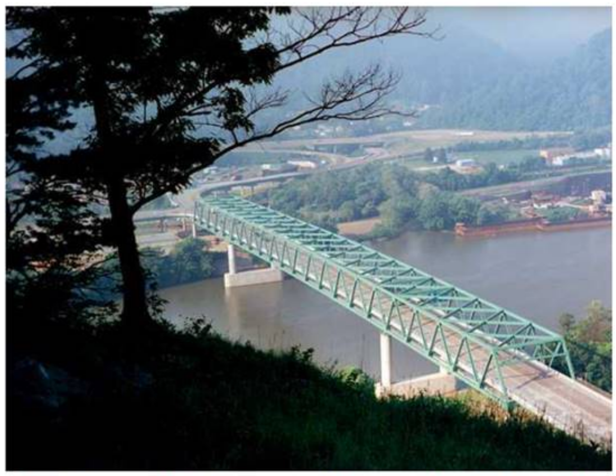

Figure 4 Photo of the Chelyan Bridge over Kanawha River, Kanawha County, West Virginia

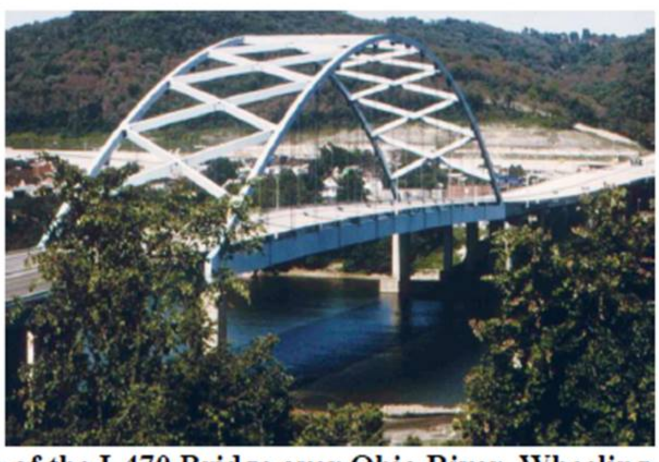

Figure 9 Photo of the I-470 Bridge over Ohio River, Wheeling, West Virginia

Figure 7. Examples of (from top to bottom) long span structural steel plate girder, steel through truss, and long span steel network arch bridges. Bridge materials would likely be weathered steel of a brown/orange color.

foundation elements, and specialized rock reinforcement strategies. While feasible, this west abutment location presents several design and mitigation complexities and may result in a less desirable bridge type option for the Park to achieve the spans for a stable foundation design. Other west abutment locations that were closer to the unstable landslide mass, and could have resulted in a shorter bridge span length, were evaluated. However, the stability analyses for these shorter bridge span length locations, informed by the test boring data, were not feasible.

BRIDGE STRUCTURE OPTIONS

The bridge types shown in Figure 7 are being conceptually presented for consideration as traditional bridges that would be utilized for long span length locations.

The long span, structural steel plate girder bridge is similar to other bridges in the Park (Steel girders with concrete decks) and is likely preferred; however, the depth of the girder would likely be on the order of 16 feet deep for a 400-foot-long span (similar to the upper photo in Figure 7). This deep of a girder and span length likely exceeds the bridge construction industry's capacity. Challenges to fabricate, ship, and erect this type of bridge at the site would be substantial, if not prohibitive. We have concerns if this bridge type at this span length could be constructed due to the limited access to the project location and the anticipated crane size and staging areas that would be required for assembly of the bridge. ***All things considered, the typical Denali Park Road preferred steel girder bridge option is not feasible.***

Two other single span options for this span length are a steel truss (middle photo in Figure 7) or a long span arch type bridge (lower photo in Figure 7). We understand that these two types of bridges may not be preferred by the Park but they are the bridge types that can accommodate this span length.

The long span arch type bridge (lower photo in Figure 7) will have similar erection and construction challenges as the steel plate girder option. The large cranes, staging areas and site constraints (need for temporary supports at midpoint of bridge within the landslide) needed to erect the structure make this option not feasible.

The most feasible and constructable, although challenging option, would be a launchable modular steel truss (middle photo of Figure 7). A launchable steel truss is assembled at one end and pushed out, or cantilevered out, over the ground or river that is intended to be crossed. For shorter spans, this can be accomplished without cranes. For the span length required at this site, a large crane will likely still be needed near the western abutment. Another construction option would be to construct the truss along a parallel alignment to the permanent location and then lift the bridge onto its foundation with two large cranes. This method would still utilize constructing this bridge from the eastern abutment and pushing it out along the temporary road alignment. The maximum span length for commercially available bridges of this type is on the order of 400 feet. The need for crane pads and the current, limited work space are still major constructability challenges for the launchable modular steel truss option and is further discussed under the traffic and construction footprint discussion below.

Other advantages of this structure type include light weight small structural pieces (easier transportation to the remote site), high friction metal plate decks (eliminating casting concrete deck in remote location), and relatively quick construction/launch of the superstructure with minimal impacts to traffic from road closures. Figure 8 shows the typical section for a single lane launchable bridge. Figure 9 shows some of the stages of a typical launched steel truss. Figure 10 provides a rough draft concept of how a modular truss bridge might appear if constructed at the project location. This figure is preliminary and is intended to provide a visualization for this bridging option.

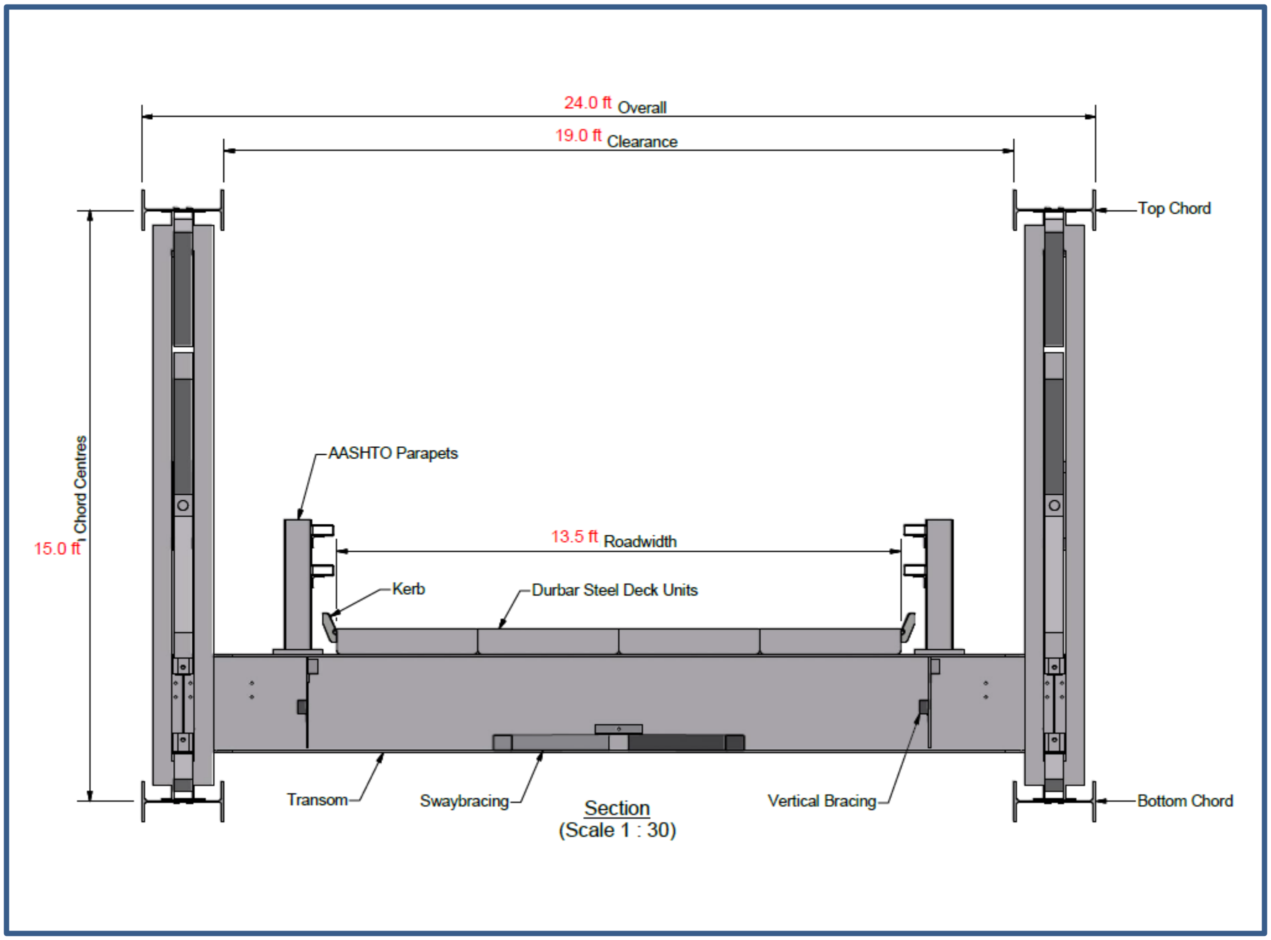

Figure 8. Typical Section single lane launchable truss

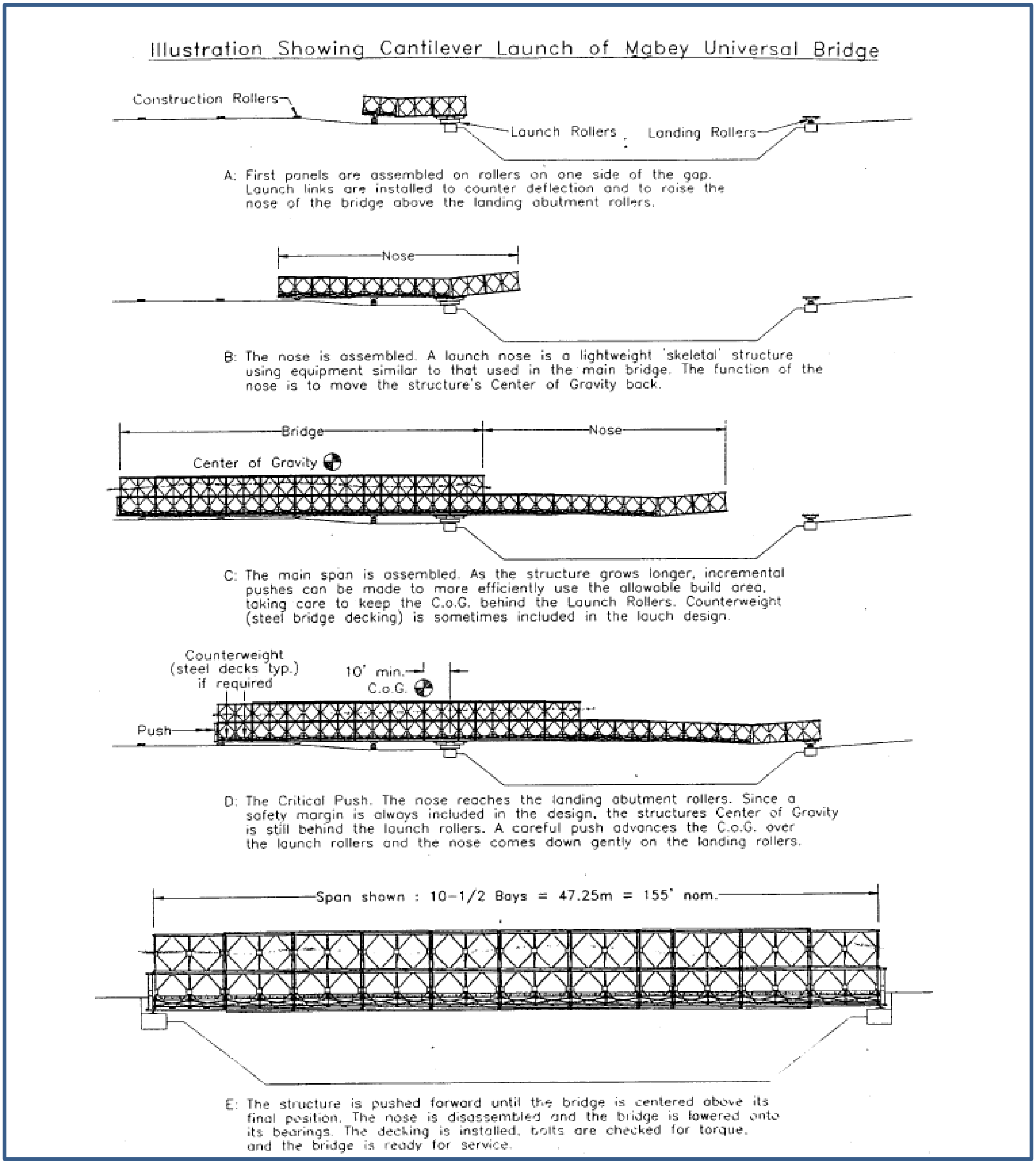

Figure 9. Stages of launching a prefabricated steel truss

Figure 10. Rough concept of a Modular Steel Truss constructed at Pretty Rocks Landslide

TRAFFIC AND CONSTRUCTION FOOTPRINT CONSIDERATIONS

The road bench across the Pretty Rocks Landslide, and adjacent to, varies in width and is typically less than the 24 feet width found between Teklanika Rest Area and Toklat along the Park Road. This narrow section of road is bordered by steep rock slopes above and steep drop offs below the road on both sides of the active landslide.

The most feasible bridge type, as discussed above, is the steel truss modular design. In order for it to be constructed, it will require rock cut excavation adjacent to the west and east abutments to allow for assembly of the bridge, staging of equipment and material, and to accommodate the bridge approach road alignment on the west side (Figure 11). The anticipated quantity of rock excavation is estimated at 150,000 to 200,000 cubic yards (CY) and extends about 100 feet above the road at the east abutment and 200 feet above the road at the west abutment (Figure 12 and 13, respectively).

As discussed above in the preliminary foundation section, the bridge abutments will require additional rock excavation and a combination of deep foundations and rock reinforcement to meet design standards for bridge foundation stability. The modular steel truss bridge design is typically assembled behind the abutment and launched into place, limiting the space requirements for assembly. However, because of the road curvature at the east and west abutments, there is not sufficient space behind either abutment to assemble in this manner, so the bridge would likely be assembled on the existing road alignment across the landslide and lifted into place, as discussed in the bridge structure options section.

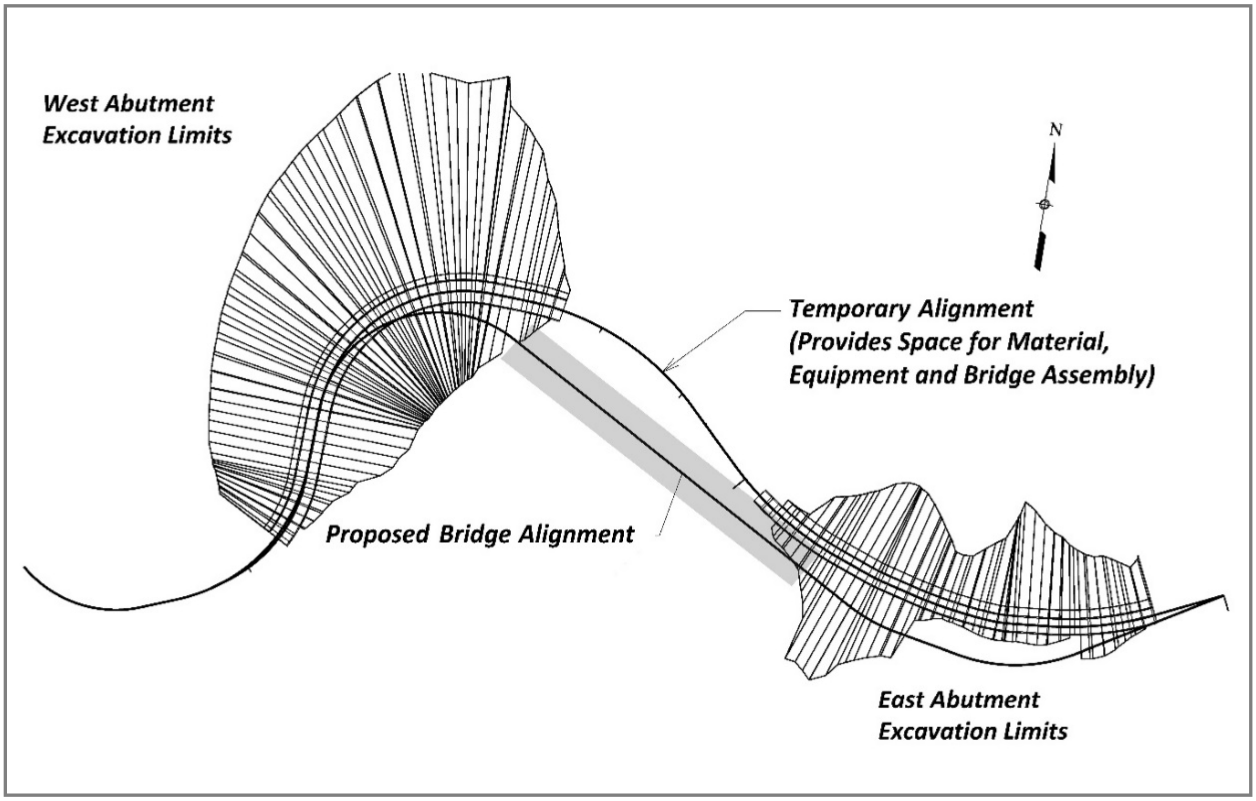

Figure 11. Excavation limits. East abutment excavation provides space for bridge assembly during construction and a rock fall ditch post construction. The west abutment excavation provides for the bridge approach road alignment (turning radius).

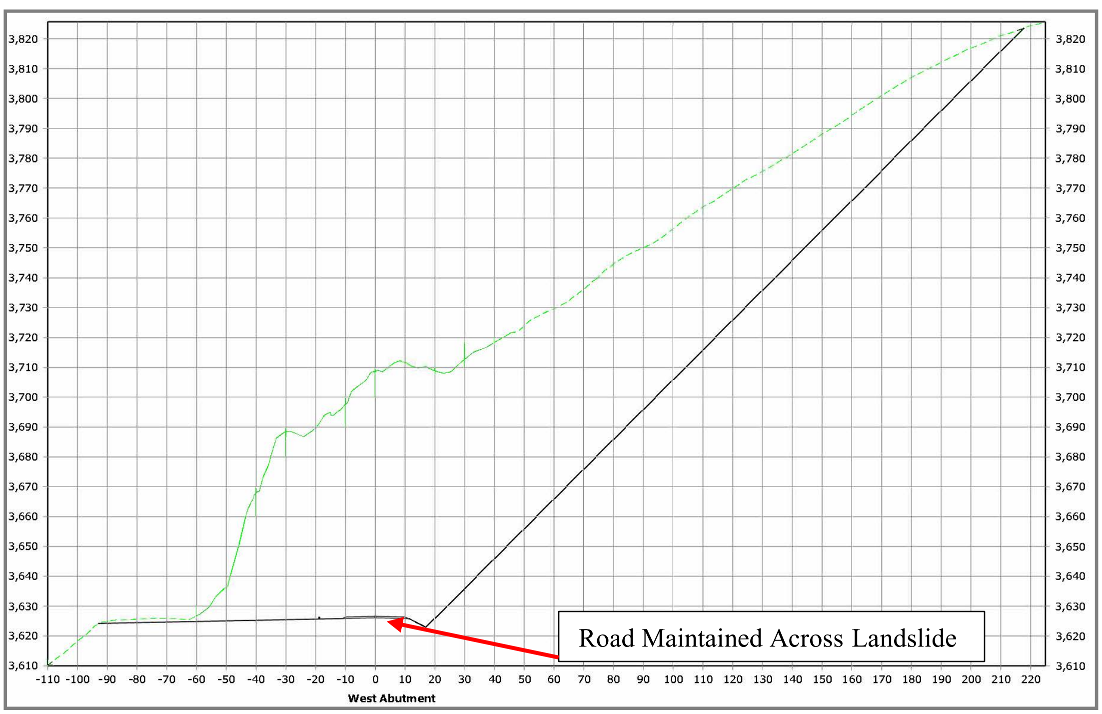

Figure 12. Cross section near the west abutment. Existing ground is represented by the green dashed line.

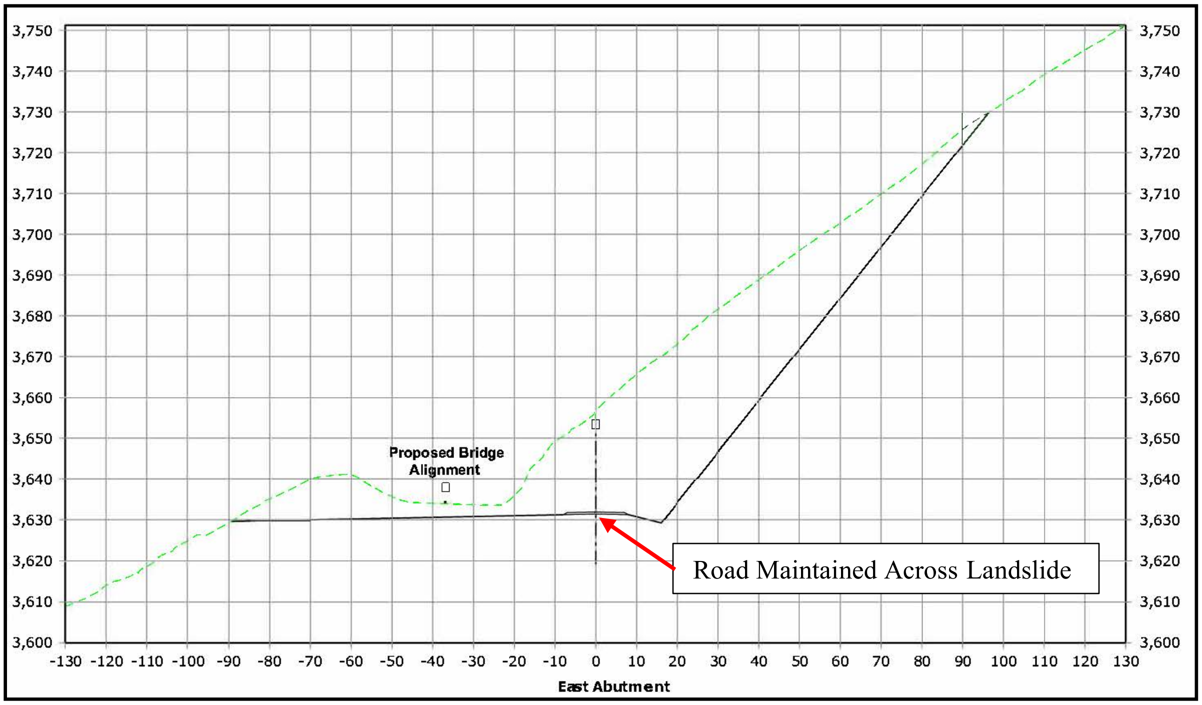

Figure 13. Cross section near the east abutment. Existing ground is represented by the green dashed line.

The duration of construction is largely dependent on where the excavated material is disposed of (on-site or off-site) and the time of year the work is performed. *If the material is disposed of on-site and the work is completed during favorable weather conditions, April thru September, the work can be completed in approximately 10 to 12 months of active construction. One full construction season followed by a partial season. If the work is performed during the visitor "off-season" October thru May, the work will require at least 2 to 3 full years (4 to 6 shoulder seasons), and if the rock excavation spoils are disposed of off-site, construction will last considerably longer.*

Based on this evaluation, the limited space available at the site, and the type of work required, it will prevent public traffic access for most construction activities.

CONCLUSION

In conclusion, if the feasible and constructable lightweight steel truss bridge option is selected to move forward, the following would likely be required in addition to the bridge work:

- Significant rock excavation at the west abutment to 1) allow for a roadway to maintain access for Park traffic and establish room for bridge construction, 2) possibly set back the bridge foundation from the edge of the landslide to improve slope stability, and 3) connect the roadway with the new bridge with a sharp radius turn (~50 to 100 feet radius would match existing constraints on Polychrome Pass).
- Rock excavation at the east abutment along the inside of the roadway into the rhyolite bedrock to provide adequate space for a roadway to maintain access for Park traffic and to provide space for construction of the bridge. This will also serve as a rockfall ditch

following construction of the bridge to minimize impacts to the bridge from rockfall. The required rock excavation adjacent to the east abutment may be reduced as the bridge design is further developed.

- Public access through the site cannot be accommodated during most construction activities. The work will require either a full road closure or a phased multi-year approach to completing the work during the visitor off-season.

LIMITATIONS

This memorandum has been prepared to assist the National Park Service in evaluating the feasibility of the bridge option for the Pretty Rocks Landslide. It should not be used, in part or in whole for other purposes without contacting the Western Federal Lands Highway Division (WFL) for a review of the applicability of such reuse. These data are not to be used for other purposes.

The conclusions and recommendations contained in this report are based on WFL's understanding of the project at the time that the memorandum was written and onsite conditions that existed at time of the field observations and subsurface exploration. If significant changes to the nature, configuration, or scope of the project occur, WFL should be consulted to determine the impact of such changes on the preliminary Pretty Rocks Landslide bridge option feasibility and constructability analyses and conclusions presented in this memorandum.

CLOSING

If you have any questions or concerns regarding the information contained in this memorandum, please contact Brandon Stokes at 360-619-7813.

CC: Michael Madar, Highway Design Manager
Eric Lim, Acting Geotechnical Functional Manager
Geotechnical File

APPENDIX B

Google Earth
Image © 2019 DigitalGlobe
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MP 44.3-44.7
MP 44.3
Denali Park Road
MP 44.4
Park Rd
LEGEND:
Slope Identification No. (R) Rock Slope
(S) Soil Slope
Milepost Indicator
MP 44.5
USMP Preliminary Rating
US 027-R
Site 863
Rockfall/Landslide
US 003-R
Site 864
MP 44.7
MP 44.6
N
500 ft

Denali National Park Unstable Slope Corridor Assessment

Site 863 (DENA_USMP_027)

Agency: NPS
Region: Alaska Region
Road: Denali Park Road (Road 10)
Side of Centerline: R

Beginning MP: 44.57 ***Ending MP:*** 44.59

Numerical Rating: 478

Problem Definition:

This unstable slope has a maximum slope height of approximately 95 feet and is approximately 180 feet long. The slope is composed of decomposing rhyolitic (igneous) rock that is producing both, structurally controlled planar and wedge failures and differential erosion failures consisting of boulders with a maximum block size of two feet.

Problem Correction:

At a minimum, rock scaling should be performed as a temporary risk reduction measure along the slope to remove loose, precariously positioned rocks. The rock scaling would need to be repeated every five to ten years to maintain the same level of risk reduction. For a higher level of risk reduction (Option 2), rock scaling should be completed and large, unsupported rock features would require rock reinforcement with rock bolts and dowels. Cleaning and maintaining the existing ditch will also be required to preserve its effectiveness to contain future rockfall.

Geotechnical Estimating Factors:

Option 1: Scaling Only

Item	Unit	Amount	Unit Cost	Total Cost
Geotechnical Design	Lump Sum	1	$ 25,000	$ 25,000
Scaling and Debris Removal	Scaler Hour	300	$ 400	$ 120,000
		Estimated Costs		$ 145,000

Option 2: Scaling and Rock Reinforcement.

Item	Unit	Amount	Unit Cost	Total Cost
Geotechnical Design	Lump Sum	1	$ 30,000	$ 30,000
Rock Reinforcement Installation	Lineal Foot	850	$ 250	$ 212,500
Scaling and Debris Removal	Scaler Hour	300	$ 400	$ 120,000
		Estimated Costs		$ 362,500

Completed By: NJF/DAA/BC Date: July 20, 2020

Denali National Park Unstable Slope Corridor Assessment

Site 864 (DENA_USMP_003)

Agency: NPS
Region: Alaska Region
Road: Denali Park Road (Road 10)
Side of Centerline: R

Beginning MP: 44.59 ***Ending MP:*** 44.64

Numerical Rating: 537

Problem Definition:

This unstable slope has a maximum slope height of approximately 60 feet and is approximately 250 feet long. The slope is composed of rhyolitic (igneous) rock that is producing primarily planar, wedge, and discrete block structurally controlled rock failures. The maximum block size is approximately three feet.

Problem Correction:

At a minimum, rock scaling should be performed as a temporary risk reduction measure along the slope to remove loose, precariously positioned rocks. The rock scaling would need to be repeated every five to ten years to maintain the same level of risk reduction. For a higher level of risk reduction (Option 2), rock scaling should be completed and large, unsupported rock features would require rock reinforcement with rock bolts and dowels. Cleaning and maintaining the existing ditch will also be required to preserve its effectiveness to contain future rockfall.

Geotechnical Estimating Factors:

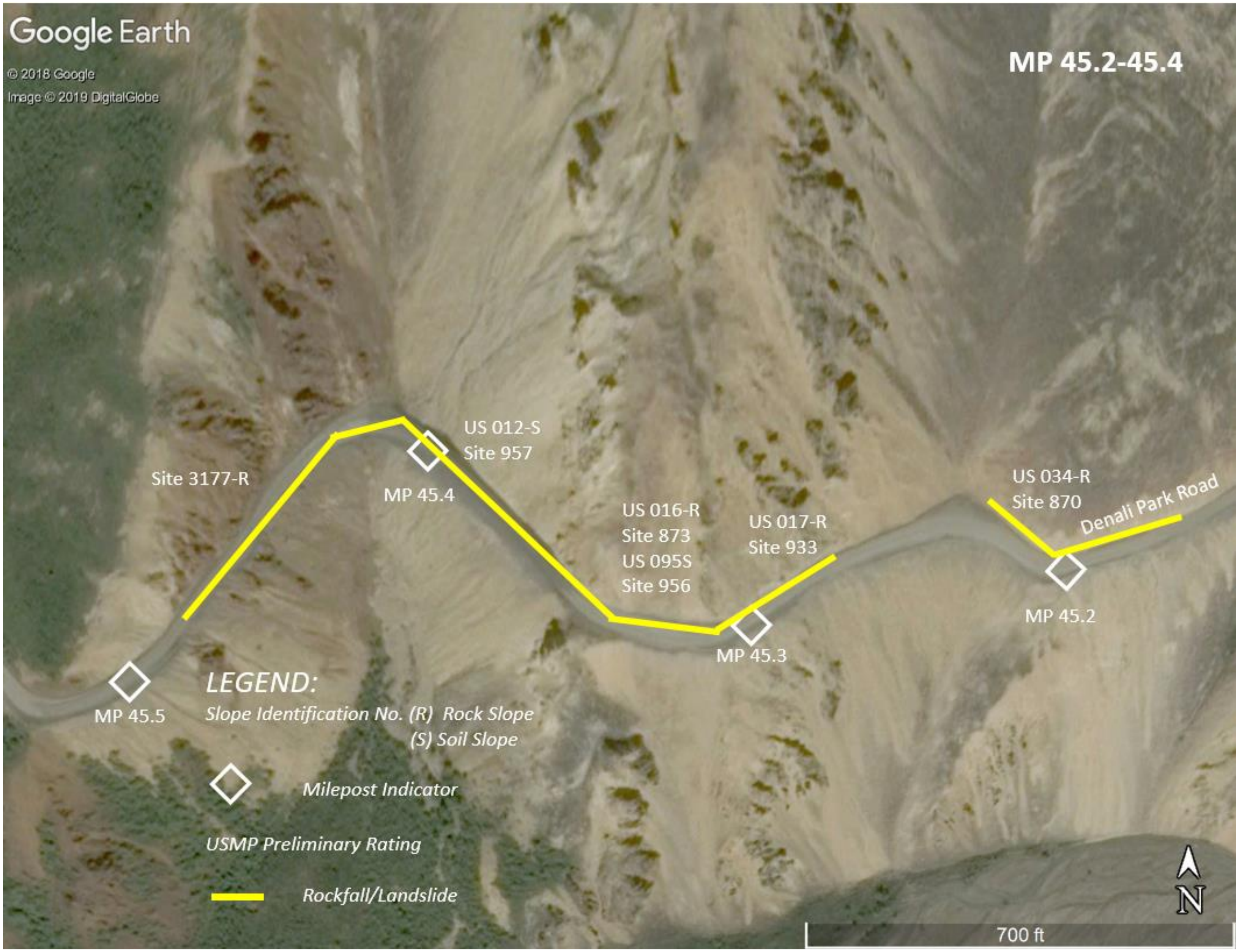
Option 1: Scaling only

Item	Unit	Amount	Unit Cost	Total Cost
Geotechnical Design	Lump Sum	1	$ 25,000	$ 25,000
Slope Scaling	Scaler Hour	300	$ 400	$ 120,000
		Estimated Costs		$ 145,000

Option 2: Scaling and Rock Reinforcement

Item	Unit	Amount	Unit Cost	Total Cost
Geotechnical Design	Lump Sum	1	$ 35,000	$ 35,000
Rock Reinforcement Installation	Lineal Foot	1,385	$ 250	$ 346,250
Scaling and Debris Removal	Scaler Hour	300	$ 400	$ 120,000
		Estimated Costs		$ 501,250

Completed By: NJF/DAA/BC Date: July 20, 2020



Google Earth
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Image © 2019 DigitalGlobe
MP 45.2-45.4
US 012-S
Site 957
Site 3177-R
MP 45.4
US 016-R
Site 873
US 095S
Site 956
US 017-R
Site 933
US 034-R
Site 870
Denali Park Road
MP 45.2
MP 45.3
MP 45.5
LEGEND:
Slope Identification No. (R) Rock Slope
(S) Soil Slope
Milepost Indicator
USMP Preliminary Rating
Rockfall/Landslide
N
700 ft

Google Earth
© 2018 Google
Image © 2019 DigitalGlobe
MP 44.7-44.8
Denali Park Road
MP 44.7
US 029-S
Site 955
MP 44.8
LEGEND:
Slope Identification No. (R) Rock Slope
(S) Soil Slope
Milepost Indicator
USMP Preliminary Rating
Rockfall/Landslide
Park Rd
N
600 ft

Denali National Park Unstable Slope Corridor Assessment

Site 870 (DENA_USMP_034)

Agency: NPS
Region: Alaska Region
Road: Denali Park Road (Road 10)
Side of Centerline: R

Beginning MP: 45.17 ***Ending MP:*** 45.21

Numerical Rating: 526

Problem Definition:

This unstable road cut slope has a maximum slope height of approximately 55 feet and is approximately 230 feet long. The slope is composed of rhyolitic rock that is producing primarily wedge and planar structurally controlled rock failures with several localized areas of raveling and undermining occurring. The average block size is approximately three feet, and the slope can also produce up to 5 foot blocks and debris slide failure events up to about nine cubic yards in volume. The existing road cut is oversteepened at a slope angle of 40 to 41°, and the upper natural slope appears stable at a slope angle of 30 to 35°. Rockfall ditch catchment is limited and sight distance is very limited along this stretch of the road.

Problem Correction:

At a minimum, rock scaling should be performed as a temporary risk reduction measure along the slope to remove loose, precariously positioned rocks. The rock scaling would need to be repeated every five to ten years to maintain the same level of risk reduction. For a higher level of risk reduction (Option 2), rock scaling should be completed and large, unsupported rock features would require rock reinforcement with rock bolts and dowels. Cleaning and maintaining the existing ditch will also be required to preserve its effectiveness to contain future rockfall. The rounding of the brow of the slope should be laid back between 35 and 39° slope angle.

Geotechnical Estimating Factors:

Option 1: Scaling Only

Item	Unit	Amount	Unit Cost	Total Cost
Geotechnical Design	Lump Sum	1	$ 25,000	$ 25,000
Scaling and Debris Removal	Scaler Hour	220	$ 400	$ 88,000
		Estimated Costs		$ 113,000

Option 2: Scaling and Rock Reinforcement

Item	Unit	Amount	Unit Cost	Total Cost
Geotechnical Design	Lump Sum	1	$ 35,000	$ 35,000
Scaling and Debris Removal	Scaler Hour	220	$ 400	$ 88,000
Rock Reinforcement Installation	Lineal Foot	655	$ 250	$ 163,750
		Estimated Costs		$ 286,750

Completed By: DAA/NJF/BC Date: July 31, 2020

Denali National Park Unstable Slope Corridor Assessment

Sites 873 (DENA_USMP_016) and 956 (DENA_USMP_095)

Agency: NPS
Region: Alaska Region
Road: Denali Park Road (Road 10)
Side of Centerline: R

Beginning MP: 45.32 ***Ending MP:*** 45.34

Numerical Rating: 460 (Site 873) and 450 (Site 956)

Problem Definition:

Site 873 (DENA_USMP_016), and Site 956 (DENA_USMP_095) are combined for the purposes of conceptual design due to their proximity and the similarity of the proposed problem correction for both sites.

Site 873, also known as "Perlite Rockfall", is slope with a maximum height of 140 feet that affects 125 feet of the Denali Park Road. It consists of decomposing rhyolite with intermittent layers of perlite that is producing structurally controlled planar, wedge, and indeterminate failures. The maximum block size is approximately one foot.

Site 956, also known as "Perlite Debris Slide", is a slope with an axial length of 80 feet and a slope angle of approximately 39° that affects 45 feet of the Denali Park Road. The failures consist of rotational debris slide events in rhyolite, perlite, and colluvium materials. These events are triggered in part by a natural spring emitting from a geologic contact between the impermeable perlite layer and the rhyolitic colluvium in the slope. A large event on August 26, 2015 blocked and closed the road for two hours during the day. The road was also closed overnight after this event for additional debris cleanup.

Problem Correction:

There are two options for risk reduction for these two unstable slope modes at this one site.

The first option would consist of installing about 10 horizontal drains approximately 50 feet long to capture the groundwater along the perlite and rhyolitic colluvium geologic contact to dewater the slope and reduce pore pressures. This would be paired with establishing a rockfall catchment ditch 12 feet wide with a 1V:4H ditch foreslope. Annual maintenance of the ditch and debris removal will be required to maintain the same level of risk reduction. The horizontal drains would also require periodic maintenance including jetting to remove any organic algae and soil material obstructions.

Based on recent LiDAR comparisons from 2018 to 2020 showing increased activity and the unstable slope retrogressing up the very steep slope, it is our opinion that this second option will offer the greatest benefit for risk reduction. The second option would consist of installing a cantilevered soldier pile wall backfilled with pervious rock material and installing horizontal drains (that possibly extend through the face of the wall) as described in Option 1 above. This option would require a geotechnical investigation for retaining wall foundation design. We have assumed the wall will be 15 feet high and 140 feet long and retaining wall tiebacks are not appropriate for this location due to the very weak strength of the perlitic ash materials in the cut slope. A conveyer belt or long-reach excavator will be required to backfill the wall up to the approximate spring location where the piping failures are occurring.

Geotechnical Estimating Factors:

Option 1 - Horizontal Drains and Rockfall Catchment Ditch

Item	Unit	Amount	Unit Cost	Total Cost
Geotechnical Design	Lump Sum	1	$ 25,000	$ 25,000
Ditch Excavation	Cubic Yards	100	$ 50	$ 5,000
Horizontal Drain Installation	Lineal Foot	500	$ 250	$ 125,000
		Estimated Costs		$ 155,000

Option 2 - Retaining Wall and Horizontal Drains

Item	Unit	Amount	Unit Cost	Total Cost
Geotechnical Investigation and Design	Lump Sum	1	$ 80,000	$ 80,000
Cantilevered Soldier Pile Wall	SqFt of Face	2,100	$ 300	$ 630,000
Retaining Wall Backfill	Cubic Yard	2,500	$ 100	$ 250,000
Horizontal Drain Installation	Lineal Foot	500	$ 250	$ 125,000
		Estimated Costs		$ 1,085,000

Completed By: DAA/NJF/BC Date: July 31, 2020

Denali National Park Unstable Slope Corridor Assessment

Site 933 (DENA_USMP_017)

Agency: NPS
Region: Alaska Region
Road: Denali Park Road (Road 10)
Side of Centerline: R

Beginning MP: 45.27 ***Ending MP:*** 45.32

Numerical Rating: 435

Problem Definition:

This unstable slope has a maximum slope height of approximately 187 feet and is approximately 430 feet long. The slope is composed of loose, rhyolitic rock that is producing primarily planar and wedge structurally controlled rock failures. The maximum block size is approximately one foot. Sight distance and the rockfall ditch is limited through this section.

Problem Correction:

At a minimum, rock scaling should be performed as a temporary risk reduction measure along the slope to remove loose, precariously positioned rocks. The rock scaling would need to be repeated every five to ten years to maintain the same level of risk reduction. For a higher level of risk reduction (Option 2), rock scaling should be completed and large, unsupported rock features would require rock reinforcement with rock bolts and dowels. Cleaning and maintaining the existing ditch will also be required to preserve its effectiveness to contain future rockfall.

Geotechnical Estimating Factors:

Option 1: Scaling Only

Item	Unit	Amount	Unit Cost	Total Cost
Geotechnical Design	Lump Sum	1	$ 30,000	$ 30,000
Mechanical Scaling and Debris Removal	Hour	30	$ 2,000	$ 60,000
Scaling and Debris Removal	Scaler Hour	80	$ 400	$ 32,000
		Estimated Costs		$ 122,000

Option 2: Scaling and Rock Reinforcement

Item	Unit	Amount	Unit Cost	Total Cost
Geotechnical Design	Lump Sum	1	$ 40,000	$ 40,000
Rock Reinforcement Installation	Lineal Foot	1150	$ 250	$ 287,500
Mechanical Scaling and Debris Removal	Hour	30	$ 2,000	$ 60,000
Scaling and Debris Removal	Scaler Hour	80	$ 400	$ 32,000
		Estimated Costs		$ 419,500

Completed By: DAA/NJF/BC Date: July 31, 2020

Denali National Park Unstable Slope Corridor Assessment

Site 955 (DENA_USMP_029)

Agency: NPS
Region: Alaska Region
Road: Denali Park Road (Road 10)
Side of Centerline: L

Beginning MP: 44.71 ***Ending MP:*** 44.83

Numerical Rating: 440

Problem Definition:

This unstable slope, known as the "Bear Cave Slump", is a rotational landslide that has an axial length of approximately 1,000 feet and could impact nearly 1000 feet of the Park Road. Currently, it is starting to impact approximately 300 feet of the Denali Park Road. The landslide headscarp is located just below the road and headscarp erosion and regression continues to impact the traveled way. The landslide was improved during a project in the late 1990s, during which subsurface and surface drainage was redirected away from the landslide area to a nearby culvert with the installation of a deep cutoff trench lined with geotextile located in the uphill ditch. WFLHD has previously investigated the landslide in the 1990s and the landslide is still active below the road, but since the construction of the drainage improvements described above, the annual movement toward the roadway has decreased. Park Geology staff continues to monitor the regression of a portion of the landslide headscarp toward the roadway with periodic GPS surveys. Between 2018 and 2020, the landslide headscarp has retrogressed to within feet of the toe of the roadway embankment.

Problem Correction:

There are several measures that can be taken for risk reduction of this slope. We present two of the most desirable options. They include the following:

Option 1: Realignment

- No additional geotechnical subsurface investigation is anticipated for this option.
- Shift the roadway up slope away from the failure area per existing conceptual design plans developed by WFLHD in 2020 for about 1300 lineal feet of realignment and upslope cutoff trench. This is estimated, under current climatic conditions, to be about a 20 to 30 year design before landslide retrogression may become problematic again. As noted, this option is not full proof because the landslide could surge with dramatic movement again, as it did in the 1990's.
- Installing additional surface drainage as detailed in the 2016 Spring Road Opening report by WFLHD. This includes the installation of a cross culvert up gradient of the failure area. This will intercept surface water and direct it across the roadway into a natural swale so it does not drain down toward the head of the landslide.

Option 2: Buried Cylinder Pile-Type Wall

- A subsurface geotechnical investigation including up to 6 test borings with slope monitoring instrumentation will be required to characterize the landslide and determine the appropriate risk reduction alternatives. Following the geotechnical subsurface investigation, more cost effective measures than the one provided here may be realized.
- Install the same cross-culvert as in Option 1.
- Assume the installation of drilled shafts (cylinder piles) along the outside of the current Park Road alignment. The shafts will likely be 6 to 8 feet in diameter and will be at least 40 feet deep to provide adequate resistance to stabilize the road and upslope area if the landslide continues to move down slope, in front of the buried structural wall. This option would be considered a mitigation of the landslide movement barring unforeseen subsurface conditions observed during the proposed subsurface geotechnical investigation.

Geotechnical Estimating Factors:

Option 1 - Realignment

Item	Unit	Amount	Unit Cost	Total Cost
Geotechnical Design	Lump Sum	1	$ 30,000	$ 30,000
Cross Culvert Installation	Lump Sum	1	$ 10,000	$ 10,000
Roadway Realignment (~1300 feet) plus cutoff trench	Lump Sum	1	$ 300,000	$ 300,000
		Estimated Costs		$ 340,000

Option 2 - Buried Cylinder Pile Wall

Item	Unit	Amount	Unit Cost	Total Cost
Geotechnical Investigation and Design	Lump Sum	1	$ 250,000	$ 250,000
Cross Culvert Installation	Lump Sum	1	$ 10,000	$ 10,000
Buried Cylinder Pile Wall (850-1000 ft)	Lineal Foot	6800	$ 2,500	$ 17,000,000
		Estimated Costs		$ 17,260,000

Completed By: NJF/DAA/BC Date: August 2, 2020

Denali National Park Road Unstable Slope Corridor Assessment

SITE 957 (DENA USMP 012)

Agency: NPS
Region: Alaska Region – Denali National Park and Preserve
Road: Denali Park Road (Road 10)
Side of Centerline: L and R

Beginning MP: 45.34 ***Ending MP:*** ***45.41***

Numerical Rating: 948

Problem Definition:

This unstable slope, known as the "Pretty Rocks Slump," is a large, ice-rich landslide that impacts approximately 310 feet of the full width of the Park Road. The landslide headscarp is currently about 150 feet in slope distance upslope of the road and movement has been observed at the toe of the landslide on the valley floor approximately 1300 feet in slope distance below the Park Road. Recent test boring and instrumentation suggest that landslide movement is between 40 and 80 feet below the Park Road. Movement was first observed in the 1980's and since 2014 has increased rapidly. As of August 2019, portions of the landslide at the road were moving at about 2 inches per day. Buses currently stop before entering the landslide area, and they proceed slowly.

In April 2020, an emergency contract placed about 5000 cubic yards of aggregate on the subsiding roadway to bring the Park Road back up to grade for the tourist season. The accelerating movement of this landslide is becoming difficult to maintain by the Park, so WFLHD has worked with Denali National Park and the Alaska Region of the National Park Service to investigate the cause of the landsliding in 2018 and characterize it for possible conceptual solutions. Denali National Park selected two conceptual solutions presented to them to move forward into the proof of concept stage: remove the upper landslide, build the road into very weak rock, and place it at the bottom of the mountain or bridge the landslide. In order for this work to be analyzed for fatal flaws, an additional geotechnical investigation was completed in 2019 and analyses were documented and provided to the Park in 2020 in two geotechnical memorandums.

Problem Correction:

The option to remove the upper landslide, build the road into very weak rock, and place material at the bottom of the mountain ended up being an intermediate solution because the very weak rock material that the road would be placed on is highly erodible and once exposed to the atmosphere, will break down rapidly.

The preferred option, and the one presented in this conceptual design, is bridging the landslide from one strong rock layer to another, allowing the landslide to continue to fail below the span of the bridge. The bridge will need to be approximately 400 feet long and rock reinforcement of the bridge foundations will be required. Under the bridging option, Site 3177, the basaltic rock cut to the west of the landslide will need to be excavated to provide for turning at the west side of the bridge. An additional test boring will be required at the west abutment location to confirm the complex geologic conditions that will influence the foundation design requirements.

Geotechnical Estimating Factors:

Item	Unit	Amount	Unit Cost	Total Cost
Geotechnical Investigation and Design	Lump Sum	1	$ 100,000	$ 100,000
Roadway Excavation and Bridge Option	Lump Sum	1	$15,000,000.00	$ 15,000,000
		Estimated Costs		$ 15,100,000

Completed By: DAA

Date: August 2, 2020

Denali National Park Unstable Slope Corridor Assessment

Site 3177

Agency: NPS
Region: Alaska Region
Road: Denali Park Road (Road 10)
Side of Centerline: R

Beginning MP: 45.41 ***Ending MP:*** 45.48

Numerical Rating: 416

Problem Definition:

This unstable slope has a maximum slope height of approximately 120 feet and is approximately 400 feet long. The slope is composed of decomposing basalt and rhyolite that is producing structurally controlled planar, wedge, and indeterminate failures. These failures consist of either blocks with a maximum block size of three feet or debris slide events with a maximum volume of six cubic yards. The existing slope is oriented at approximately 70° from horizontal.

Problem Correction:

The slope should be laid back from the existing 70° from horizontal to 53°. In addition, certain large structurally controlled features at either end of the rock outcrop should be strategically reinforced with rock bolts.

Geotechnical Estimating Factors:

Item	Unit	Amount	Unit Cost	Total Cost
Geotechnical Design	Lump Sum	1	$ 15,000	$ 15,000
Slope Excavation	Cubic Yards	23,000	$ 50	$ 1,150,000
Rock Reinforcement	Lineal Foot	670	$ 250	$ 167,500
		Estimated Costs		$ 1,332,500

Completed By: NJF/DAA/BC Date: October 5, 2018

APPENDIX C

WESTERN FEDERAL LANDS HIGHWAY DIVISION – FEDERAL HIGHWAY ADMINISTRATION

GEOTECHNICAL REPORT 05-20
AK NPS DENA 10(49)

GEOTECHNICAL SUMMARY REPORT OF EXISTING CONDITIONS

FINAL

PROJECT NO.:	2000003	DATE:	August 20, 2020

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY
701 12th Street – Suite 211
Golden, CO USA 80401
Telephone (720) 598-5982

August 20, 2020
Project No.: 2000003

Brandon Stokes, PE, Project Manager
Western Federal Lands Highway Division, FHWA
610 East Fifth Street
Vancouver, WA 98661

Dear Mr. Stokes,

Re: Geotechnical Report 05-20, AK NPS DENA 10(49), Geotechnical Summary Report of Existing Conditions

This report presents the summary of understanding of existing conditions in August 2020 for the Polychrome Pass portion of the Denali Park Road and surrounding areas in Denali National Park and Reserve, Alaska. It is based upon a review of past work conducted by Western Federal Lands Highway Division (WFLHD) and their contractors, and Denali National Park, as well as a literature review.

Review of these data sources is ongoing as additional data are being acquired and reviewed. A primary source of new data is satellite InSAR that was recently collected to judge past movement, help identify areas of interest, and provide a baseline for any future InSAR surveys.

An "interim" version of this report was submitted on April 1, 2020 in advance of the Expert Based Risk Assessment (EBRA) meeting held May 5-7, 2020.

During the EBRA meeting, the expert panel identified a section of Option 3A that could be rerouted to lessen geotechnical risk. WFLHD provided an updated Option 3A alignment, and the expert panel reconvened on June 22, 2020 to asses two realigned segments of Option 3A. Both versions of Option 3A are shown in Figure 3-8.

Part of the basis of the update to this report is a change to Option 3A subsequent to the EBRA meeting. The following has been updated in this final version of the report:

- Section 3.4.1 GIS Intersection Analysis
- References to the length of Option 3A throughout the report
- Drawings 01 – 05 in Appendix A.

The objective GIS comparisons (Section 3.4.1) reflect the changed alignment, but other imagery, presented results, proposed investigation plans, etc. have not been adjusted for the new alignment. The change is small and doesn't have significant impact to these items.

Yours sincerely,

BGC ENGINEERING INC.
per:

Scott A. Anderson, Ph.D.
Principal Geotechnical Engineer

Copies:
Mr. James Potts, P.E., Jacobs Engineering

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LIMITATIONS

BGC Engineering Inc. (BGC) prepared this document for the account of Western Federal Lands Highway Division – Federal Highway Administration, Western Federal Lands Highway Division and Jacobs Engineering. The material in it reflects the judgment of BGC staff in light of the information available to BGC at the time of document preparation. Any use which a third party makes of this document or any reliance on decisions to be based on it is the responsibility of such third parties. BGC accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this document.

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1.0 INTRODUCTION

The Federal Highway Administration (FHWA) is providing engineering geology and geotechnical assistance to the Denali National Park (DENA) and the Alaska Region (AKR) of the National Park Service (NPS) for the Pretty Rocks Landslide in Denali Borough, Alaska. Landslide movement is increasingly impacting approximately 350 feet of the Denali Park Road at approximately MP 45.3 (Figure 1-1). The landslide impacts the road in the section between approximately MP 43 and MP 48 where it crosses Polychrome Pass. In this section there are also other landslides and signs of active or past slope movement (Figure 1-2).

The Denali Park Road is the primary access into the Park and is 92 miles long, starting from the Parks Highway south of Healy and ending in Kantishna to the west. The need to keep the road open and safe, and the deterioration over Polychrome Pass and rapid acceleration of the Pretty Rocks Landslide since 2014, has resulted in consideration of alternative routes. The concern is that the existing route over Polychrome Pass may not be sustainable and an alternate route will be needed to access the Toklat Road Camp, Eielson Visitor Center (EVC), and the historic views of Denali that can be seen from Stony Overlook to Wonder Lake to Kantishna. For this reason, the NPS, with assistance from WFLHD, has considered several alternatives and determined that a north alignment (Option 2 – 6.0 miles long) and two south alignment options (Option 3A and 3B – 6.2 and 5.3 miles long, respectively) should be compared to capital improvements to the existing alignment (Option 1 – 6.4 miles long). Current understanding of the existing conditions on these alternate alignments is also included in this report.

This report has been prepared by BGC Engineering Inc. (BGC) through subcontract with Jacobs Engineering Inc. under Contract No. DTFH7015D00004, Task Order No. 69056720F000025, AK NPS DENA 10(49), Polychrome Pass Alternatives Analysis dated December 10, 2019. The report serves a few purposes, primarily to:

- Summarize past work that has been performed by WFLHD and others
- Present new satellite InSAR results and their interpretation
- Identify gaps in understanding that can be addressed by a subsequent investigation program
- Provide a summary document for communication during an upcoming expert-based risk assessment.

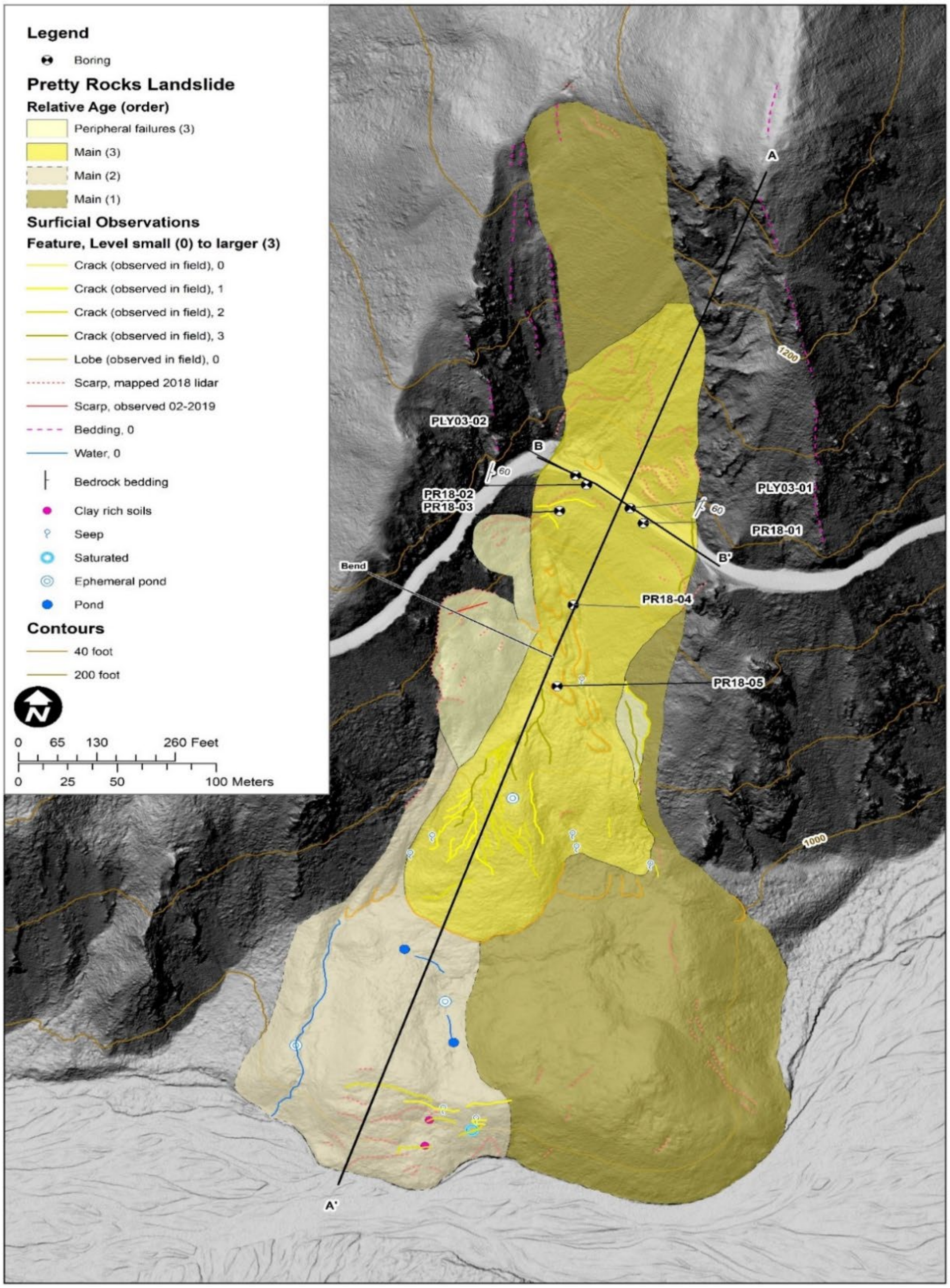


Figure 1-1. Pretty Rocks Landslide mapping and 2003 and 2018 test boring locations (Source: Task Order No. 69056720F000025).

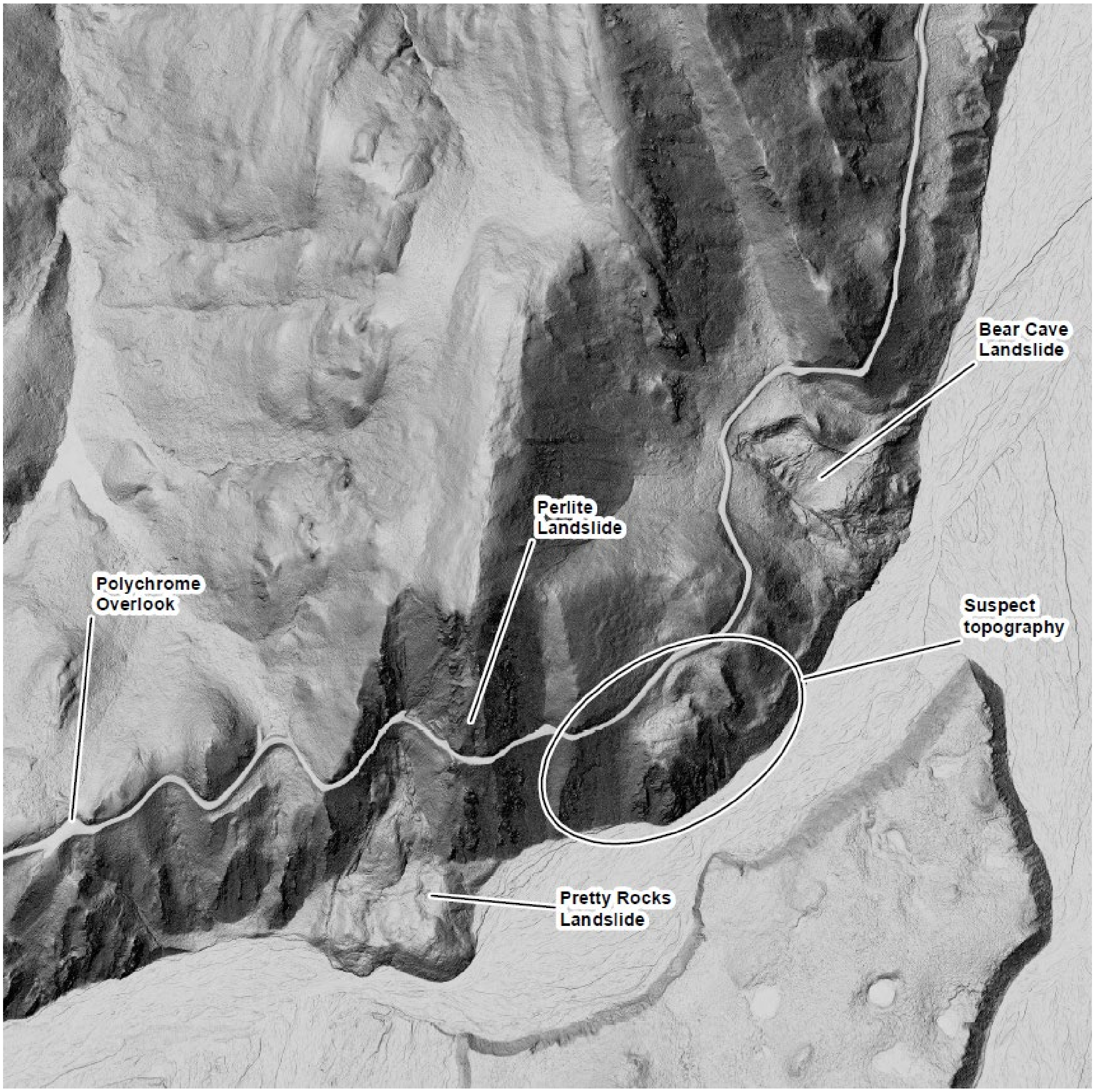


Figure 1-2. Other known landslides on Polychrome Pass (Source: Task Order No. 69056720F000025).

2.0 BACKGROUND

Along the Denali Park Road are over 140 unstable slopes with varying degrees of operational impact potential. There are three locations of particular concern within the Polychrome Pass area: Bear Cave Landslide (Mile Post (MP) 44.8), Pretty Rocks Landslide (MP 45.3) and the Polychrome Rest Stop/Outlook area (MP 45.8 to 46.2). The Pretty Rocks Landslide's rate of movement has increased in recent years. In Spring 2018, the road movement was measured at approximately 0.2 to 0.3 inches per day and it was difficult to maintain through the summer season by park maintenance crews. From September 2018 to March 2019, road surface movement measurements had increased to 0.4 inches per day. Following record warm average temperatures in the summer of 2019 and monsoonal rain events in August 2019, the rate of road subsidence has increased significantly at the Pretty Rocks Landslide. From August 2019 to January 2020, landslide surface change measurements have been, on average, 2 inches per day.

Denali National Park has also experienced warming temperatures over the last 14 years. A temperature analysis was conducted by NPS (2020a) to best characterize the changing conditions at the Pretty Rocks Landslide from 2006 to 2019. Figure 2-1 illustrates the increase in 12-month running mean temperatures at the Eielson Visitor Centre (EVC), Denali Headquarters (HQ) and at the Toklat River. This warming has changed the climatic regime to one where temperatures are now greater than 0 °C. Climate and soil conditions control permafrost stability and it tends to degrade at air temperatures greater than 0 °C (NPS, 2020a). The trend from past data and climate models indicate that most years will experience average mean annual temperatures over 0°C, soon after the construction of new roads in Denali National Park.

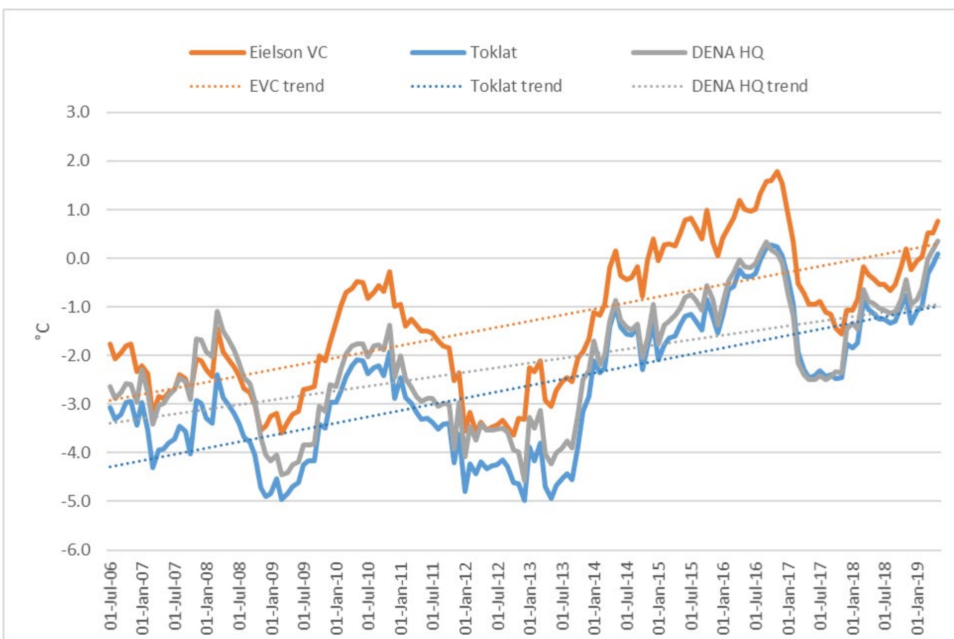


Figure 2-1. 12-month running mean temperatures at EVC (orange), Toklat (blue), and Denali Park HQ (grey) with 14-year linear trend (dashed lines) (NPS, 2020a).

3.0 ALTERNATIVE ALIGNMENTS

In addition to the existing Denali Park Road alignment, three alternative alignments are currently being considered. The general character of each alignment is briefly summarized below and shown on Drawing 01 in Appendix A.

3.1. Existing Alignment (Option 1)

The existing alignment traverses a precipitous section of road known as Polychrome Pass. Built in the 1920s and 1930s and known as the high-line route, this scenic section of road is at roughly the mid-way point on the 92-mile long road. The Pretty Rocks Landslide (Figure 3-1 and Figure 3-2) at Mile Point (MP) 45.3 is one of several known landslides in that general area. Recent data indicates the rate of movement in this area increased significantly during the late summer of 2019 following warm seasonal average temperatures in the region and historic summer rain events in August 2019.

A 6.4-mile section of the road, between approximately MP 42 and MP 48.4, is being considered for comparison with the proposed alternative north and south alignments. The alternative alignments would bypass this section of the road. For the EBRA, Pretty Rocks Landslide and Bear Cave Landslide are assumed to be mitigated according to WFLHD's Polychrome Pass Project Delivery Plan (1st Revision), and all the Unstable Slope Management Program (USMP) sites would be improved to at least a "fair" condition.

Figure 3-1. Denali Park Road at the Pretty Rocks Landslide. NPS photo (Date unknown).

Figure 3-2. Denali Park Road at the Pretty Rocks Landslide. FHWA photo (2018).

3.2. North Alignment (Option 2)

The proposed 6-mile-long north alignment would depart the existing alignment near the East Fork Toklat River Bridge (MP 43) and rejoin the road near MP 48. The alignment crosses several rivers and drainages, as well as several areas identified as permafrost and landslides. The general character of the landscape along the north alignment is shown in Figure 3-3 and Figure 3-4.

Figure 3-3. The north alignment traverses a valley. Photo location shown as #106 in Figure 3-7. FHWA photo (2019).

Figure 3-4. Landslide near north alignment. Note the stream at the bottom of the valley. Photo location shown as #103 in Figure 3-7. FHWA photo (2019).

3.3. South Alignments (Option 3A and 3B)

There are currently two proposed south alignments – Option 3A and Option 3B. The 6.2-mile and 5.3-mile-long south alignments would depart the existing alignment near the East Fork Toklat River Bridge (MP 42.1 and MP 44.3, respectively) and rejoin the road near MP 48. The south alignments traverse a broad valley with wide floodplains, discontinuous permafrost, and muskeg (Figure 3-6), and would bridge several active braided river and stream channels (Figure 3-5).

During the EBRA meeting, the expert panel identified a section of Option 3A that could be rerouted to lessen geotechnical risk. WFLHD provided an updated Option 3A alignment, and the expert panel reconvened on June 22, 2020 to assess two realigned segments of Option 3A. Both versions of Option 3A are shown in Figure 3-8.

Figure 3-5. Braided channel characteristic of the south alignment. Photo location shown as #014 in Figure 3-7 FHWA photo (2019).

Figure 3-6. Tundra characteristic of the south alignment. Photo location shown as #029 in Figure 3-7 FHWA photo (2019).

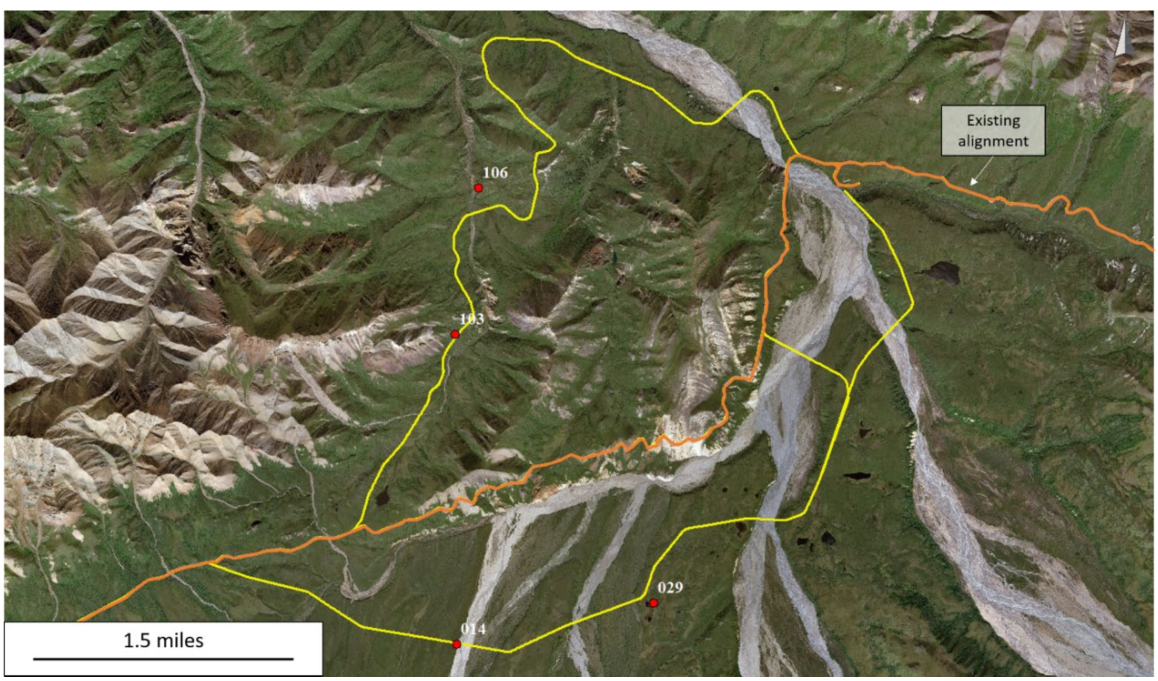


Figure 3-7. North and south alignment photo locations.

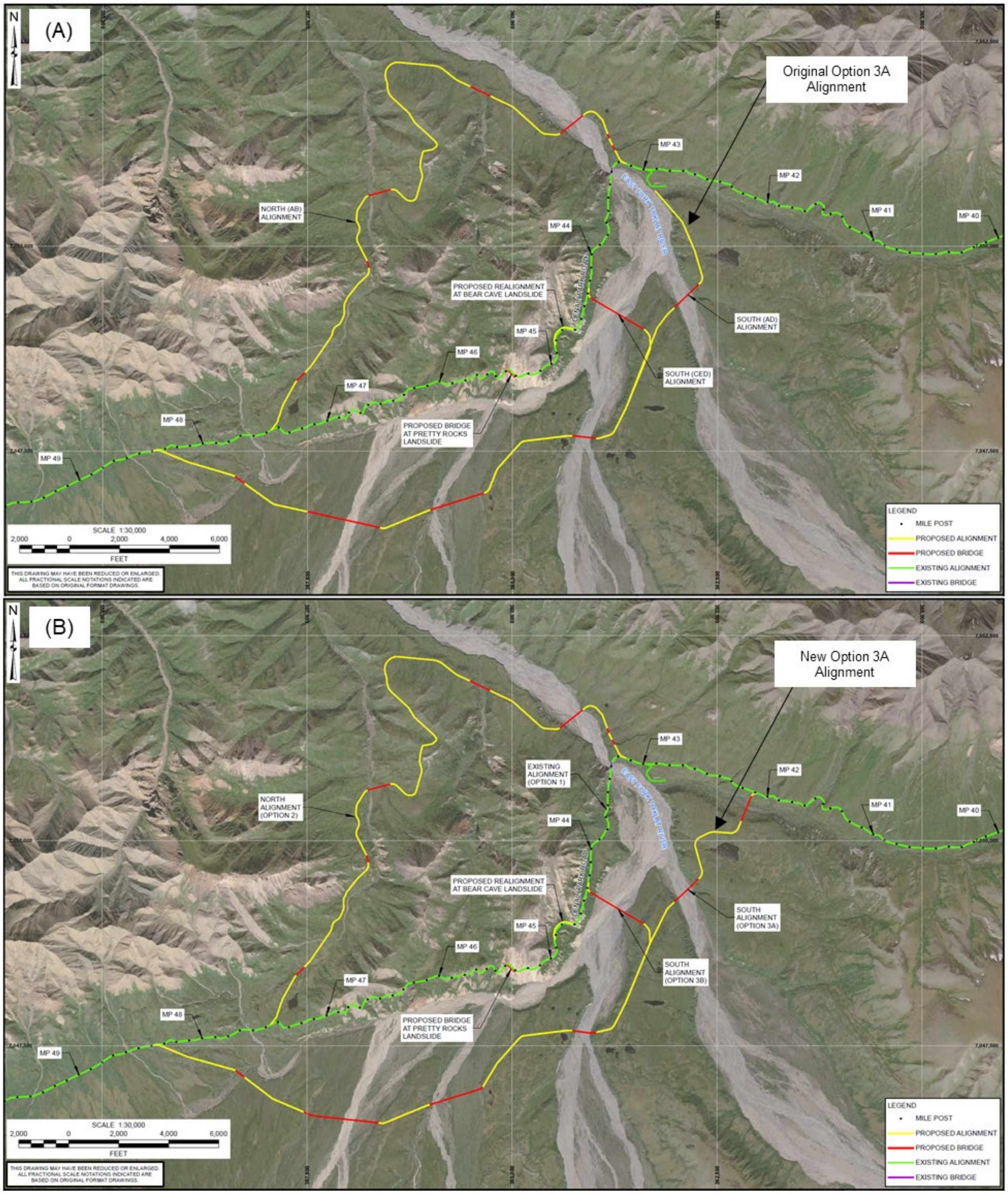


Figure 3-8. Overview map showing the original Option 3A alignment (A) and the new Option 3A alignment (B).

3.4. Quantitative Hazard Crossing Comparison

There is a considerable difference in the amount of geotechnical information available for each of the alignments. The performance of the existing alignment has been observed for nearly 100 years and there have been several investigations targeted at understanding the geological and geotechnical issues along the road. In contrast, there is very little known about the geological and geotechnical conditions along the north and south alignments; so far, knowledge is limited to what can be synthesized from the following:

- Review of air photos and satellite imagery (Drawing 01, Appendix A)
- Observations collected by FHWA in September of 2019 while walking along the proposed alignment corridors. This includes photos, geological hazard observation (e.g., landslides and permafrost) which is presented as geomorphic mapped units in Drawing 02 in Appendix A. Scattered ground temperature measurements along the south alignment are summarized in Table 3-1.
- Review of existing geological maps (Drawing 03 and Drawing 04, Appendix A)
- Satellite InSAR collected and processed by TRE ALTAMiRA in March of 2020 (Section 4.0)

Table 3-1. Temperature probe measurements.

Location	Alignment	Depth of Temperature Probe Below Ground Surface (ft)	Temperature (Celsius)
005	South	3	-0.4
006	South	2.5	-0.6
007	South	2	0
009	South	5	4.75
018	South	1.5	-0.4
019	South	0	1
020	South	2.5	-0.6
021	South	1.5	-0.8
027	South	3	6.4
032	South	2	3.1
034	South	2.5	2.1
035	South	2.5	-0.1
036	South	1	-0.8
040	South	0.5	3.6
042	South	3	-0.9
043	South	1	-0.6

3.4.1. GIS Intersection Analysis

To compare the alignments directly, an intersection analysis was performed in GIS to tabulate the cumulative length of each alignment crossing the geomorphic unit with associated hazards mapped by FHWA in 2019. The alignments and hazard polygons are shown on Drawing 02 in Appendix A and intersection analysis results are presented in Table 3-2 and Table 3-3. For the intersection analysis, each alignment is assumed to start at MP 42.0 and end at MP 48.4. This provides a common basis tied to the existing road.

For comparison between the alignments, the slopes have been classified in three categories: less than 20 degrees inclination, 20 to 34 degrees, and greater than 34 degrees. When analyzed for intersection with natural slope inclinations, most of the terrain that the alignments intersect is less than 20 degrees in slope (measured in any direction) (Table 3-2), ranging from approximately 61 percent of the total alignment length for the north alignment to 84 percent of the total alignment length for others. The existing alignment has the largest percentage of slope intersections greater than 34 degrees.

The north alignment has the largest percentage of slope intersections greater than 20 degrees. The south alignments do not have as much intersection with steep slopes, but they do intersect a higher percentage of 20- to 34-degree slopes when compared to the existing alignment.

The distribution in slope inclination intersections and the hazard mapping intersections provide an objective measure for developing comparisons and describing the general character of the alignments and associated hazards. For instance, the existing alignment has more length intersecting steep slopes than other alignments (Table 3-2) and has more length intersecting landslides (Table 3-3); the south alignments cross a larger percentage of flatter terrain with permafrost, muskeg, and flood/erosion hazards, and the north alignment is more of a mixture of geomorphic characteristics with permafrost, flood/erosion hazards, and mapped landslide (Table 3-3).

Despite the objectivity of the analysis and the general agreement with observed character, the numbers and the proportioning of the alignments should not be used alone. Geologic and topographic interpretations can be focused on certain areas based on the findings, and this will allow for informed comparison between the alignments.

Table 3-2. Summary of slope class along the existing alignment and the north and south alignments.

Slope Class (degrees)	Percentage and Length (ft) of Alignment[1]							
	Existing Alignment (Option 1)		North Alignment (Option 2)[2]		South Alignment (Option 3A)		South Alignment (Option 3B)	
	Percentage	Length	Percentage	Length	Percentage	Length	Percentage	Length
0 - 20	84%	28,517	61%	26,168	84%	27,742	84%	33,686
20 - 34	8%	2,634	20%	8,425	14%	4,501	12%	4,758
> 34	8%	2,706	4%	1,747	3%	829	4%	1422
Total	100%	33,856	85%	42,610	100%	33,071	100%	39,866

Notes:

1. Each alignment is assumed to start at MP 42.0 and end at MP 48.4, thereby providing a common basis tied to the existing road.
2. DEM missing for part of North Alignment. Only 85% of the North Alignment is accounted for.

Table 3-3. Summary of hazards along the existing alignment and the north and south alignments.

Hazard Type		Percentage and Length (ft) of Alignment[1]							
		Existing Alignment (Option 1)		North Alignment (Option 2)[2]		South Alignment (Option 3A)		South Alignment (Option 3B)	
		Percentage	Length (ft)	Percentage	Length (ft)	Percentage	Length (ft)	Percentage	Length (ft)
Permafrost	Discontinuous	-	-	32%	13,595	57%	18,792	35%	13,757
Flood/ Erosion	Active Braided Channel	0.5%	155	3%	1,157	11%	3,650	11%	4,281
	Lower Terrace	1%	273	1%	637	12%	3,927	10%	4,112
	Upper Terrace	11%	3,890	14%	5,976	69%	22,931	53%	20,943
Fans	Debris Fan	-	-	3%	1,193	-		-	
	Alluvial Fan	1%	340	1%	340	-		-	
Muskeg	Muskeg	-	-	-	-	3%	1,085	3%	1,003
Landslides	Confirmed	1%	302	-	-	-	-	-	-
	Likely	-	-	-	-	-	-	-	-
	Uncertain	5%	1,641	4%	1,495	-	-	-	-
Alignment Length			**33,856 (6.4 miles)**		**42,610 (8.1 miles)**		**33,071 (6.3 miles)**		**39,866 (7.6 miles)**

Notes:

1. Each alignment is assumed to start at MP 42.0 and end at MP 48.4, thereby providing a common basis tied to the existing road.
2. DEM missing for part of North Alignment. Only 85% of the North Alignment is accounted for.

4.0 SATELLITE INSAR

To improve spatial and temporal understanding of the prior deformation patterns along the proposed alignments, BGC contracted TRE ALTAMiRA (TRE) to collect and process satellite-based interferometric synthetic aperture radar (InSAR) data for an area covering the alignment options. As there is a regularly collected archive of data available (2015 –present) using the European Space Agency's (ESA) Sentinel-1 SAR satellite, there is the opportunity to look back to assess existing deformation patterns to support the preliminary requirements for the site investigation. A report from TRE outlining the details of the SAR data, processing techniques and outputs is provided in Appendix B.

For this study, two beams/tracks of Sentinel 1 data were used. Details are provided in Table 4-1.

Table 4-1. Sentinal 1 descending and ascending track details.

Geometry	Look Direction	Incidence Angle	Repeat Frequency	Collection Period
Descending	WNW	36.74	12 day	2018-05-14 to 2019-09-30
Ascending	ENE	40.69	12 day	2015-04-20 to 2020-01-06

Processed data is delivered by TRE via their TREMaps viewer in the following formats:

- SqueeSAR™ Line-of-sight (LOS) Data (ascending and descending): This data set represents points where consistent high-quality data points are observed throughout the entire period of monitoring. These data points are called permanent or distributed scatterers (TRE ALTAMiRA Inc., 2020). Typically for this technique to provide effective results at least 15-20 scenes of data are analyzed, and trends can be plotted to understand the temporal movement patterns (Figure 4-1).
- SqueeSAR™ 2D Motion: With the above processing, where there are common points for which permanent scatterers (PS) and distributed scatterers (DS) are identified with both ascending and descending mode data, then the vertical and east/west components of the deformation can be reported. This is especially useful in supporting the understanding of the dominant direction of movement of the ground movement (i.e., Slope movement vs. Subsidence) but provides less value if there are dominant north/south components of movement, which are largely blind to the satellites (Figure 4-2).
- Temporary Coherent Scatterers (TCS): The points provide a broader spatial coverage and incorporate data that is observed over multiple time frames but cannot be continuously tracked as a point across the monitoring interval. Therefore, the trends value are reported but time series plots are not available (Figure 4-3).

By integrating the above data sets with a knowledge of the geology, landforms and site conditions, inferences can be made around the patterns and styles of movement on the ground surface. For each of the proposed alignments, these data sets were reviewed and areas where deformation patterns were observed were assessed and comments provided to support the planning for field observations or a focused preliminary geotechnical investigation. Preliminary comments from

InSAR review completed by BGC are summarized in Table 4-2 and a key to the approximate location of these observations is provided in Figure 4-4.

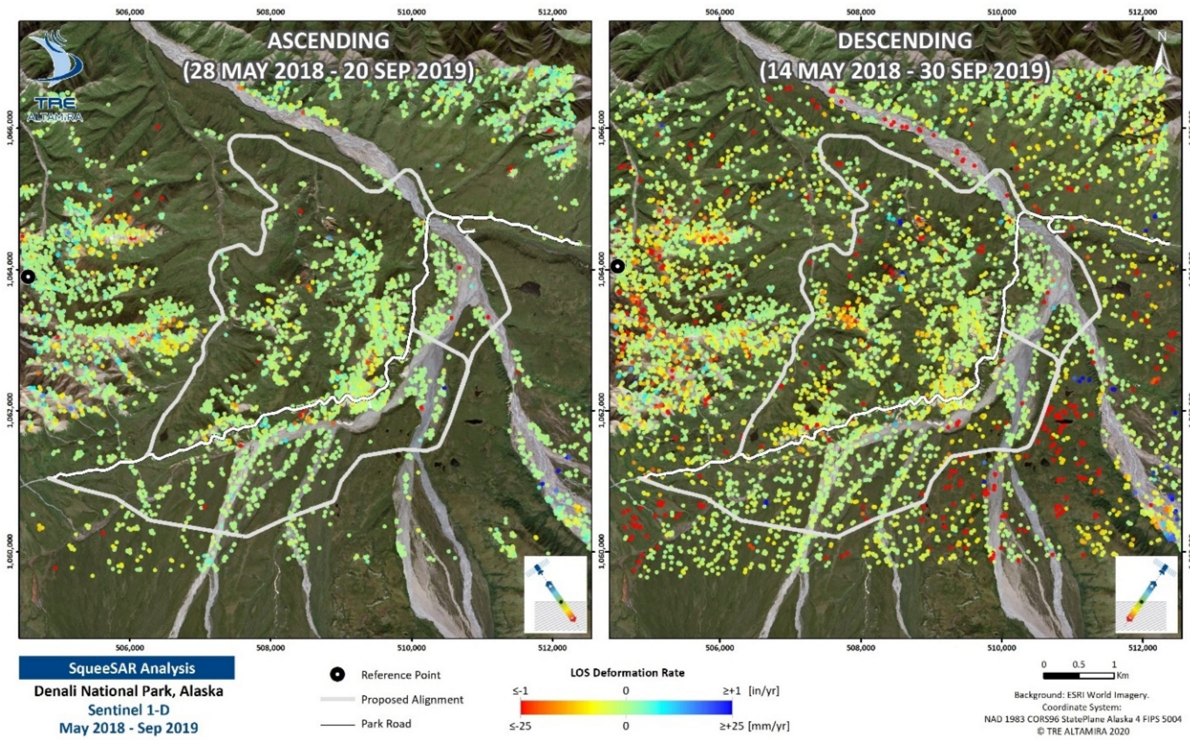


Figure 4-1. SqueeSAR™ Line-of-sight (LOS) (full-size figures are included in Appendix B).

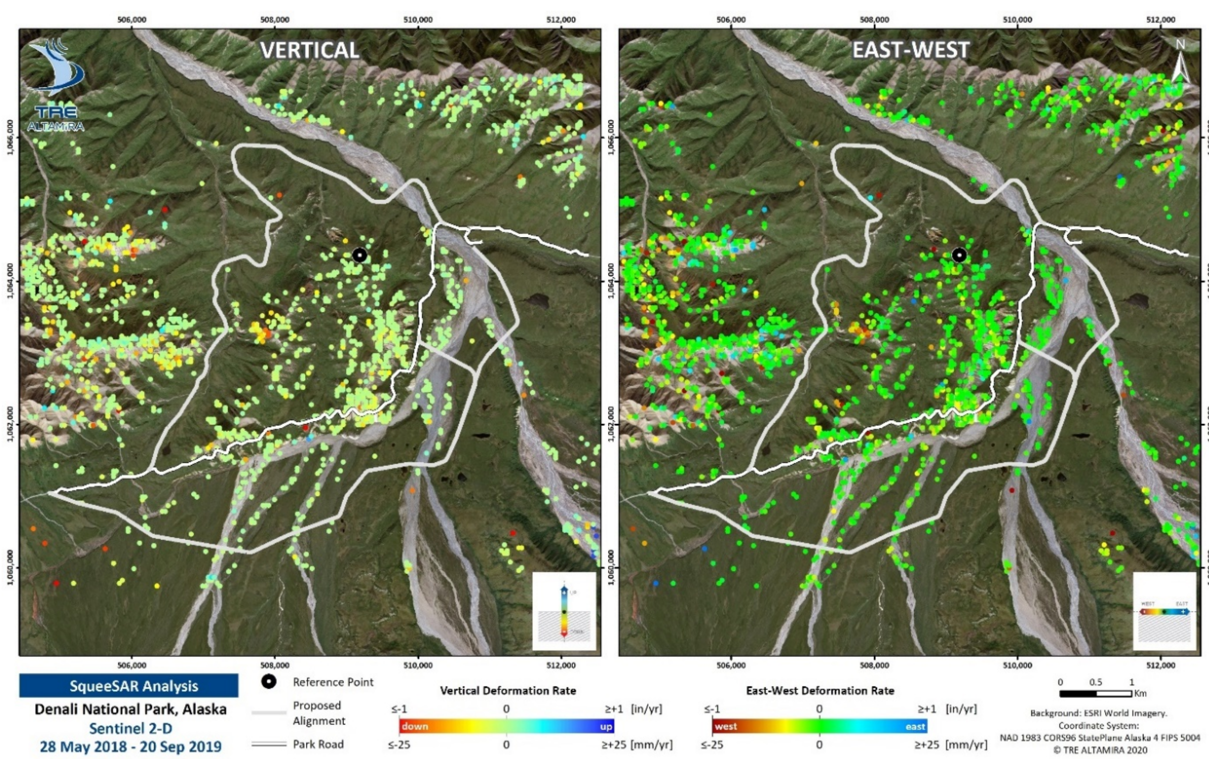


Figure 4-2. SqueeSAR™ 2D Motion (full-size figures are included in Appendix B).

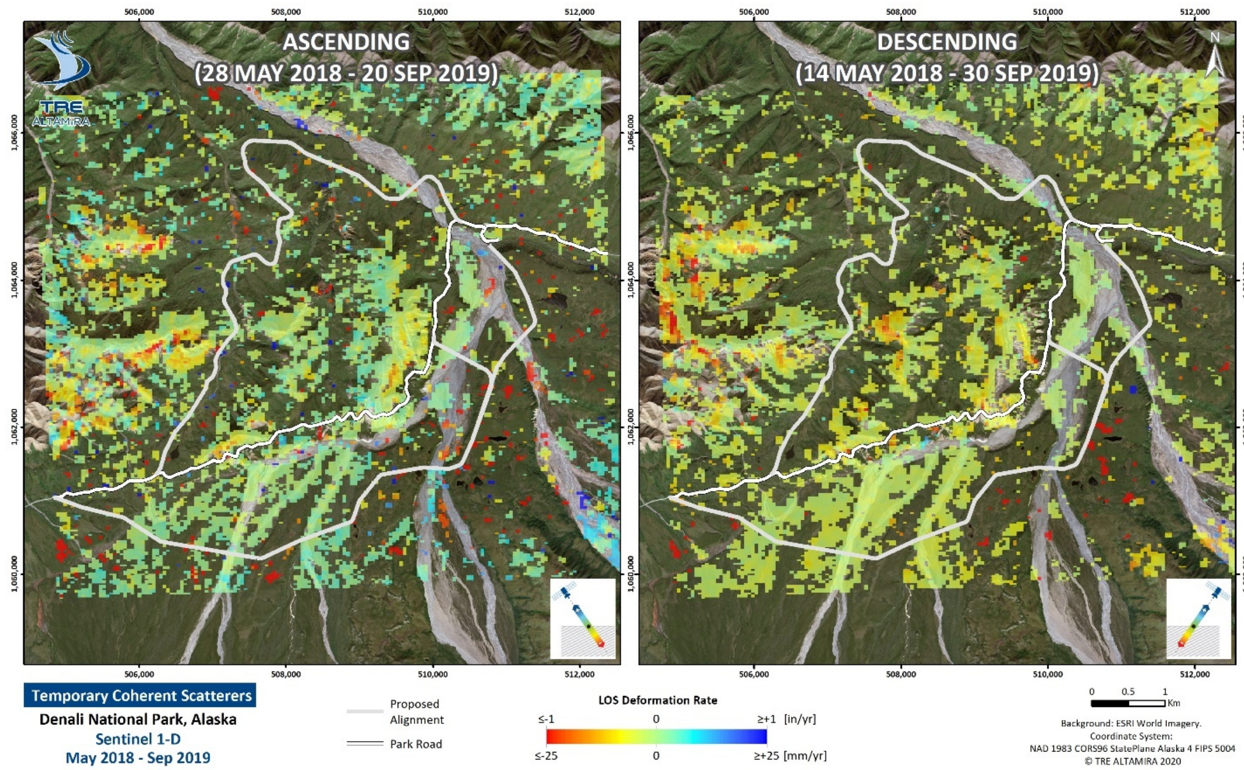


Figure 4-3. Temporarily Coherent Scatterers (TCS) (full-size figures are included in Appendix B).

Table 4-2. InSAR observation for locations on Figure 4-4.

Comment Location	Comment
1	Both the ascending and descending TCS data indicated spatial deformation trends but there are no PS/DS points available to assess temporal trends.
2	Signs of movement with pronounced horizontal component are observed in the ascending and descending TCS data. Permanent scatterers upslope of the proposed alignment (on upslope apron) indicated LOS deformations of up to 30 mm/year (1.2 in/year) LOS but data a along the proposed alignment does not provide indication of LOS deformations. It will be important to review centerline location in relation to the slope apron.
3	There is evidence of LOS deformations up to 20 mm/year (0.8 in/year) in the ascending TCS and DS/PS data across and below alignment at this location. The LOS deformations appear localized near the steeper slope scarp and do not appear to progress further upslope. Watch for offset of alignment from crest of slope.
4	The TCS LOS data indicates some very minor movement down slope (10 mm/year (0.4 in/year)) in this general area. Will need to assess grading requirements and impacts of disturbance.
5	There are indications of some LOS movements on over-steepened slope in TCS data.
6	There are a few ascending TCS pixels in this area that are showing LOS deformations in the range of 25 mm/year (1 in/year). It will be important to look to be along a slope so focus visual observations and investigations on trying to understand the mechanics as

Comment Location	Comment
	to what is happening in this zone as there are no reliable PS/DS data points available to discern temporal trends.
7	Although data is generally sparse in this area, the available TCS data pixels do not highlight any LOS deformations.
8	There is a grouping of ascending TCS pixels (no descending) which could be indicative of downslope movements visible to the ascending LOS (East-Northeast) but not the descending LOS (West Northwest).
9	There is some indication of localized LOS deformation on the approach to the river crossing in the both the ascending and descending TCS pixels.
10	There are distinct LOS deformations observed in both the ascending and descending TCS pixels in this area. It will be important to understand the mechanics of deformations in relation to proposed road grading.
11	There are signs of activity in this area.
12	From this point, moving to the south there is a consistent downward motion observed in both ascending and descending data. As similar trends are observed from both satellite geometries there is likely a dominant vertical component to the deformations.
13	Data coverage in this area is sparse but there are descending PS/DS points just off alignment showing up to 30 mm/year (1.2 in/year) LOS deformation. This could be organic terrain but will require a closer look in the field.
14	A few ascending TCS pixels indicate downslope movement with LOS deformations of up to 25 mm/year (1 in/year). It will be important to have a close look at cross slopes and target investigation on understanding of mechanisms and impact of grading for road construction.
15	There are signs of deformation on the fringes of the known landslide. Likely the movements are too fast within landslide to measure with InSAR.
16	Indications of slow systematic LOS deformations above road in this area in both the ascending and descending TCS pixels.
17	There is sparse data in this area but signals of slow deformation (likely less than 10 mm/year (0.4 in/year) are observed in the TCS/DS/PS LOS.
18	Indications of LOS deformation coming across the road alignment in this location.
19	Strong movement trends in ascending LOS TCS below road.
20	Continued indications of subsidence in the range of 20-25 mm/year (0.8-1 in/year) in LOS for both Ascending and Descending geometries.
21	Definitive signs of downslope movement in this area with rates up to 25 mm/year (1 in/year) in ascending LOS and slower movement in descending LOS. This would be indicative of a stronger trend to the east.
22	Systematic LOS deformations observed in the ascending TCS/PS points in the bare slopes upslope of road.

Comment Location	Comment
23	A cluster of TCS pixels are showing LOS deformations in the range of 20mm/year (0.8 in/year) in this area with a couple of PS data points exhibiting the same LOS deformation trend. This area is worth consideration in field investigations/observations.
24	There are a couple of descending PS/TCS pixels are showing movements in the order of 10 mm/year (0.4 in/year) LOS (west).
25	Some isolated LOS deformations downslope of road observed in ascending TCS (ENE)
26	There are both ascending and descending TCS pixels in this area showing LOS deformation (likely vertical) of up to 20 mm/year (0.8 in/year).
27	There are a couple of ascending TCS points showing LOS deformation up to 25 mm/year (1 in/year). This appears to indicate downslope movement to the east.
28	There are a few ascending and descending TCS pixels giving indication of up to 20 mm/year (0.8 in/year) LOS deformation in this area. Likely subsidence.
29	There is a defined zone of LOS deformation observed in both ascending and descending TCS that is moving away from descending geometry and towards ascending geometry (movement either vertically or to the west) but pattern is very consistent and in the range of 10 mm/year (0.4 in/year). These observations coincide well with a mapped discontinuous permafrost polygon. Observed trends may be indicative of a circular/slump type movement/subsidence.
30	There are general indications of vertical movements in LOS from both ascending and descending data up to 25 mm/year (1 in/year).
31	There are indications of LOS deformations in both ascending and descending TCS data on this slope
32	Generally, appears to be a more stable landform. No indications of LOS deformation from either ascending or descending geometries.
33	There are no indications of LOS deformation in either ascending or descending data.
34	There are indications of ground deformation in this zone that appear to be accentuated movement in the horizontal plane, possibly indicative of lateral slope movements. This has been predominantly observed in ascending TCS so more pronounced to moving to the east (no indication in descending TCS).
35	Vertical deformations observed in this landform up to 25 mm/year (1 in/year) LOS.
36	No signs of LOS deformation in this landform unit.
37	There are no indications of LOS deformations in this landform unit.

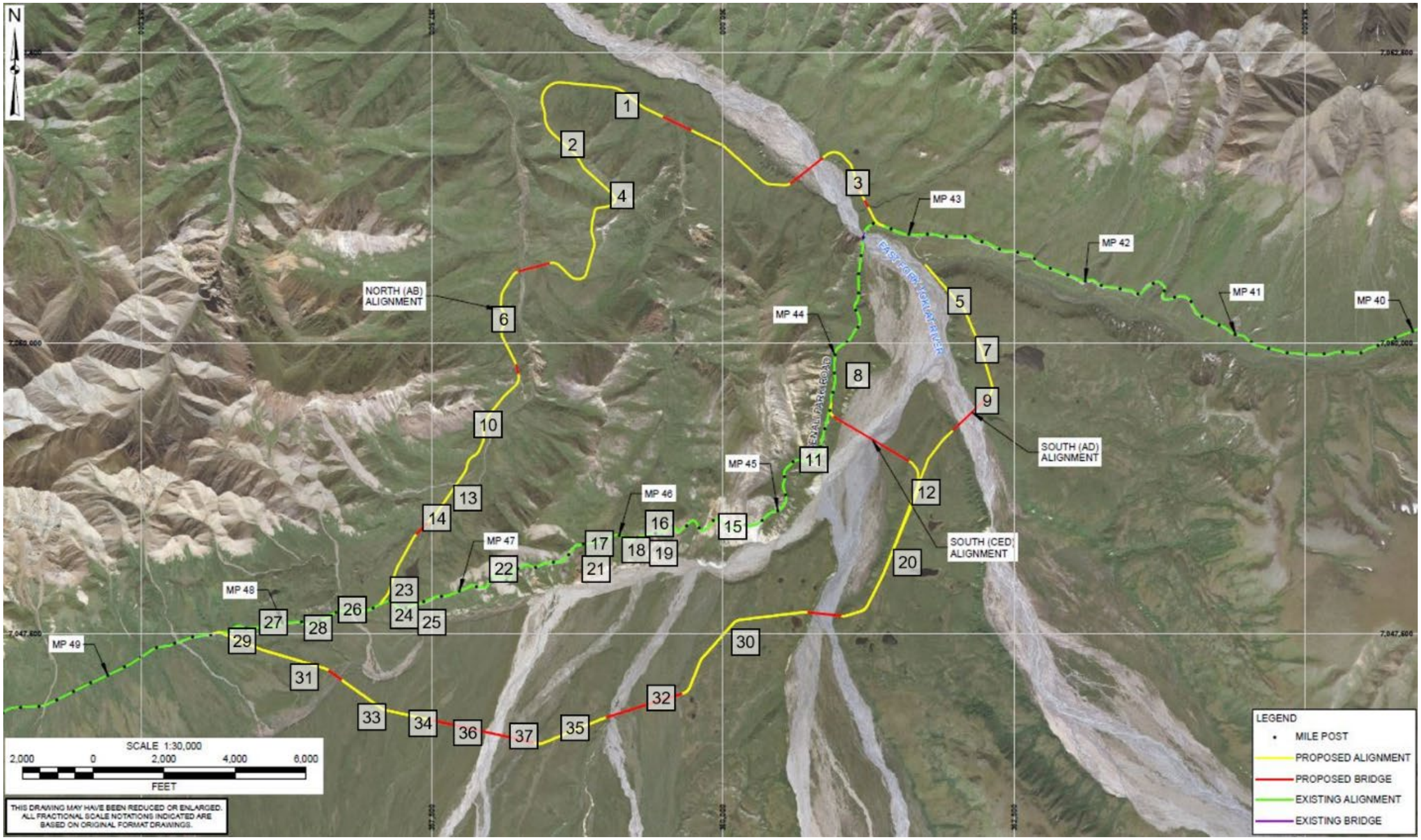


Figure 4-4. Location of InSAR observations from Table 4-2.

5.0 EXISTING ALIGNMENT

Construction of the Denali Park Road begin in 1922 and was completed in 1938. Denali National Park and Reserve is accessed via a single road, the 92-mile-long Denali Park Road. There have been numerous challenges associated with geohazards along this route, with over 140 known unstable slopes along the entire road. The park has invested in several geohazard investigations on the existing alignment dating from 1994 to 2019, as summarized in Section 5.1.

National Park Service records reviewed indicate several key milestones, observations and events:

1922: Construction of the park road began.

1924: NPS Assistant Director Arno Cammerer wrote a letter to Steese outlining guidance on scenic road construction.

1930: Denali Park Road had been constructed as far as the East Fork River. The next steps were to continue construction to the middle fork of the Toklat River via an area called Polychrome.

1931: The new section of the road was completed that summer (East Fork Bridge through Polychrome Pass).

1938: Road completed. National Park Service scenic road designers utilized curvilinear stretches and radial curves (instead of a series of tangents).

1957: Major advancements to improve connectivity to the larger road system. Denali Highway opened.

1966: Widening of the road to Teklanika River completed.

1968: Road to Savage River paved.

1971: George Parks Highway completed.

Pre-1980's Had to "sweeten up" once every 2-3 years across the Pretty Rocks Slump (MP 45.3).

1980s: Road was widened by 4 ft from Stony Creek to Eielson Visitor Center.

1987: Vertical "drop" movement at Pretty Rocks Slump requires heavy maintenance each year at the Pretty Rocks landslide. Day-labor type project installed geosynthetic reinforcement layers and subsurface groundwater cutoff trench in upper ditch line.

1990: Wet summer triggered movement at massive Bear Cave landslide (MP 45).

1991: Bear Cave landslide major scarp near the roadway first observed.

1990s: Continued landslide movement caused small cracks in road surface. Geotextile-lined trench installed on uphill side of road at Bear Cave Slump to redirect water to a culvert (away from the landslide). Movement of this slump has slowed since this mitigation.

2002: Polychrome rest stop slump began to develop in late summer 2002.

2003: August – 4 ft vertical drop measured at Polychrome rest stop slump, occurring over one week. Slump increases in speed with wet conditions (slows in drier periods). No movement was noted in slump until summer rains began early July. No instability prior to August 2002.

2004: Elevated roadbed in the Igloo Creek drainage.

2004: Pretty Rocks Landslide – vertical movement between 1 and 3 inches/month.

2009: Installed pullouts from MP 73 to 86.

2013: September, Igloo Landslide (Tattler Grade) MP 38 closes roadway with 20 to 40 feet of debris over it. Approximately one week is required to remove material from the roadway with a dozer working from the top of the mountain to the roadway. Material was sidecast and consisted of house to bus-sized blocks of frozen ground. Landslide shear zone was highly plastic fat clay from Teklankika Formation.

2014: Pretty Rocks Landslide- vertical movement increases.

2015: August 26 - Perlite Debris Slide at MP 45.32-MP 45.34 blocked and closed the Denali Park Road for two hours during the day. Road was also closed overnight for additional debris cleanup.

2016-2017: Pretty Rocks Landslide –a 300 ft section subsided up to 6 inches/month.

2018: Pretty Rocks Landslide - April-May - Movement was 6-9 inches/month. Day-labor type project installs deep patch across the landslide and brings road grade back up with 12% grade in and out of the body of the landslide. Used rock in the landslide headscarp with limited aggregate surfacing from Tek to minimize additional weight being added to the head of the landslide.

Sept 2018 -
Mar 2019: Pretty Rocks Landslide movement informed by drilling and instrumentation readings indicates movement at 0.4 inches/day or 12 inches/month. Dug down east edge as far as possible and reconstructed across landslide like 2017. Ground surface prism monitoring with total station indicates subsurface landslide movement corresponds to surface measurements at the road and below the road to where the slope softens.

2019: Spring – Pretty Rocks Landslide was displacing a 100-yard section up to 0.45 inches/day vertically and horizontally.

Aug 2019 -
Jan 2020: Landslide surface measurements are 2 inches/day or 5 ft/month.

5.1. Historical Records

Detailed road plans were never drafted; however, correspondence between Assistant NPS Director Arno Cammerer and ARC President James Steese were preserved and offer insight to the priorities and purposes of this park road. The NPS desired that the road be built where best possible views of the country were available, avoiding a straight-line approach, and cutting through terrain and vegetation to shorten the route (NPS, 2019). National Park Service Landscape Architect Thomas Vint had significant influence on direction of road building. He commented on the challenges of road construction in this area. “Construction is difficult and unusual in this type of country. It is first necessary to remove the moss cover and build ditches along the right of way to allow the subsoil to thaw and drain for a season. The next season the grading is done. For several seasons following the subsoil continues to thaw and settle so more or less grading must be done each year until the grade is established. Staged construction is necessary due to these special conditions. The standard of width is a one-way road with turnouts. This is ample for the traffic that will be using this road for many years to come” (NPS, 2019).

In 1929, Thomas Vint made a route recommendation that came to be known the “High Line Recommendation” and is the current alignment. The route option was identified as requiring “heavier” work, but was shorter, eliminated two bridges and was on “a more permanent location” compared to other route options (NPS, 2019). It avoided stream crossings but involved more excavation. Figure 5-1 and Figure 5-2 show the “High Line” route under construction. Figure 5-3 shows the high line road location, prior to construction, on the mountainside.

Figure 5-1. Drilling on the high line road, July 1930. (NPS, 2019).

Figure 5-2. High line road being built, August 30, 1931. (NPS, 2019).

Figure 5-3. Foot trail on permanent high line road location, July 1930. (NPS, 2019).

5.2. Landslide Characterizations

Pretty Rocks Landslide (MP 45.4) has increased in subsidence in recent years, to a point where park staff are unable to keep up with road maintenance and maintain reliable, safe access. The adjacent Bear Cave Landslide (MP 45) is also of concern in this area, however its movement has slowed in recent years until measurements of the landslide headscarp retrogression toward the roadway was observed following the August 2019 historic precipitation events. At its shortest distance, the headscarp of the landslide is within 10 feet of the road embankment now. A series of investigations have been conducted for the Bear Cave Landslide from 1994 to 2019 and the boreholes performed (and installed instrumentation, if any) are listed in Table 5-1. The locations of the boreholes listed are displayed in Drawing 05, Appendix A.

A summary of the investigation programs and findings for the Bear Cave, Pretty Rocks and Polychrome Rest Stop and Overlook landslide areas is presented in the following subsections.

Table 5-1. Summary of historical borings at Bear Cave and Pretty Rocks Landslides in Denali National Park and Reserve.

Location	Year	Borings	Instrumentation
Bear Cave	1994	B-1	Vibrating Wire Piezometer (VWP), Slope Inclinometer (SI)
Bear Cave	1994	B-2	SI
Bear Cave	1994	B-3	
Bear Cave	1994	B-4	
Bear Cave	1994	B-5	
Bear Cave	1996	96-1	
Bear Cave	1996	96-2	Thermistor, SI
Bear Cave	1996	96-3	
Bear Cave	1996	96-4	
Bear Cave	1996	96-5	
Bear Cave	1996	96-6	
Bear Cave	1997	RM-1	Piezometer, Thermistor
Bear Cave	1997	RM-2	Thermistor
Bear Cave	1997	RM-3	Piezometer, Thermistor
Bear Cave	1997	RM-4	Thermistor, Inclinometer casing
Bear Cave	1997	RM-5	Piezometer, Thermistor
Bear Cave	1997	RM-6	Thermistor
Bear Cave	1997	RM-7	Piezometer, Thermistor
Bear Cave	1997	RM-8	Piezometer, Thermistor
Bear Cave	1997	RM-9	Thermistor
Bear Cave	1998	B-98-1	SI, VWP
Bear Cave	1998	B-98-2	SI, VWP
Polychrome Rest Stop	2003	PS03-1	Open standpipe piezometer

Location	Year	Borings	Instrumentation
Polychrome Rest Stop	2003	PS03-2	SI
Pretty Rocks	2003	PLY03-1	
Pretty Rocks	2003	PLY03-2	
Polychrome Overlook	2016	BH16-02	
Polychrome Overlook	2016	BH16-03	
Pretty Rocks	2018	PR18-01	SI, ShapeArray (SAAV), VWP, Thermistor
Pretty Rocks	2018	PR18-02	SI, SAAV, VWP, Thermistor
Pretty Rocks	2018	PR18-03	SI, SAAV, VWP, Thermistor
Pretty Rocks	2018	PR18-04	SI, VWP, Thermistor
Pretty Rocks	2018	PR18-05	SI, VWP, Thermistor
Pretty Rocks	2019	PR19-06	
Pretty Rocks	2019	PR19-07	SI, VWP, Thermistor
Pretty Rocks	2019	PR19-08	SI, VWP, Thermistor
Pretty Rocks	2019	PR19-09	
Pretty Rocks	2019	PR19-11	SI, 2 VWP, Thermistor

5.2.1. Bear Cave (MP 44.8)

5.2.1.1. Soils Investigation at 45 Mile Slump (Bear Cave Landslide) - 1994

In 1994, a soil investigation was conducted at the 45 Mile Slump (Bear Cave Landslide) (Shannon & Wilson Inc., 1995). The purpose of this investigation was to provide soils information to aid in determining subsurface conditions responsible for the landslide. At the time of investigation, it did not appear the landslide was impacting the road, with the shortest horizontal distance between the landslide and the road being approximately 40 ft.

As part of this investigation, five borings were drilled, and soil samples were collected. Two borings were completed with slope inclinometer casing. In general, the soils consisted of a mix of clay, sand, gravel and cobbles in various proportions, with clay pervading in most soils. Permafrost was encountered in all borings except B-1 at depths ranging from 17.5 ft to 60 ft. Groundwater was encountered at 54.1 ft in B-1 and was also found during drilling of B-4 and B-5. See Table 5-1 for a complete list of boreholes (B-1 to B-5) and the instrumentation installed.

5.2.1.2. Phase I Geotechnical Investigation – 1996

An investigation in 1996 consisted of a surface reconnaissance of the 45 Mile Slide (Bear Cave Landslide) and a drilling program in the slope area above the roadway. The landslide reconnaissance was performed to determine (1) the overall surface extent and geometry of the landslide, (2) the surface conditions of the landslide such as scarp locations and heights, seeps, springs and ponded water and (3) to determine possible structural relationships between the landslide material and the adjacent, undisturbed soils and bedrock and to determine a landslide mechanism (U.S. Department of Transportation, 1996). The purpose of the subsurface investigation was to determine the feasibility of moving the road uphill onto a new alignment that would either not be susceptible to future landslide movements or would be underlain by bedrock at a shallow enough depth that mitigation of the landslide would be possible with a retaining wall system. Six boreholes were drilled as part of this investigation. Figure 5-4 and Figure 5-5 show the Bear Cave Landslide adjacent to the Denali Park Road. See Table 5-1 for a complete list of boreholes (96-1 to 96-6) and the instrumentation installed.

Figure 5-4. Looking east at the MP 45 (Bear Cave) Landslide from the west side of the landslide area in May 1996.

Figure 5-5. Looking upslope toward the MP 45 landslide headscarp from the lower center of the landslide.

5.2.1.3. Test Borings – 1997

In 1997, nine boreholes were drilled as part of a geotechnical investigation at the Bear Cave Landslide (U.S. Department of Transportation, May 2003). Borings RM-1 and RM-9 are located within the roadway or at the road shoulder and an assortment of thermistors, piezometers and slope inclinometers have been installed in these boreholes. See Table 5-1 for a complete list of boreholes (RM-1 to RM-9) and instrumentation installed.

5.2.1.4. Denali Park Mile Post 45 Landslide Phase III - 1999

A report on the MP 45 Landslide (Bear Cave Landslide) was published in 1999. It appeared that the main landslide was a reactivated ancient landslide where frozen ground is not a significant factor. Stability analysis and movement rates indicated the main landslide was unlikely to capture the roadway with the next few decades. The secondary landslide feature was relatively shallow movement in the northwest corner of the main landslide caused by slumps and flow of saturated material. This landslide was noted to likely affect the road within the next few years. Two borings (B-98-1 and B-98-2) were completed and instrumented with inclinometer casings and vibrating wire piezometers (Foundation Engineering Inc., 1999). Figure 5-6 illustrates the Bear Cave Landslide as well as some borehole locations relative to the landslide. Figure 5-7 shows core samples taken from B-98-01. See Table 5-1 for a list of boreholes and the instrumentation installed.

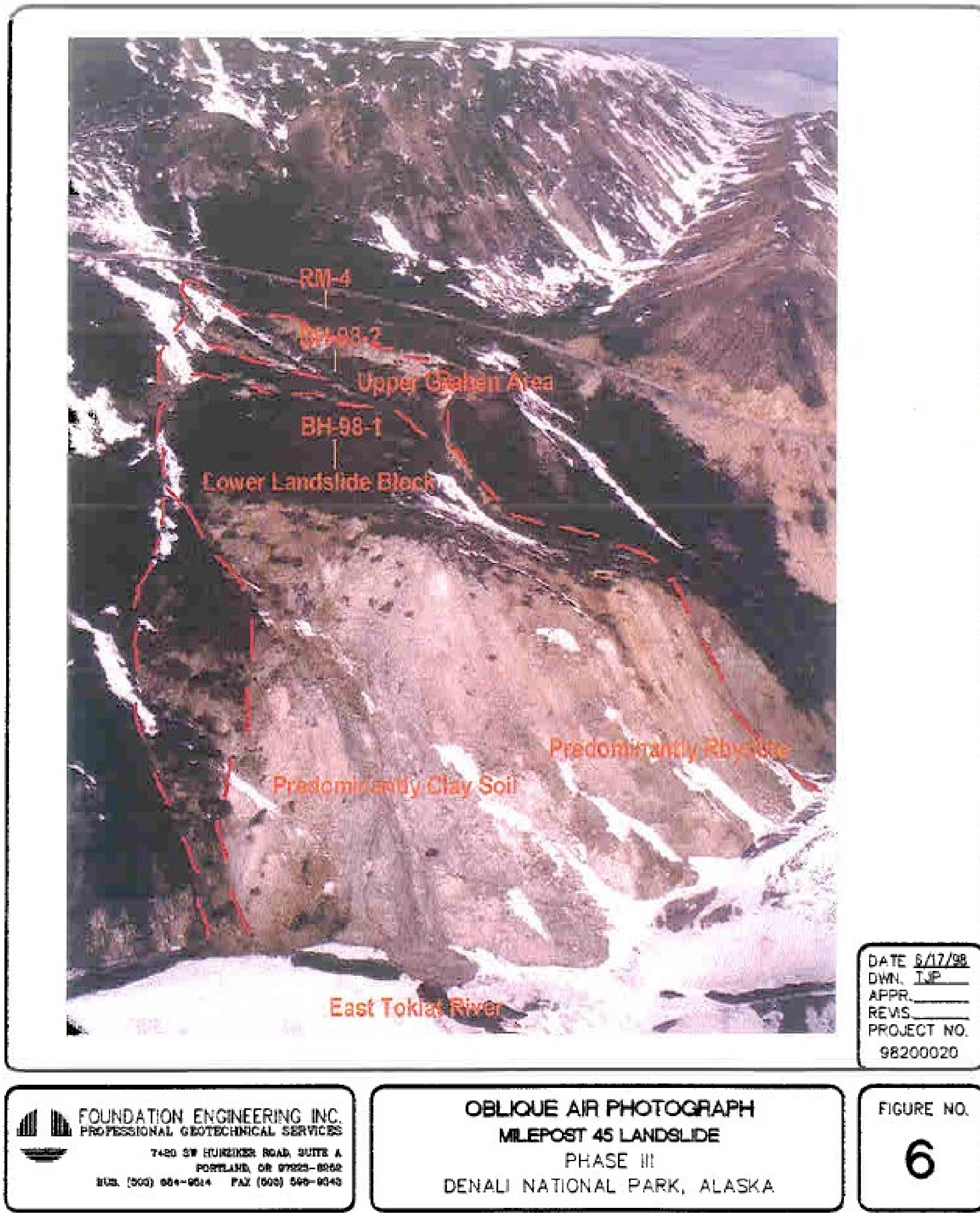
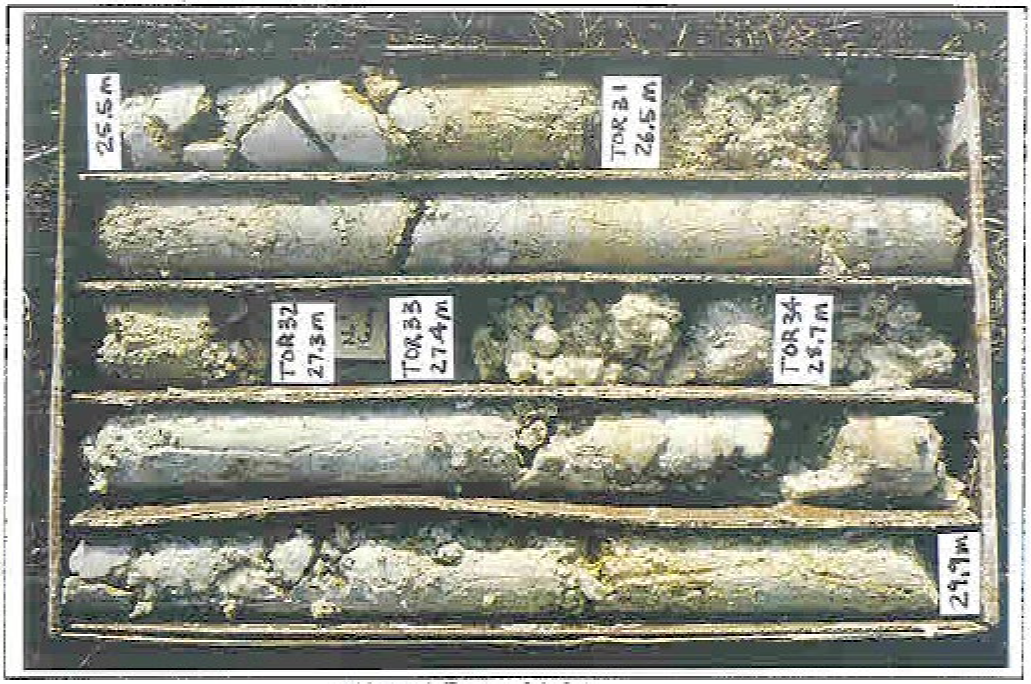
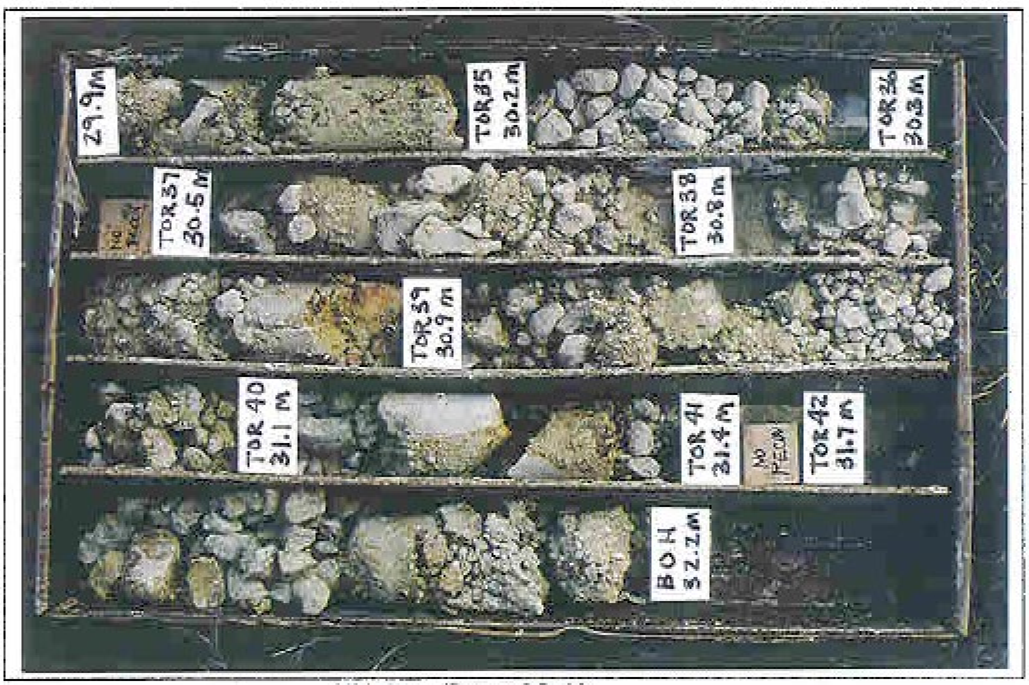


Figure 5-6. Oblique air photograph of the Milepost 45 Landslide.



BH 98-1 Runs 31-34



BH 98-1 Runs 35-42

Figure 5-7. Core samples from borehole B-98-1.

5.2.1.5. Geophysical Investigation – 2016 (MP 44-46)

In August 2016, geophysical investigations of four sections of Denali Park Road were conducted (U.S. Army Engineer Research and Development Center, 2017). The purpose of these investigations was to determine the presence and extent of subsurface features and anomalies impacting road infrastructure. Geophysical techniques such as capacitive-couple resistivity (CCR), ground-penetrating RADAR (GPR) and electrical resistivity tomography (ERT) were utilized to survey the subsurface. A survey was conducted at Polychrome Pass (MP 44-46). The Bear Cave Landslide is displayed in Figure 5-8.

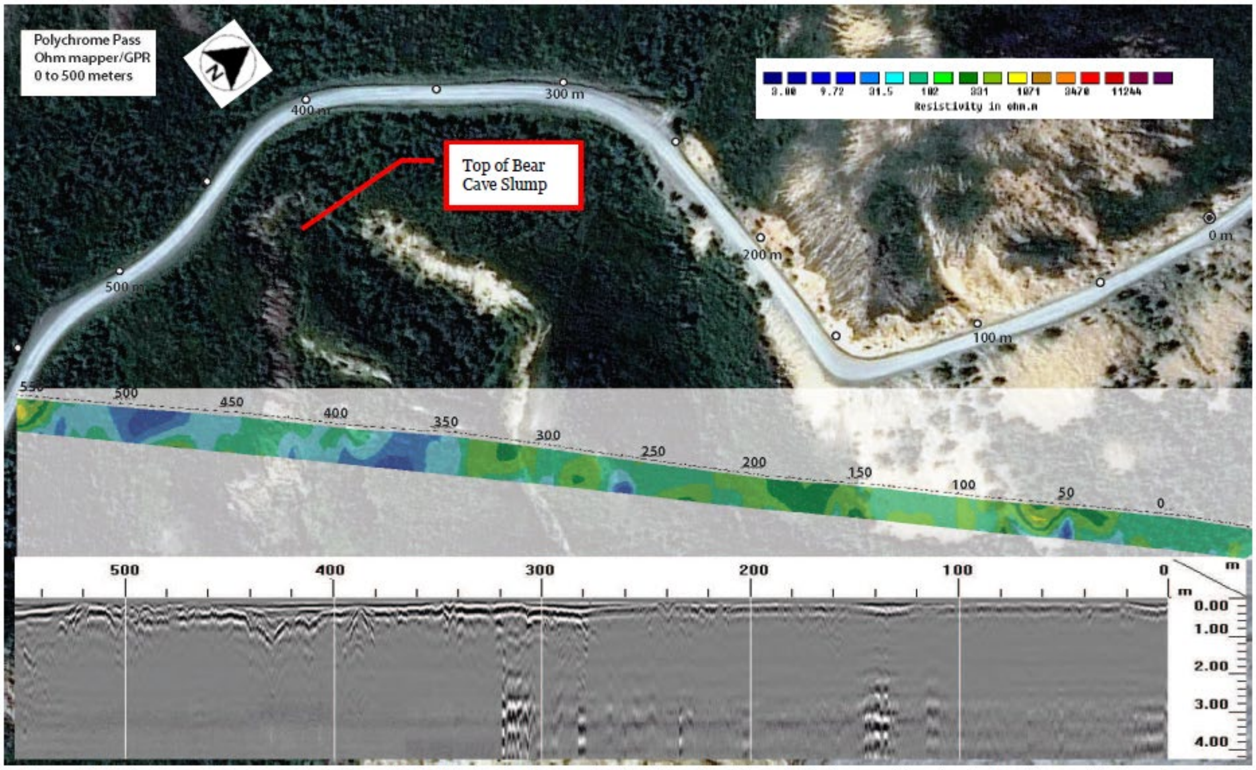


Figure 33. Polychrome Pass, 0m to 500m (1640-ft), start of transect. The view direction of this transect is oriented looking north, therefore the transect moves from right to left, east to west. The ohm-m values are very low through this section, with very low values across and above the headscarp of the Bear Cave Slump, from 250m to 500m possibly indicating higher moisture/ground water. The GPR is not remarkable at depth but a distinct reflector is visible at approximately 0.5m (1.6-ft) in depth, possibly representing the cut surface of the sub-grade. Chaotic structure is not visible in the GPR due to radar energy attenuation possibly from high moisture contents.

Figure 5-8. Resistivities at Bear Cave Landslide from the 2016 geophysical survey of Denali Park Road.

5.2.2. Pretty Rocks Landslide (MP 45.3)

5.2.2.1. Test Borings – 2003

In 2003, two boreholes were drilled as part of a geotechnical investigation at the Pretty Rocks Landslide (U.S. Army Engineer Research and Development Center, 2017). Ice and frozen material were found at a depth of 20 ft below the ground surface in PLY03-1. In PLY03-2, ice was logged at 40 ft below ground surface with silty, gravelly material at the top of this section. See Table 5-1 for a list of these boreholes.

5.2.2.2. Geophysical Investigation – 2016 (MP 44-46)

Geophysical investigations of four sections of Denali Park Road in August 2016 were conducted (U.S. Army Engineer Research and Development Center, 2017). The purpose of these investigations was to determine the presence and extent of subsurface features and anomalies impacting road infrastructure. Geophysical techniques such as capacitive-couple resistivity (CCR), ground-penetrating RADAR (GPR) and electrical resistivity tomography (ERT) were utilized to survey the subsurface. A survey was conducted at Polychrome Pass (MP 44-46). It appeared that the subsurface at Pretty Rocks Landslide contained significant ground ice and appeared to be an active rock wedge-controlled landslide feature. The Pretty Rocks Landslide is displayed in Figure 5-9.

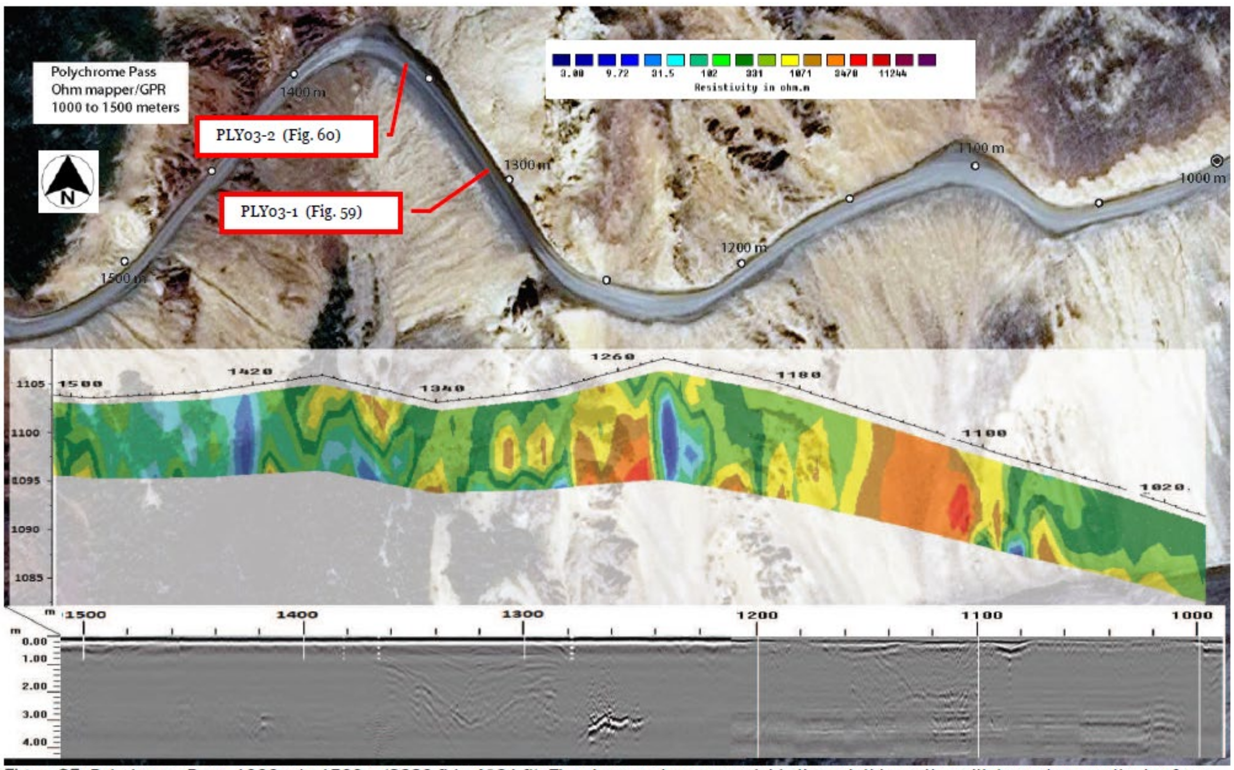


Figure 35. Polychrome Pass, 1000m to 1500m (3280-ft to 4921-ft). The ohm-m values are variable through this section with low values continuing from 950m and up to approximately 1100m. Here the roadway transcends an exposed ridge and much higher values begin and continue as the roadway cuts back into the strata and through a valley to the next ridge which is located at 1200m. This ridge has very low ohm-m values suggesting either very conductive mineralogy or moist/wet conditions at depth. At 1260m the road passes through a cut with high ohm-m values at depth, and then leads into the Pretty Rocks slide section. Through the slide section moderate near surface ohm-m values are measured with very low values at depth suggesting moist/wet conditions. The survey transcends the crotch at approx. 1390m, which is the contact between the mafic strata to the west and the felsic strata (rhyolite) to the east. After this valley, low ohm-m values are measured through to the end of the section. The GPR is very unremarkable through this section except for distinct reflections starting at 1260m where the road passes through the cut.

Figure 5-9. Resistivities at Pretty Rocks Landslide from the 2016 geophysical survey of Denali Park Road.

5.2.2.3. Test Borings – 2018

In 2018, five boreholes within the Pretty Rocks Landslide were drilled with depths between 108 ft and 140.3 ft from the ground surface (U.S. Department of Transportation, 2018). Slope inclinometer casing, VWPs and thermistors were installed in all boreholes. The groundwater level in PR18-02 was about 60 ft below ground surface; however, all other VWPs indicated that groundwater levels were below the depth of the instruments. SAAVs were installed in PR18-01, PR18-02 and PR18-03. See Table 5-1 for a complete list of boreholes (PR18-01 to PR19-05) and instrumentation installed.

5.2.2.4. Test Borings – 2019

In 2019 five boreholes within the Pretty Rocks Landslide were drilled with depths between 100.3 ft and 157.1 ft from the ground surface (U.S Department of Transportation, 2019). Following selection of two conceptual alternatives by the NPS in June 2019, these boreholes were installed to determine feasibility and constructability of a bridging and earthwork option. The four 2019 borings on the roadway were specific to the bridging option feasibility and PR19-11 was needed to define the stratigraphic model of the Pretty Rocks Landslide in the lower part of the landslide for the earthwork options feasibility. Downhole geophysical surveys were performed following drilling in boreholes PR19-06, PR19-07, PR19-08, and PR19-09. VWPs in PR19-07 and PR19-08 indicated that groundwater levels were below the depth of the instruments (90 and 93 ft, respectively). Two VWPs were installed in PR19-11, one at 55 ft and the other 98 ft. Groundwater depths were steady over time for each instrument at 39 ft and 69 ft deep, respectively. See Table 5-1 for a complete list of boreholes (PR19-06 to PR19-09 and PR19-11) and instrumentation installed.

5.2.2.5. Pretty Rocks Photos

Figure 5-10 through Figure 5-15 give an overview of recent conditions at the Pretty Rocks Landslide along the Denali Park Road.

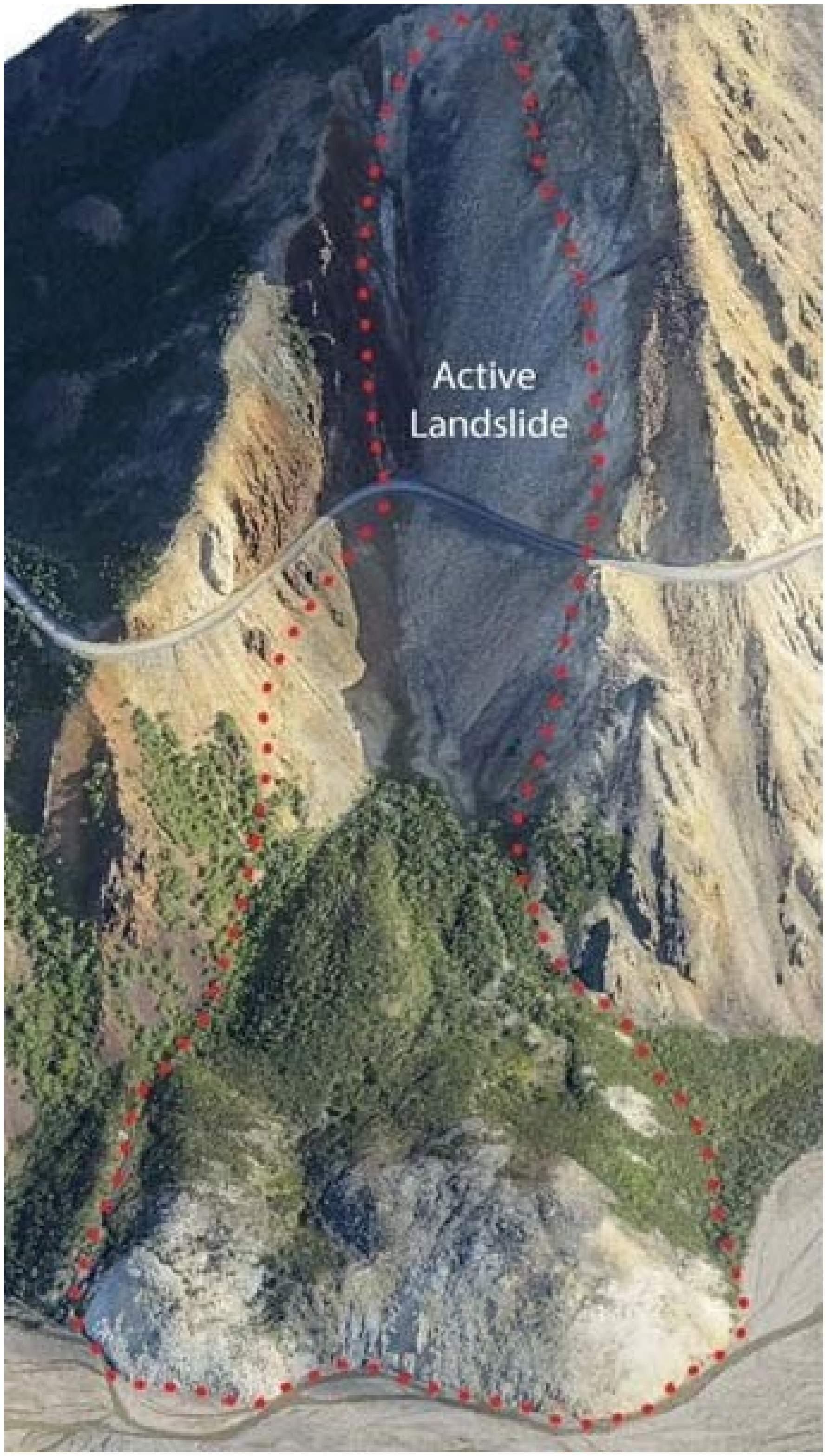


Figure 5-10. Oblique view of the Pretty Rocks Landslide. Image is from June 15, 2015 and the red dots outline the approximate landslide extents (NPS, 2020).

Figure 5-11. Aerial view of the Pretty Rocks Landslide area on the Denali Park Road from November 5, 2019 (Williams, 2019).

Figure 5-12. Looking at the western portion of the toe of the Pretty Rocks Landslide from across the Toklat River. FHWA photo (2019).

Figure 5-13. Pretty Rocks Landslide scarp at the Denali Park Road in November 2019 (NPS, 2020). The road had been displaced approximately 10 ft since September 2019 (from red arrow to yellow arrow).

Figure 5-14. The same location as Figure 5-13 at the Pretty Rocks Landslide in January 2019 (NPS, 2020). The road had been displaced approximately 15 ft since September 2019.

Figure 5-15. At the eastern Pretty Rock Landslide scarp. The area has fallen approximately 6.5 ft from September 2018 to when this photo was taken in March 2019 (NPS, 2020).

5.2.3. Polychrome Pass Rest Stop/Overlook (MP 45.8 – 46.2)

5.2.3.1. Emergency Repair Recommendations for Polychrome Rest Stop/Overlook – 2003 (MP 45.8)

A slump at the Polychrome rest stop began to develop in late summer 2002. Winter and spring 2002-2003 were abnormally dry and no movement was observed in the slump until summer monsoonal rains began in July 2003. Over a one-week period in August 2003 the slump dropped vertically 4 ft. Two subsurface boreholes were drilled with an open-standpipe piezometer installed in PS03-1 and an inclinometer installed in PS03-2 (U.S. Department of Transportation, September 2, 2003). No groundwater measurements were able to be recorded due to an unexplained block in the hole when attempting to take measurements. It was apparent that the slump reacted quickly to precipitation. Slumping at the rest stop and roadway are shown in Figure 5-16 and Figure 5-17, respectively. See Table 5-1 for a list of boreholes (PS03-01 and PS03-02) and the instrumentation installed. Reactivation of this slump was first observed in May 2019 during spring road opening (Figure 5-18 and Figure 5-19). Subsequent orthographic imagery review suggests the landslide movement reactivated between June 6 and September

27, 2018. A vertical drop of 2 to 8 inches occurred along the shoulder along about 75 feet of road length and the scarp was promptly coned by Maintenance.

Photo 3: View to north toward rest stop facilities, August 18, 2003.
NPS/Martin Grosnick photo

Figure 5-16. View to the north toward rest stop facilities, August 18, 2003. NPS/Martin Grosnick photo.

Figure 5-17. Looking east at slump on August 18, 2003. NPS/Martin Grosnick photo.

Figure 5-18. Looking west at slump during road opening in May 2019. FHWA photo, 2019.

Figure 5-19. Looking east at slump during road opening in May 2019. FHWA photo, 2019.

5.2.3.2. West of Polychrome Overlook Test Borings – 2016 (MP 46.1 - 46.2)

A subsurface investigation was conducted in 2016 that involved performing 15 boreholes at locations of interest along the entire Denali Park Road (U.S. Department of Transportation, 2016). Of the 15 boreholes performed, two were within the Polychrome Pass area, BH16-02 and BH16-03. Groundwater was encountered after drilling at 21.8 ft and 16.7 ft, respectively. No instrumentation is reported to have been installed at these locations. See Table 5-1 for a list of boreholes.

5.2.3.3. Geophysical Investigation – 2016 (MP 44 - 46)

As discussed previously, in 2016, geophysical investigations of four sections of Denali Park Road were conducted (U.S. Army Engineer Research and Development Center, 2017). The purpose of these investigations was to determine the presence of permafrost and the extent of subsurface features and anomalies impacting road infrastructure. A survey was conducted at Polychrome Pass (MP 44-46) and the study deliverable for the Polychrome Overlook is displayed in Figure 5-20.

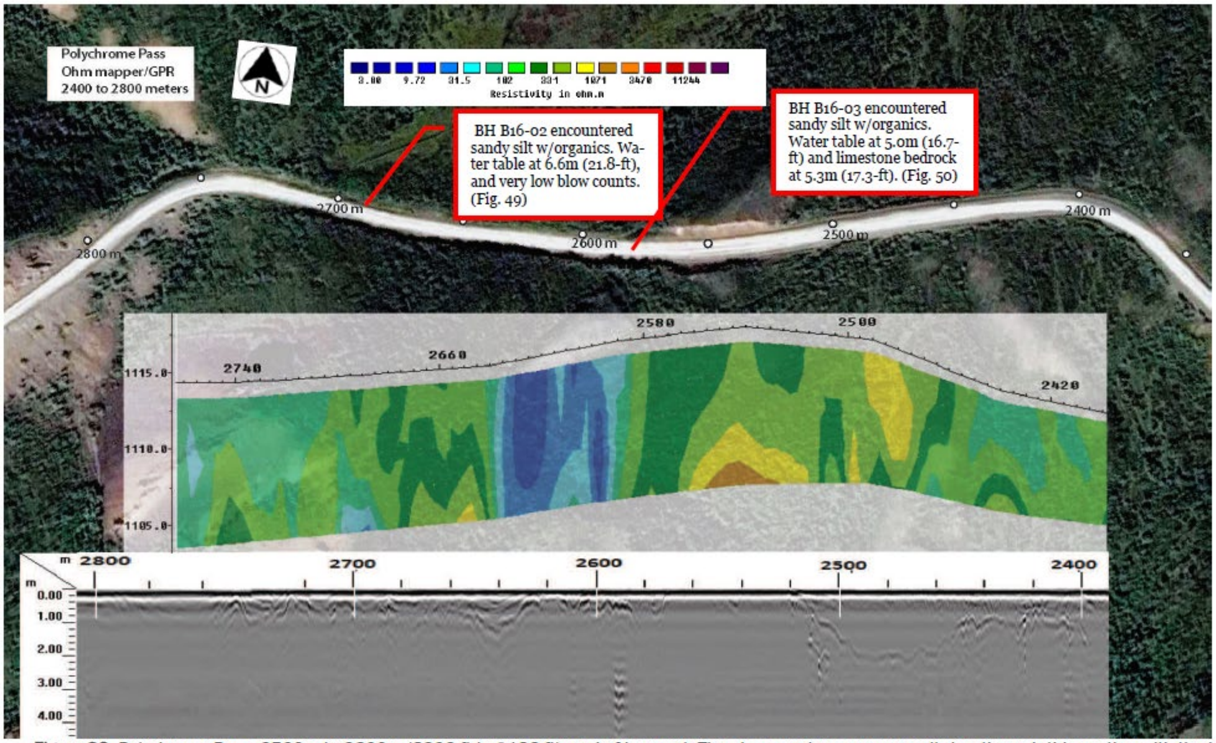


Figure 38. Polychrome Pass, 2500m to 2800m (8202-ft to 9186-ft), end of transect. The ohm-m values are generally low through this section with the low values continuing from the previous section at 2400m to 2480m. Near surface and deeper values increase while passing through a small road cut at 2540m, indicating bedrock which was encountered in BH B16-03. Very low values are encountered at 2600m to 2640m and this coincides with surface water immediately adjacent to the road on the north side. The values increase only slightly through to the end of the section. The GPR is unremarkable except for distinct near surface reflections visible showing a down drop of the subgrade surface from 2440m to 2500m, and a smaller dip from 2625m to 2660m.

Figure 5-20. Resistivities at Polychrome Overlook from the 2017 geophysical survey of Denali Park Road.

5.3. Unstable Slope Management Program (USMP) Sites

The Denali Park Road is impacted by numerous slopes affected by geotechnical hazards have been identified through the Unstable Slope Management Program for Federal Land Management Agencies (USMP) along the Denali Park Road (Figure 5-21). These slope hazards and their associated risks include rockfalls and landslides and are assigned a relatively good, fair or poor condition rating as it relates to impact on infrastructure and cultural and environmental impacts with criteria outlined in Figure 5-22. Nine sites with USMP ratings above 400 points (relatively poor condition) are defined in more detail in the sections below and in Table 5-2.

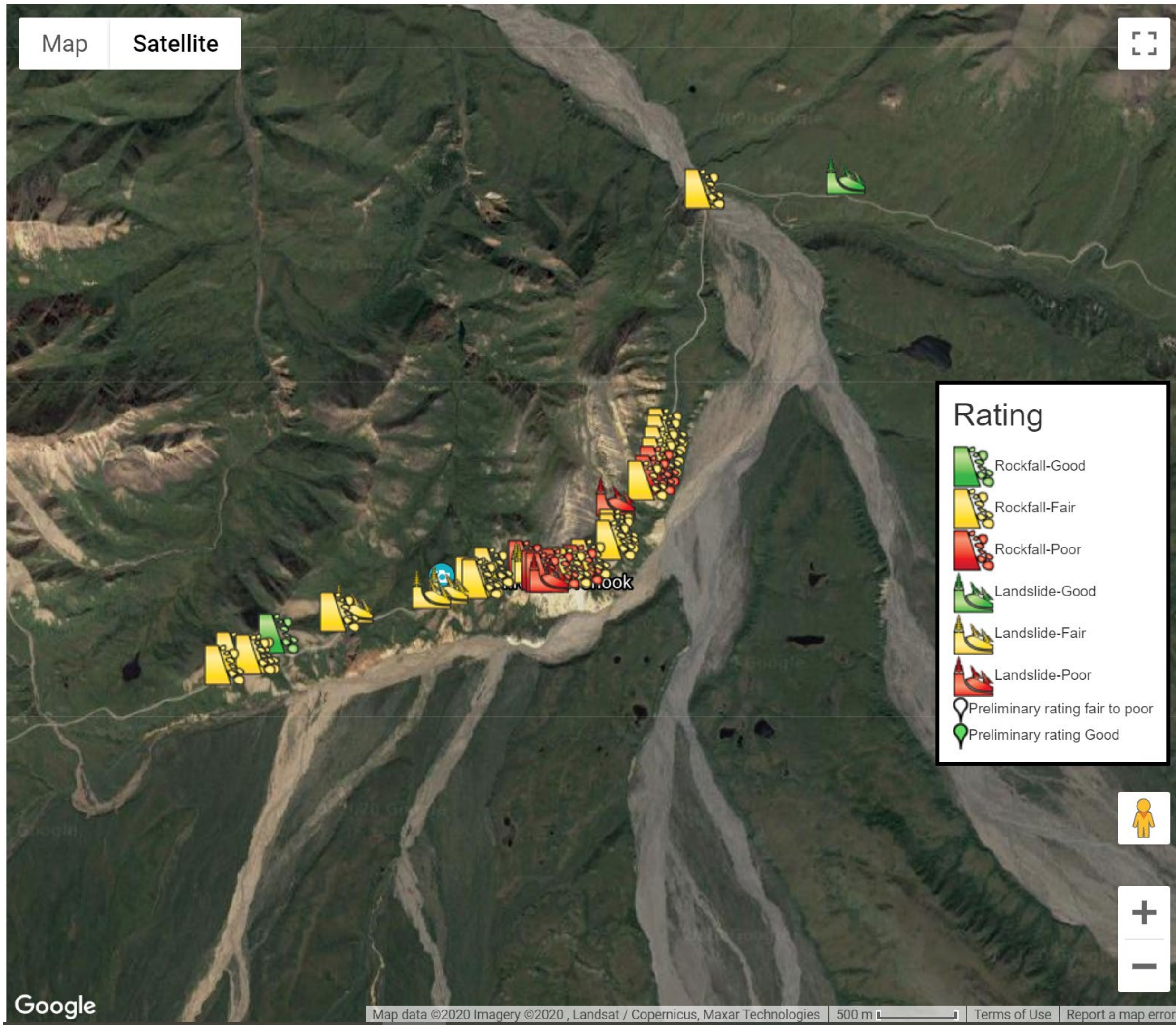


Figure 5-21. Overview of USMP sites on the Denali Park Road at Polychrome Pass in Alaska (U.S Department of Transportation, March 11, 2020).

SLOPE RATING FORM – DETAILED SLOPE HAZARD RATING									
				Category Rating	**3**	**9**	**27**	**81**	**Score**
				I. All – Slope Drainage	Slope appears dry or well drained; surface runoff well controlled	Intermittent water on slope; mod. well drained; or surface runoff moderately controlled	Water usually on slope; poorly drained; or surface runoff poorly controlled	Water always on slope; very poorly drained; or surface water runoff control not present	
				J. All – Annual Rainfall	0-10"	10-30"	30-60"	60"+	
				K. All – Slope Height (rockfall) / Axial length of slide (landslide)	25 ft	50 ft	75 ft	100 ft	CALC
Select One Unstable Slope Type	**Landslides/ Erosion (add A, B, C)**			**L. Thaw Stability (cold climates)**	Unfrozen/Thaw Stable	Slightly Thaw Unstable	Moderately Thaw Unstable	Highly Thaw Unstable	
				M. Instability-Related Maint. Frequency	Every 10 years	Every 5 years	Every 2 years	Every year	
				N. Movement History	Minor movement or sporadic creep	Up to 1 inch annually or steady annual creep	Up to 3 inches per event, one event per year	>3" per event, >6" annually, more than 1 event per year (includes all debris flows)	
	Rockfalls (add D, E, F)			**O. Rockfall-Related Maint. Frequency**	Normal, scheduled maintenance	Patrols after every storm event	Routine seasonal patrols	Year-round patrols	
		Geologic Character	**Case 1**	**P. Structural Condition**	Favorable	Random	Adverse Discontinuous	Adverse Continuous	
				Q. Rock Friction	Rough/ Irregular	Undulating	Planar	Clay infilled/ Slickensided	
			Case 2	**R. Structural Condition**	Few differential erosion features	Occasional differential erosion features	Many differential erosion features	Major differential erosion features	
				S. Diff. in Erosion Rates	Small difference	Moderate difference	Large difference	Extreme difference	
								T. LANDSLIDE HAZARD TOTAL (A+B+C+I+J+K+L+M+N)	CALC
								U. ROCKFALL HAZARD TOTAL (D+E+F+I+J+K+O+(greatest of P+Q or R+S))	CALC
DETAILED RISK RATING									
				V. Route Width or Trail Width	36 ft 14 ft	28 ft 10 ft	20 ft 6 ft	12 ft 2 ft	CALC
				W. Human Exposure Factor	12.5% of the time	25% of the time	37.5% of the time	50% of the time	CALC if AADT avail
				X. % of Decision Sight Distance (Judge avoidance ability on trails)	Adequate, 100% of low design value	Moderate, 80% of low design value	Limited, 60% of low design value	Very Limited, 40% of low design value	CALC for roads
				Y. Right of Way (R/W) Impacts (If Left Unattended)	No R/W implications	Minor effects beyond R/W	Private property, no structures affected	Structures, roads, RR, utilities, or Parks affected	
				Z. Environmental/Cultural Impacts if Left Unattended	None/No potential to cause effects	Likely to effect/No hist. prop. affected	Likely to adversely affect/Finding of no adverse effect	Current adverse effects/Adverse effect	
				AA. Maintenance Complexity	Routine effort/In-House	In-House Maint./ Special project	Specialized equip./contract	Complex/Dangerous effort/location/ contract	
				BB. Event Cost	$0-2k	$2-25k	$25-100k	>$100k	
								CC. RISK TOTALS: (G+H+V+W+X+Y+Z+AA+BB)	CALC
								TOTAL USMP SCORE: LANDSLIDES (T+CC) OR ROCKFALL (U+CC)	CALC
Total USMP Score Good (< 200 pts) | Fair (200 - 400 pts) | Poor (> 400 pts)									

Figure 5-22. Preliminary rating criteria for unstable slopes along the Denali Park Road (U.S. Department of Transportation, June 23, 2017).

Table 5-2. Summary of USMP sites greater than 400 points along Denali Park Road.

Site ID	USMP ID	MP (begin)	MP (end)	Type	Rating
863	27	44.57	44.59	Rockfall	478
864	3	44.59	44.64	Rockfall	537
955	29	44.81	44.83	Landslide	440
870	34	45.17	45.21	Rockfall	526
933	17	45.27	45.32	Rockfall	435
873	16	45.32	45.34	Rockfall	460
956	95	45.32	45.34	Landslide	450
957	12	45.34	45.41	Landslide	948
3177	N/A	45.41	45.48	Rockfall	416

5.3.1. Site 863 and 864 (DENA USMP 027 and 003)

Sites 863 and 864 are both unstable slopes that generate rockfall. Site 863 has a maximum slope height of approximately 95 feet and is approximately 180 feet long. It is made up of weak to moderately strong rhyolite (igneous) rock that is causing structurally controlled planar and wedge failures and differential erosion failures consisting of boulders with a maximum block size of two feet. Site 864 has a maximum slope height of approximately 60 feet and is approximately 250 feet long. This slope is also comprised of rhyolite rock, producing structurally controlled rock failures with a maximum block size of approximately 3 feet. The approximate locations and proximities of Site 863 and Site 864 to the Denali Park Road are shown in Figure 5-23 (U.S Department of Transportation, December 2019).

Figure 5-23. Overview of Sites 863 and 864 between MP 44.57 and MP 44.64 on the Denali Park Road in Alaska (U.S Department of Transportation, December 2019).

5.3.2. Site 955 (DENA USMP 029)

Site 955 is also known as the “Bear Cave Slump” and is a rotational landslide with an axial length of approximately 1,000 feet. It affects approximately 300 feet of the Denali Park Road. The landslide headscarp is located below the road and headscarp erosion and regression continue to impact the road. In the 1990s a deep cutoff trench lined with geotextile located in the uphill road ditch was installed to direct subsurface and surface drainage away from the landslide to a nearby culvert to mitigate landslide movement. Since construction of this ditch the regression of the landslide towards the road has slowed. However, after the historic precipitation events in August 2019, measurement of the landslide headscarp retrogression toward the roadway was observed. Regression of the landslide headscarp towards the road is monitored with periodic GPS surveys. At its shortest distance, the headscarp of the landslide is within 10 feet of the road embankment now. The location and proximity of Site 955 to the Denali Park Road is shown in Figure 5-24 (U.S Department of Transportation, December 2019).

Figure 5-24. Overview of Site 955 between MP 44.81 and MP 44.83 on the Denali Park Road in Alaska (U.S Department of Transportation, December 2019).

5.3.3. Site 870 and 933 (DENA USMP 034 and 017)

Site 870 is an unstable road cut slope with a maximum height of 55 feet and a length of 230 feet. This slope is made up of rhyolite rock that is causing wedge and planar structurally controlled rock failures with some areas of raveling and undermining occurring. The average block size is approximately 3 feet, but 5-foot blocks and debris landslide failure events up to 9 cubic yards in volume have been observed. The existing road cut is over-steepened with an angle of 40 to 41 degrees. The upper natural slope appears to be stable at an angle of 30 to 32.5 degrees.

Site 933 is an unstable slope with a maximum height of 187 feet and a length of 430 feet. This slope is comprised of loose, rhyolite rock that causes planar and wedge, structurally controlled rock failures. The maximum block size is approximately 1 foot. Rockfall ditch catchment is limited and sight distance is very limited along this section of road. The locations and proximities of Site 870 and Site 933 to the Denali Park Road are shown in Figure 5-25 (U.S Department of Transportation, December 2019).

Figure 5-25. Overview of sites 870 and 933 between MP 45.17 and MP 45.32 on the Denali Park Road in Alaska (U.S Department of Transportation, December 2019).

5.3.4. Site 873 and 956 (DENA USMP 016 and 095)

Site 873 is known as "Perlite Rockfall" and has a maximum height of 140 feet and affects 125 feet of Denali Park Road. It consists of degrading rhyolite with intermittent perlite beds that is causing structurally controlled planar, wedge and indeterminate failures. The maximum block size is approximately one foot. Site 956 is known as "Perlite Debris Slide" with an axial length of 80 feet and a slope angle of approximately 39 degrees that affects 45 feet of Denali Park Road. The failure mechanism is rotational debris slide events in rhyolite, perlite and colluvium materials. A natural spring from a geologic contact between the impermeable perlite layer and the rhyolitic colluvium is a piping trigger for these events. The locations and proximities of Site 873 and Site 956 to the Denali Park Road are shown in Figure 5-26 (U.S Department of Transportation, December 2019).

Figure 5-26. Overview of Sites 873 and 956 between MP 45.32 and MP 45.34 on the Denali Park Road in Alaska (U.S Department of Transportation, December 2019).

5.3.5. Site 957 (DENA USMP 012)

Site 957, known as "Pretty Rocks Landslide", is a large-scale slump feature with an axial length of approximately 490 feet. The landslide affects approximately 294 feet of Denali Park Road. This slope consists of loose rhyolitic rock underlain by ice and frozen material. The failure mechanism appears to be dominantly translational debris slide events in these materials with minor rotational movement. Sight distance is limited along this section of road. In 1987, drainage control was installed below the road surface however, it has since been buried and is now ineffective (U.S Department of Transportation, December 2019). In 2018, on average, the road movement was measured at approximately 0.4 inches per day and it was difficult to maintain through the summer season by Park maintenance crews. Since August 2019, the rate of road subsidence, as a result of the continued landslide movement of the Pretty Rocks Landslide, has increased to nearly 2 inches (vertically and horizontally) per day. The location and proximity of Site 957 to the Denali Park Road is shown in Figure 5-27.

Figure 5-27. Overview of Site 957 between MP 45.34 and MP 45.41 on the Denali Park Road in Alaska (U.S Department of Transportation, 2020).

5.3.6. Site 3177

Site 3177 is an unstable slope with a maximum height of 120 feet and is 400 feet long. This slope consists of degrading basalt and rhyolite which are causing structurally controlled planar, wedge and indeterminate failures. These failures are either a block with a maximum size of 3 feet or debris slide events with a maximum volume of 6 cubic yards. The existing slope is oriented at 70 degrees from horizontal. The location and proximity of Site 3177 to the Denali Park Road is shown in Figure 5-28 (U.S Department of Transportation, December 2019).

Figure 5-28. Overview of Sites 3177 between MP 45.41 and MP 45.48 on the Denali Park Road in Alaska (U.S Department of Transportation, December 2019).

6.0 POTENTIAL FUTURE INVESTIGATIONS

This summary of known existing conditions reveals that there is considerably more known about the ground conditions and movement history on the existing alignment than any of the proposed alternatives. This is not surprising, given that it has been in service for many years and has had recent study of the Pretty Rocks and other landslides. Nevertheless, through use of field surveys, published maps and the InSAR results presented here, there is enough known about the other alignment options to base judgments of ground movement expectations during construction and for long term performance. A general familiarity with road building and maintenance in the park and elsewhere in this environment, and the impacts of climate, also inform performance expectations.

Because these expectations are significantly judgment based, it will be appropriate to adjust them based on an investigation program as part of the alternatives analysis and preliminary design process. A conceptual preliminary geotechnical investigation and instrumentation plan is being developed for this purpose under separate cover. The plan is summarized here and presented in plan maps and summary tables in Appendix C.

Knowledge of the existing conditions on the proposed north and south alignments is based on mapping, a traverse performed on foot in 2019, and InSAR collected in 2020. These data sources and mapping efforts have informed the proposed preliminary investigation plan for the potential new alignments. The conceptual investigation plan along the alternative alignments will include subsurface explorations at the abutments of proposed bridges, three identified landslides, 5.2 miles of earthwork on the north alignment and 4.5 miles on the south alignment.

Preliminary drilling for the structures will identify the conditions for foundation design, including material type and frost depth. Given the need to establish site variability and subsurface conditions for type, size, and location (TSL) plans, it is proposed to drill each abutment of each bridge to a depth that would be required for final design. Until bridge TSL plans are complete, no intermediate pier foundations are recommended for drilling under this preliminary phase of investigation.

Preliminary drilling for the landslides will characterize the subsurface materials, presence of groundwater and/or ice, depth of potential landslide movement, and current level of activity. The field exploration program will help develop an understanding of how climate or proposed construction could affect landslide activity. The drilling will also provide insight into whether the landslides could be mitigated, would need to be avoided, or will likely be an ongoing maintenance or safety issue throughout the life of the alignment.

Preliminary drilling for the earthwork will provide a better understanding of the spatial variation of permafrost, the depth of seasonal ice, and distribution of subsurface materials and presence of seasonal groundwater conditions. Note that there are means and methods for this work that would cause a relatively high degree of disturbance, such as pioneering roads to provide access to locations for rubber tire or track rigs. To limit disturbance, these methods are not recommended given the long-lasting impacts, and helicopter access is specified for the boring location plan in Drawing 01, and summarized in Tables 1, 2, and 3 of Appendix C.

An alternative approach for accessing sensitive drilling locations would be to use lightweight equipment that can be carried by a team of people, such as the Talon drill by Kryotek. This type of lightweight equipment will not likely be as successful at drilling to depths greater than about 20 feet, may more often hit refusal on cobbles and boulders, and would not provide SPT results, but it would allow for more holes to be drilled, and better characterization of depth of seasonal ice, presence of permafrost, and frost susceptibility variability along the alignments. An alternate plan of test hole locations using this lightweight equipment is proposed in Drawing 02 of Appendix C.

Electrical resistivity tomography (ERT) geophysical surveys will be coupled with boreholes and downhole instrumentation to provide additional insight into the spatial variability of ground ice conditions at bridge, landslide, and earthwork locations along each alignment. Other geophysical methods may be used in conjunction with ERT.

Although the existing alignment has had more study, there are some areas where additional investigation is desired to understand current ground movement or the potential for future ground movement. These six holes will be located between MP 43 and MP 48. Five holes will be drilled from the existing road and one will be drilled below the road and will require helicopter access.

Prior investigations at the Polychrome Pass Rest Stop/Overlook have focused on sliding impacting the road. However, the lidar, orthophotos, and InSAR presented herein suggest there may be two existing landslides lower on the slope. These landslides have a toe at the river elevation or below and while they have not impacted the road yet, if they are active landslides or were to reactivate, they could impact the road in the future.

Possible aggregate source locations have also been identified in channels and low terraces for preliminary sampling and testing because new aggregate sources will be needed if the north or south alignments are selected, and possible even for work on the existing alignment. These locations are shown on Drawings 01 and 02 of Appendix C. Test pits will be approximately 10 feet deep and will include mapping and bulk samples for grain size analysis and testing for aggregate suitability.

7.0 CLOSURE

We appreciate the opportunity to assist you on this project and trust the above satisfies your requirements at this time. Should you have any questions or comments, please do not hesitate to contact us.

Yours sincerely,

BGC ENGINEERING INC.
per:

Scott A. Anderson, Ph.D.
Principal Geotechnical Engineer

Cole Christiansen, M.Sc.
Geological Engineer

Reviewed by:

Mark Vessely, M.Sc.
Principal Geotechnical Engineer

SA/MV/mp/syt

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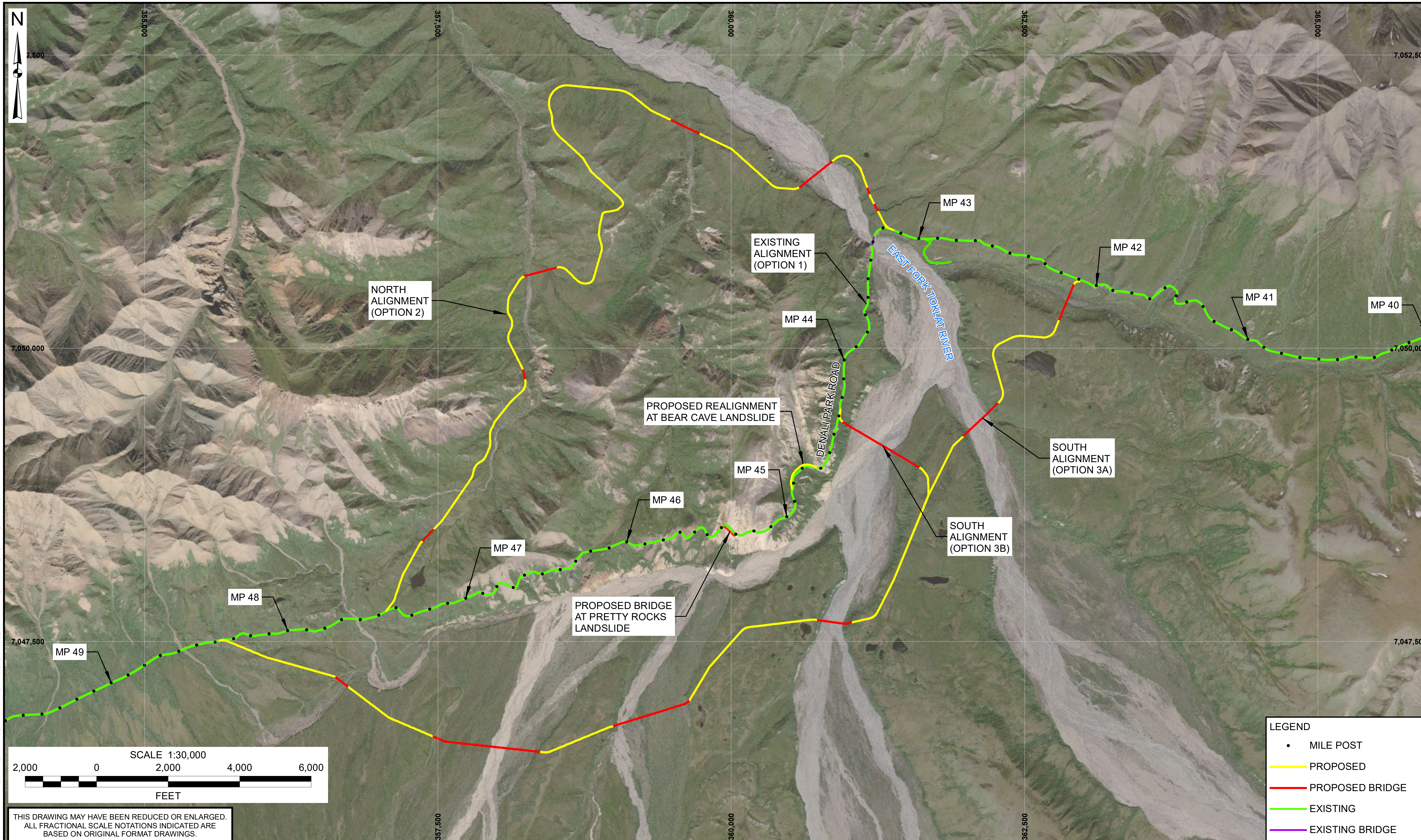
APPENDIX A
SUMMARY GEOTECHNICAL REPORT OF EXISTING CONDITIONS – DRAWINGS



NOTES:
1. ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE NOTED.
2. THIS DRAWING MUST BE READ IN CONJUNCTION WITH BGC'S REPORT TITLED "SUMMARY GEOTECHNICAL REPORT OF EXISTING CONDITIONS", AND DATED AUGUST 2020.
3. EXISTING ALIGNMENT COURTESY OF THE U.S. GEOLOGICAL SURVEY. ALTERNATIVE ALIGNMENTS PROVIDED BY FHWA - WESTERN FEDERAL LANDS HIGHWAY DIVISION (2019).
4. IMAGERY PROVIDED BY ESRI WORLD IMAGERY, DATED JULY 30, 2014.
5. HORIZONTAL PROJECTION IS NAD83 UTM ZONE 6N. VERTICAL DATUM IS NAVD88.
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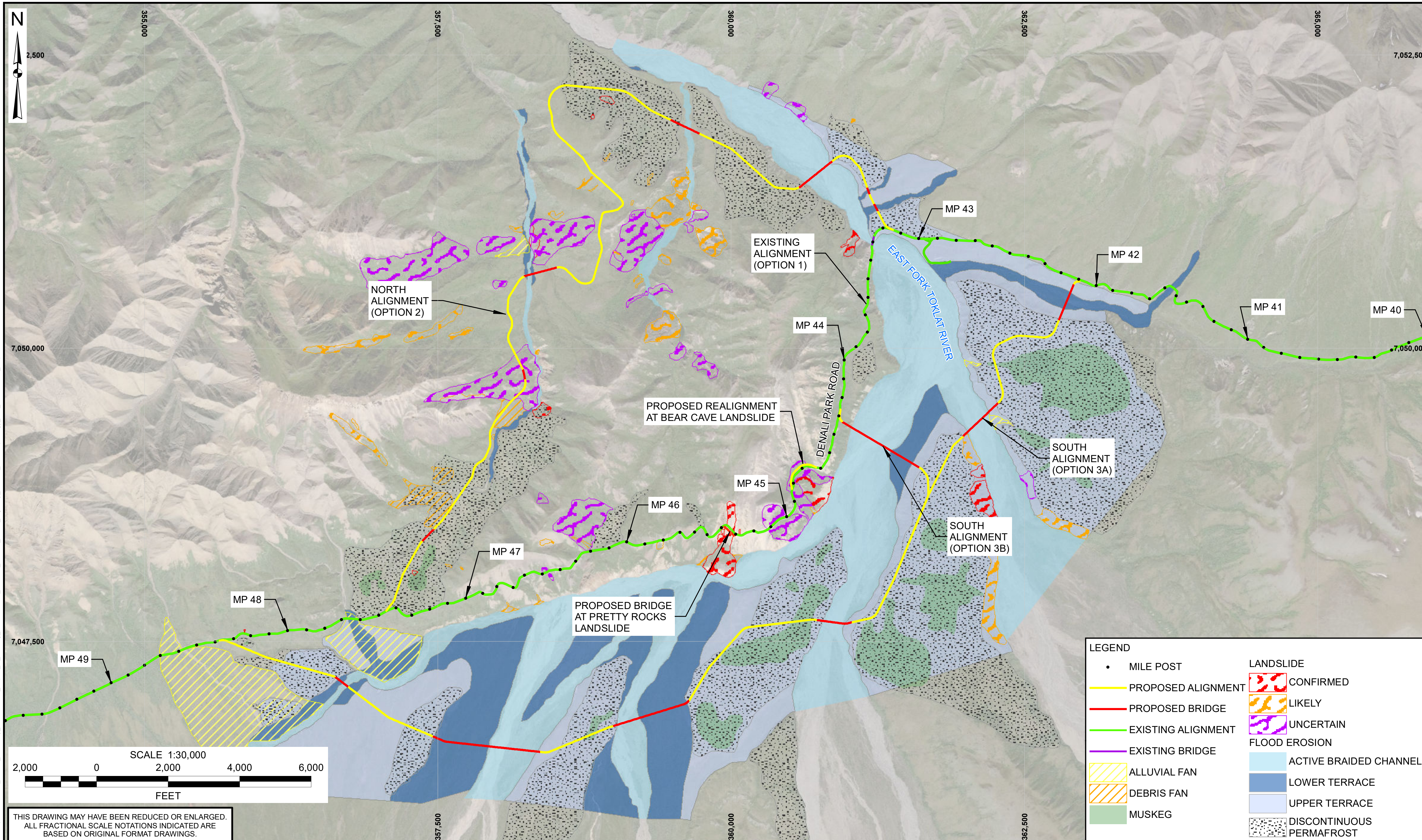
CLIENT:
JACOBS ENGINEERING / FHWA - WESTERN FEDERAL LANDS HIGHWAY DIVISION

PROJECT:
SUMMARY GEOTECHNICAL REPORT OF EXISTING CONDITIONS

TITLE:
ALTERNATIVE ALIGNMENT OVERVIEW MAP

PROJECT No.: 2000003
DWG No: 01



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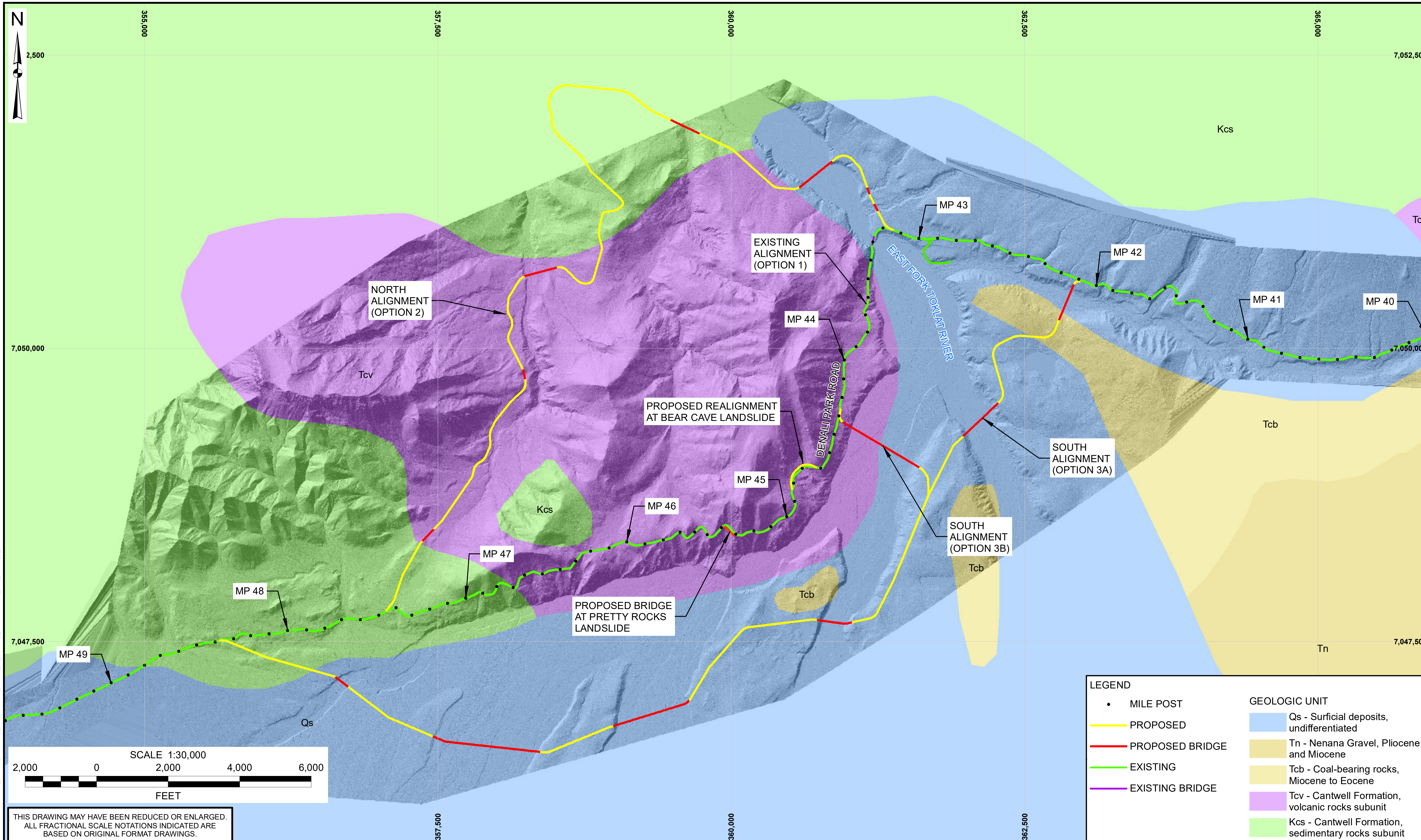
PROJECT:
SUMMARY GEOTECHNICAL REPORT OF EXISTING CONDITIONS

TITLE:
WESTERN FEDERAL LANDS HIGHWAY DIVISION GEOMORPHOLOGY AND GEOHAZARDS MAP

PROJECT No.:	DWG No:
2000003	02



NOTES:
1. ALL DIMENSIONS ARE IN FEET UNLESS OTHERWISE NOTED.
2. THIS DRAWING MUST BE READ IN CONJUNCTION WITH BGC'S REPORT TITLED "SUMMARY GEOTECHNICAL REPORT OF EXISTING CONDITIONS", AND DATED AUGUST 2020.
3. LIDAR PROVIDED BY KODIAK MAPPING INC, DATED AUGUST 22, 2018. RESOLUTION IS 0.25 ft.
4. EXISTING ALIGNMENT AND GEOLOGY UNITS COURTESY OF THE U.S. GEOLOGICAL SURVEY. ALTERNATIVE ALIGNMENTS PROVIDED BY FHWA - WESTERN FEDERAL LANDS HIGHWAY DIVISION (2019).
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BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

CLIENT:
JACOBS ENGINEERING / FHWA - WESTERN FEDERAL LANDS HIGHWAY DIVISION

PROJECT:
SUMMARY GEOTECHNICAL REPORT OF EXISTING CONDITIONS

TITLE:
SITE GEOLOGY MAP

PROJECT No.: 2000003
DWG No: 03

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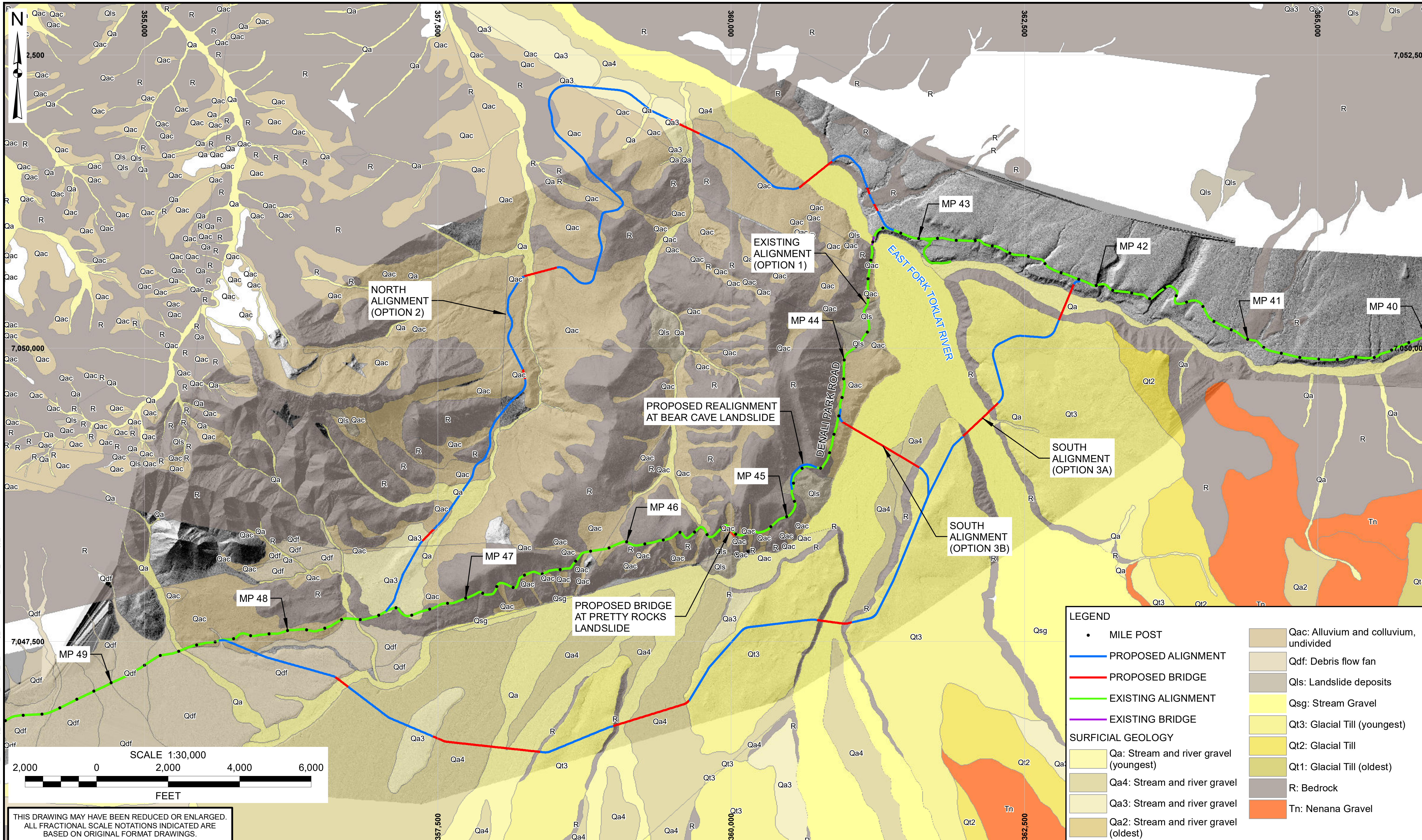

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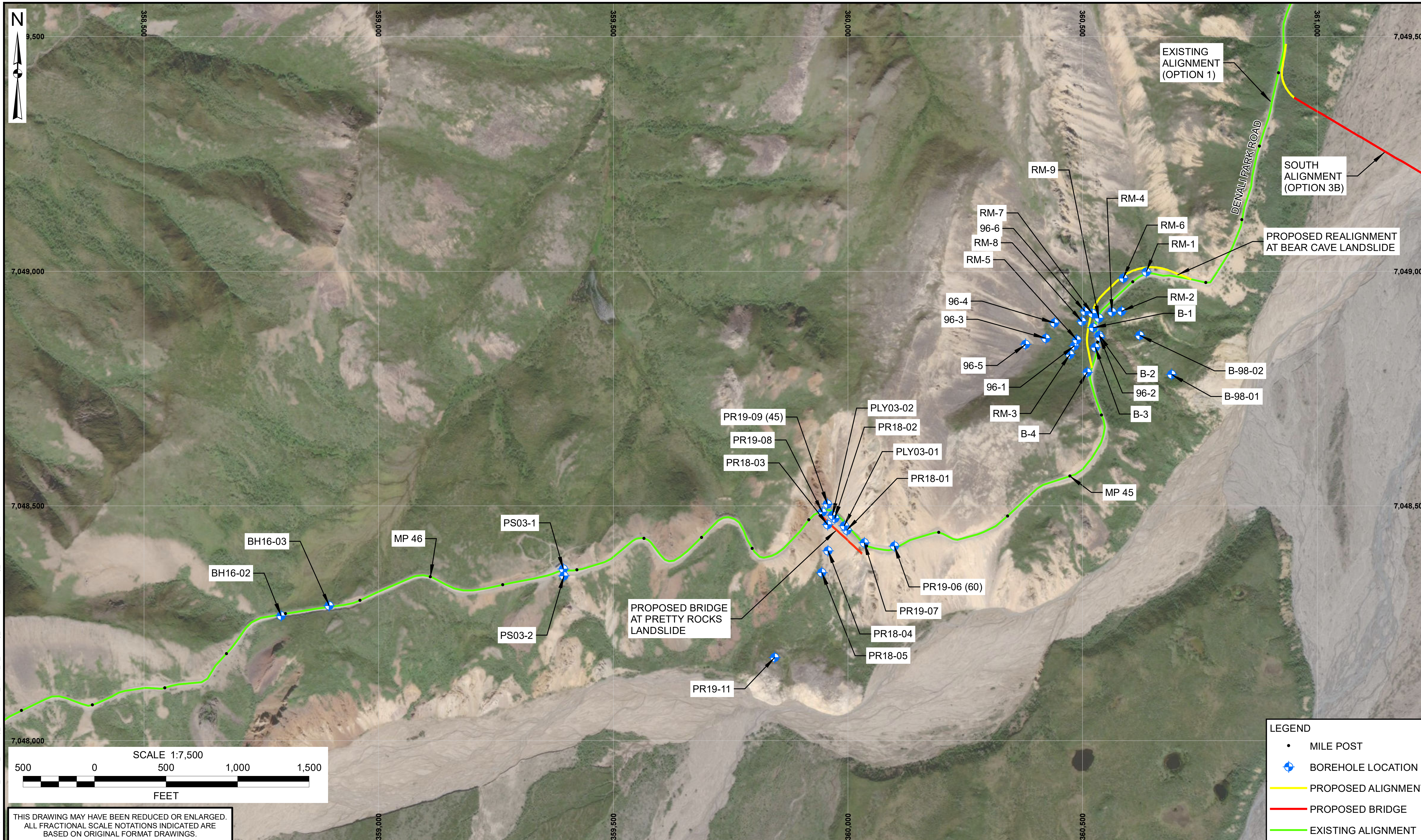
PROJECT:
SUMMARY GEOTECHNICAL REPORT OF EXISTING CONDITIONS

TITLE:
SURFICIAL GEOLOGY MAP

PROJECT No.: 2000003
DWG No: 04



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PROJECT:
SUMMARY GEOTECHNICAL REPORT OF EXISTING CONDITIONS

TITLE:
EXISTING BOREHOLE MAP

PROJECT No.:	DWG No:
2000003	05

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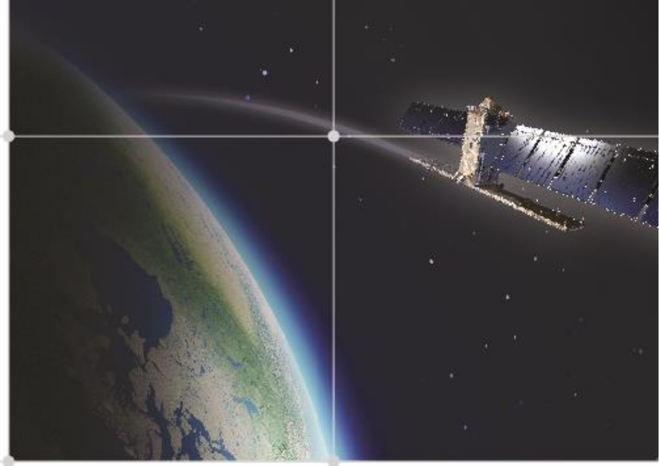
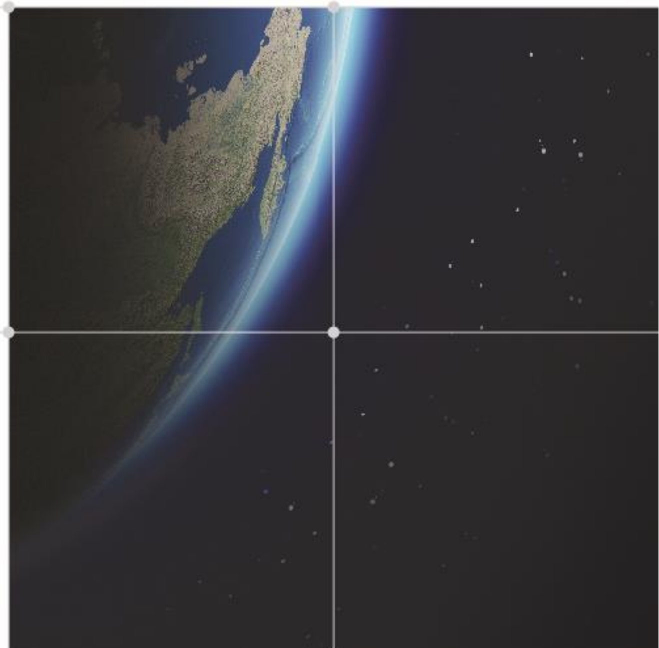
APPENDIX B
INSAR ANALYSIS OF HISTORICAL GROUND DEFORMATION OVER THE DENALI NATIONAL PARK



InSAR Analysis of Historical Ground Deformation over the Denali National Park

Processing Report

March 2020



TRE ALTAMIRA
A CLS Group Company

Report Specifications

Client:	**BGC Engineering Inc.**
Attention:	Scott Anderson
Address:	701 12th Street, Suite 211 Golden, Colorado USA, 80401

Reference:	
Title:	InSAR Analysis of Historical Ground Deformation over the Denali National Park
TRE ALTAMIRA Delivery Reference:	JO20-1064-CA REP 1.0
Client Reference (PO):	2000003.05.01

Prepared by:	**TRE ALTAMIRA Inc.**
Author(s):	Geidy Baldeon
Approved by:	Giacomo Falorni
Date:	13 March 2020
Version:	1.0

Table of Contents

Acronyms and Abbreviations

AOI	Area of Interest
DS	Distributed Scatterer(s)
ESA	European Space Agency
GIS	Geographic Information System
InSAR	Interferometric Synthetic Aperture Radar
LOS	Line of Sight
MP	Measurement Point
PS	Permanent Scatterer(s)
SAR	Synthetic Aperture Radar
SNT	Sentinel Satellite
SqueeSAR®	The most recent InSAR algorithm patented by TRE
TCS	Temporary Coherent Scatterers

1. Introduction

The Denali Park Road is the main access route to the Denali National Park in Alaska, United States. There are a few active landslides, including the Pretty Rocks Landslide, along a segment of the Denali Park Road. As a result of landslide activity, which is causing continuous road repairs and maintenance, two alternative routes are being explored. The area of interest (AOI) is located approximately 50 kilometres (30 miles) southwest of the Denali National Park main entrance and covers the existing Denali Park Road near the Pretty Rocks Landslide and the proposed road alternatives (Figure 1).

BGC Engineering Inc. (BGC) is interested in understanding the historical ground movement occurring along the existing Denali Park Road and the proposed road alternatives to aid in the geotechnical investigation of the site. For this purpose, BGC contracted TRE Altamira Inc. (TRE) to carry out a historical InSAR analysis over the site. The current processing report highlights the 2018-2019 ground deformation results, where TRE used its proprietary SqueeSAR® algorithm and low-resolution Sentinel C-band imagery. Appendix 1 provides a summary of all the deliverables.

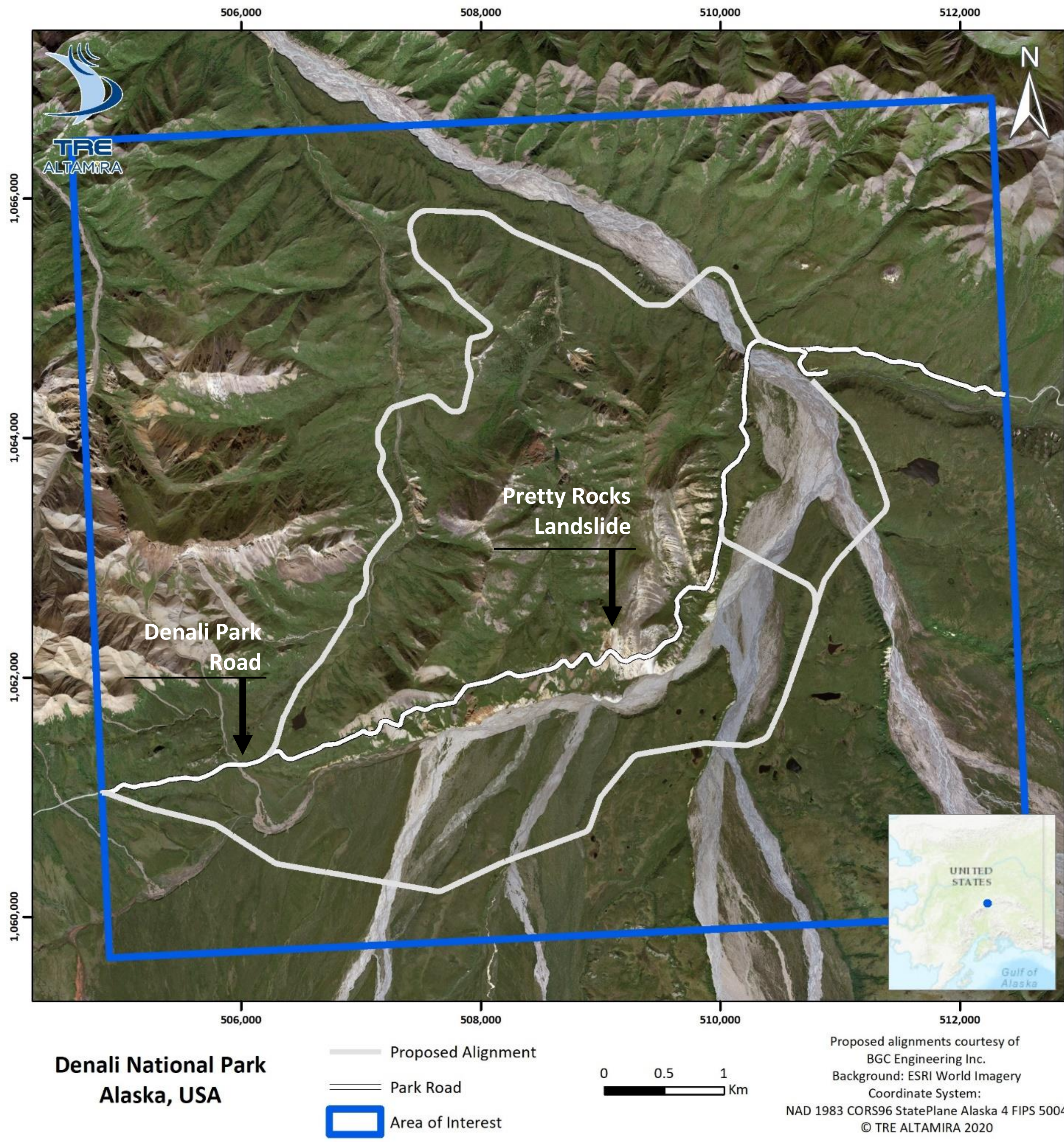


Figure 1: Area of interest.

2. Radar Data

The radar data available over the site consists of publicly available low-resolution images acquired by the European Space Agency (ESA) Sentinel (SNT) satellite from both ascending and descending orbits at a 12-day revisit frequency (Table 1). In an ascending orbit the satellite travels from south to north and images to the east, while in a descending orbit the satellite travels from north to south and images to the west.

To maximize measurement point density, the data processing covers the period May 2018 - September 2019 for both orbits (Figure 2 and Table 1). Images acquired between October 2014 - April 2018 were removed from the processing due to their longer revisit frequency (24-day). Low quality images (most of which are affected by snow coverage) were also removed, as were those acquired after September 2019.

Appendix 2 provides additional information on satellite acquisition parameters used for the current processing.

Table 1: Satellite acquisition parameters and image acquisition information.

Satellite	Spatial Resolution	Orbit	Track	LOS Angle (Θ)	# of Images	Date Range
Sentinel	20 m x 5 m	Ascending	65	40.69°	19	28 May 2018 – 20 Sep 2019
		Descending	131	36.74°	42	14 May 2018 – 30 Sep 2019

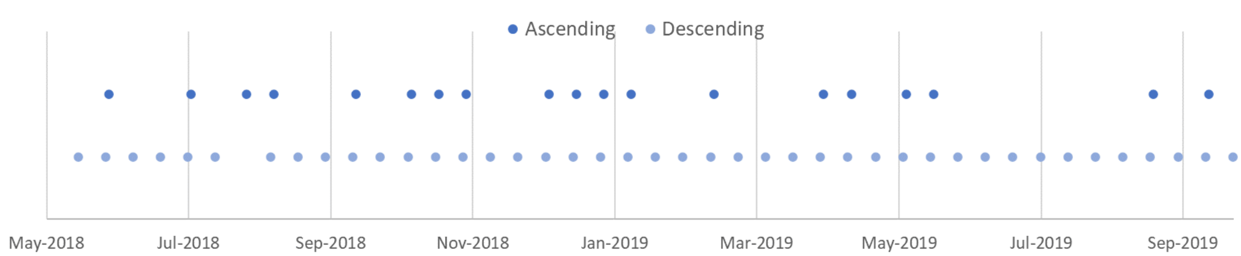


Figure 2: Temporal distribution of Sentinel radar images processed over the site. Gaps denote missed acquisitions.

3. Results

The 2-D SqueeSAR analysis used the temporally overlapping portion of the archives (28 May 2018 – 20 September 2019) and spatially overlapping Line-of-Sight (LOS, ascending and descending data) measurement points on a 40 x 40 m spatial grid to obtain true vertical and east-west horizontal movements. Figure 3 shows the vertical and horizontal (east-west) deformation rates over the entire AOI as measured from the Sentinel data in millimetres per year. Overall, the 2-D SqueeSAR analysis provided an average density of 32 measurement points per square kilometre and an average measurement precision, indicated by the average standard deviation values, of ±2.1 mm/yr (Table 2).

The LOS or 1-D SqueeSAR deformation rates measured in millimetres per year from the ascending archive (28 May 2018 – 20 September 2019) and descending archive (14 May 2018 – 30 September) are shown in Figure 4. These data sets are used as input to produce 2-D (East-West and Vertical) results. The descending data set provides the most coverage over the site, with an average density of 157 measurement points per square kilometre (Table 2) compared to 122 points per square kilometre for the ascending data.

Table 2: Properties of the SqueeSAR analyses.

Attribute	Ascending	Descending	Vertical	East-West
Date Range	28 May 2018 – 20 Sep 2019	14 May 2018 – 30 Sep 2019	28 May 2018 – 20 Sep 2019	
N. of Images	19	42	52	
Total points (PS + DS) **Number of PS** **Number of DS**	6,391 3,419 2,972	8,209 3,524 4,685	1,697	1,697
Average Point Density (pts/km^2)	122	157	32	32
Average Deformation Rate Standard Deviation (mm/yr)	±0.5	±1.7	±2.1	±2.8
Average Time Series Error Bar (mm)	±4.5	±2.9	-	-
Reference Point Location	504,562.6 1,063,894.9	504,551.1 1,064,042.8	509,189 1,064,360	
Coordinate System	NAD 1983 (CORS96) State Plane Alaska 4			
Area (km^2)	52.22			

Figure 5 shows the Temporary Coherent Scatterer (TCS) results for the same ascending and descending archives. The TCS provide higher coverage in areas where there are limited LOS SqueeSAR measurement points.

Overall, the 1-D and 2-D SqueeSAR and 1-D TCS results highlight hot spots of deformation over the existing Denali Park road and the proposed alternatives routes for 2018-2019. The shorter processing timeframe allowed for maximum measurement point coverage, especially in the descending data set. Whenever possible, TRE recommends looking at clusters of points or pixels instead of relying on isolated pixels or points. To further visualize the SqueeSAR data and its deformation time series, log in to TREmaps or download the data to use it in a GIS environment.

Refer to Appendix 3 for an overview of the InSAR techniques used in the current processing.

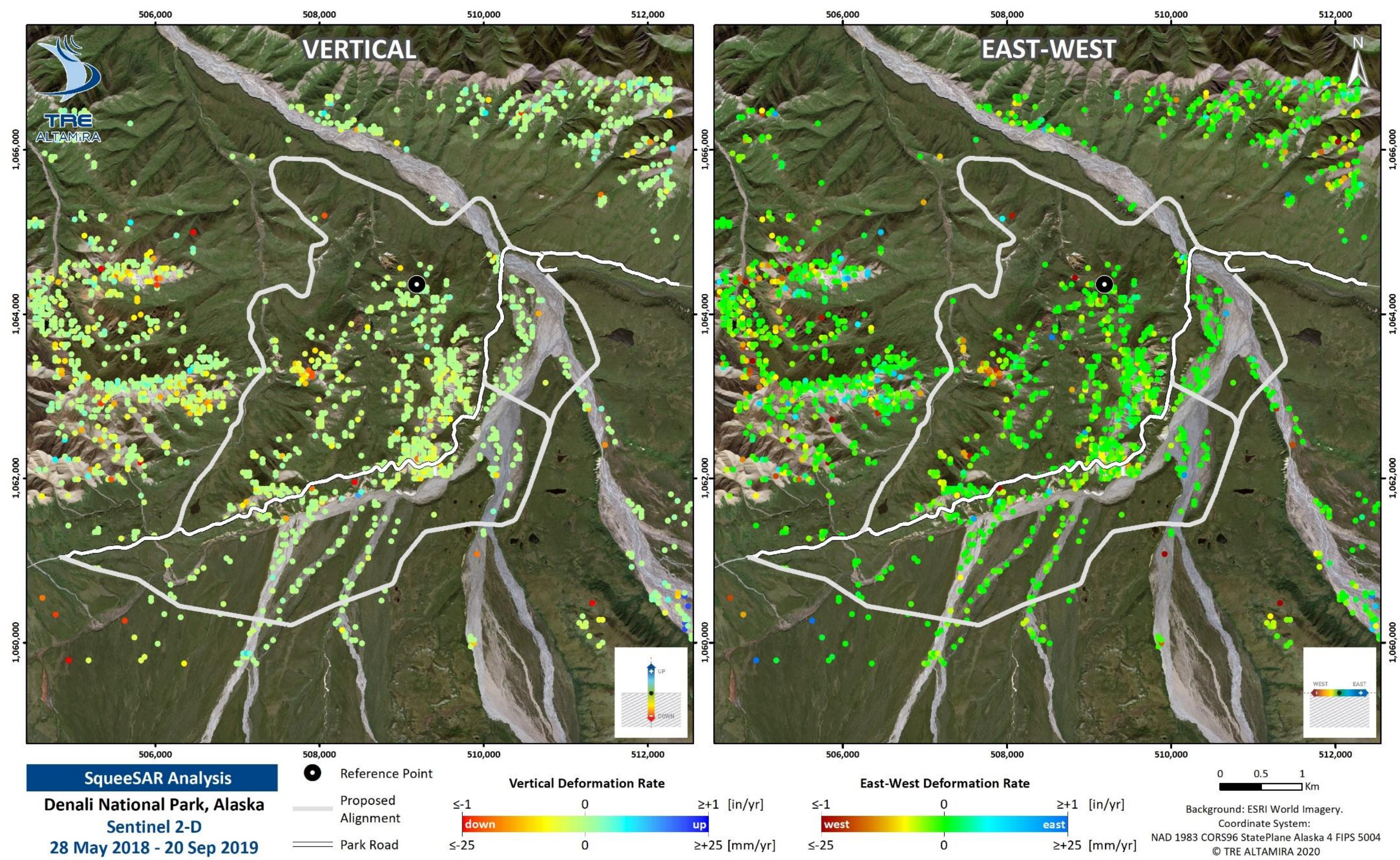


Figure 3: 2-D SqueeSAR results.

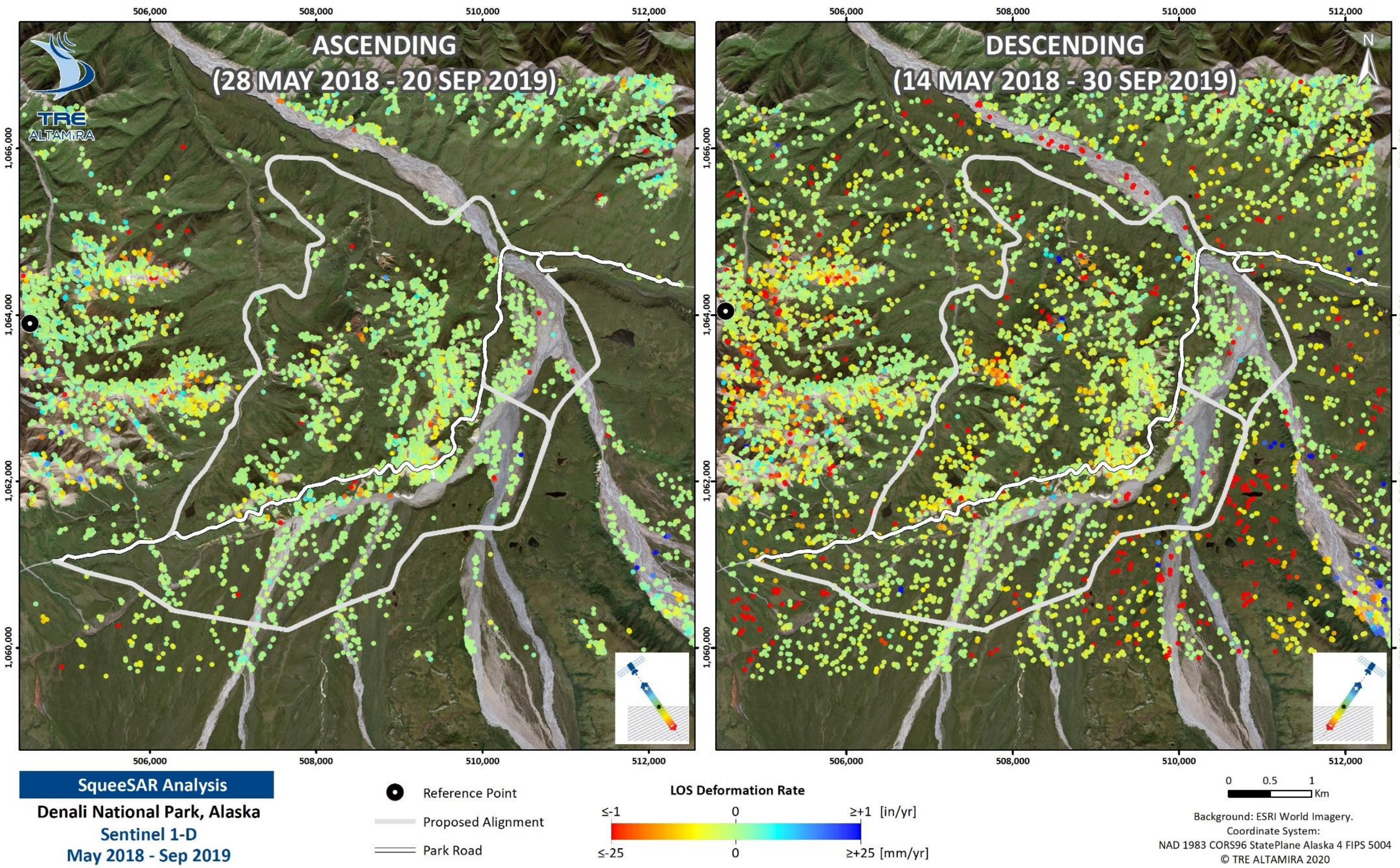


Figure 4: Line-of-Sight SqueeSAR results.

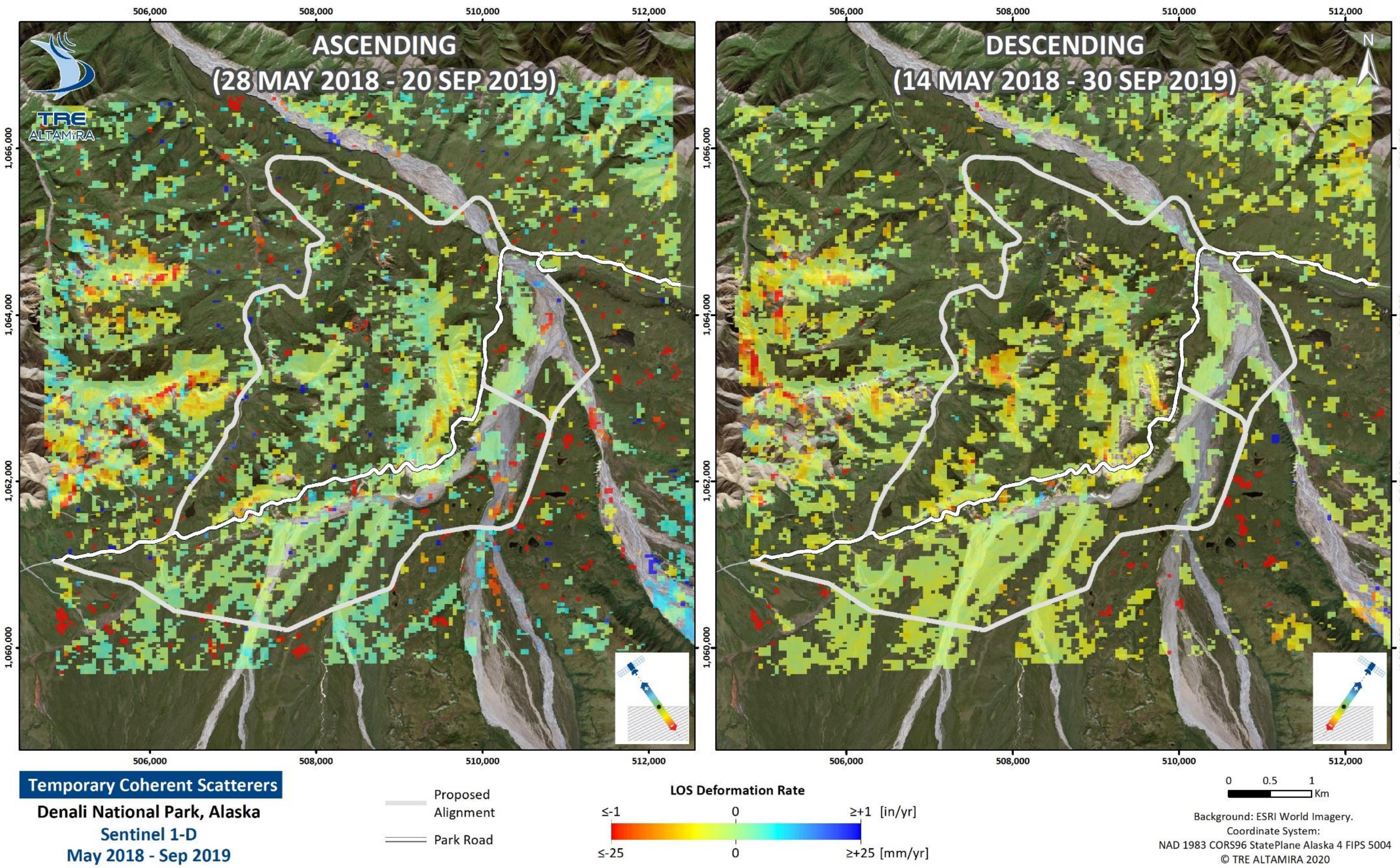


Figure 5: Line-of-Sight TCS results.

Appendix 1: Delivered Files

List of Deliverables

Table 3 list the deliverables including the present report, the InSAR data files and an updated version of the TRE toolbar, a software tool for assisting with the loading, viewing and interrogation of the data in ESRI ArcGIS 10.x software (For set-up procedure and functionalities, see the attached manual *TREToolbarSetup_5.0.pdf*).

Table 3: List of deliverables.

Description	File name
SqueeSAR Data	**Ascending (LOS):** DENALI_PARK_SNT_A_CA2170A1S.shp **Descending (LOS):** DENALI_PARK_SNT_D_CA2170A5S.shp **2-D:** Vertical: DENALI_PARK_SNT_VERT_CA2170A3V.shp East-West: DENALI_PARK_SNT_EAST_CA2170A4E.shp
Temporary Coherent Scatterers (TCS) Deformation rate in GeoTiff format	**Ascending (LOS):** DENALI_PARK_SNT_A_TCS.tif **Descending (LOS):** DENALI_PARK_SNT_D_TCS.tif
MXD project file containing all the data (ESRI ArcGIS version 10.0 and 10.7)	DenaliPark_SNT_Historical_v10-0.mxd DenaliPark_SNT_Historical_v10-7.mxd
Processing Report	Denali Park InSAR Processing Report.pdf
TRE Toolbar (ESRI® ArcGIS 10.x)	TREToolbar_5.0 TREToolbarSetup_5.0.pdf

Database Structure

The SqueeSAR vector data are delivered in a shapefile format and projected to NAD 1983 (CORS96) State Plane Alaska 4 coordinates. The shapefile of each elaboration contains details about the measurement points identified, including deformation rate, elevation, cumulative deformation and quality index. The information associated within the database files (dbf) are described in Table 4.

Table 4: Description of the fields contained in the database of the vector data. *Field is only present in LOS data sets.

Field	Description
CODE	Measurement Point (MP) identification code.
HEIGHT*	Topographic Elevation referred to WGS84 ellipsoid of the measurement point [m].
H_STDEV*	Height standard deviation of the measurement point [m].
VEL	MP deformation rate [mm/yr]. • **Ascending LOS:** Positive values correspond to motion toward the satellite (i.e. uplift and/or westward movement); negative values correspond to motion away from the satellite (i.e. downward and/or eastward movement). • **Descending LOS:** Positive values correspond to motion toward the satellite (i.e. uplift and/or eastward movement); negative values correspond to motion away from the satellite (i.e. downward and/or westward movement). • **Vertical (VEL_V):** Positive values indicate uplift; negative values indicate downward movement. • **E-W Horizontal (VEL_E):** Positive values indicate eastward movement; negative values westward movement.
V_STDEV	Deformation rate standard deviation [mm/yr].
ACC	Acceleration rate [mm/yr^2].
A_STDEV*	Standard deviation of the acceleration value [mm/yr^2].
STD_DEF*	Deformation time series error bar [mm]
EFF_AREA*	This parameter represents the effective extension of the area [m^2] covered by Distributed Scatterers (DS). For permanent scatterers (PS), its value is set to 0.
Dyyyymmdd	Series of columns that contain the deformation values of successive acquisitions relative to the first acquisition available [mm].

TREmaps

TREmaps® is TRE's proprietary online platform that provides users with secure access to view, interrogate and download InSAR data overlaid on an optical image. Little or no training is required to use TREmaps and no specialized GIS software is necessary.

Functionalities include:

- Time-Series tool to view the history of deformation for each measurement point
- Average Time-Series tool to view the average history of deformation for a group of selected points.
- Cross-section tool to view the evolution of the ground surface over time
- Data export (subsets of data) to common formats (SHP, KML, GeoDB, CSV)
- Download of InSAR data (Shapefile format) and reports
- Temporal filtering tool to time slice data on a specified time period
- Integration with client data, including optical images, benchmark locations, wells, etc.

Clients can quickly and securely login with their personalized username and password. TRE Altamira has adopted systems and procedures that comply with industry standards to ensure maximum security and confidentiality of the products.

TREmaps website: https://tremaps5.tre-altamira.com/treaviewer

For assistance on any of the functions, please click the Help icon on the viewer or go to:

https://site.tre-altamira.com/tremaps-getting-started/

For optical performance, we recommend using Google Chrome or Mozilla Firefox.

Appendix 2: Additional Radar Data Details

InSAR-based approaches measure surface deformation on a one-dimensional plane, along the satellite line-of-sight (LOS) and satellite orbit. An ascending orbit denotes a satellite travelling from south to north and imaging to the east, while a descending orbit indicates a satellite travelling from north to south and imaging to the west. The LOS angle varies depending on the satellite and on the acquisition parameters while another important angle, between the orbit direction and the geographic North, is nearly constant. The symbol θ (theta) represents the angle the LOS forms with the vertical and δ (delta) the angle formed with the geographic north. Table 5 lists the values of the angles for this study, while Figure 6 and Figure 7 show the geometry of the image acquisitions over the site for the ascending and descending orbits, respectively.

Table 5: Satellite viewing angles for the study.

Satellite	Wavelength	Orbit	Beam Mode/ Track	Angles	Versors
Sentinel	C-Band 5.55 cm	Ascending	IW / 65	$\theta = 40.69^{o}$ $\delta = 8.12^{o}$	V = 0.758 N = -0.092 E = -0.645
		Descending	IW / 131	$\theta = 36.74^{o}$ $\delta = 14.43^{o}$	V = 0.801 N = -0.149 E = 0.579

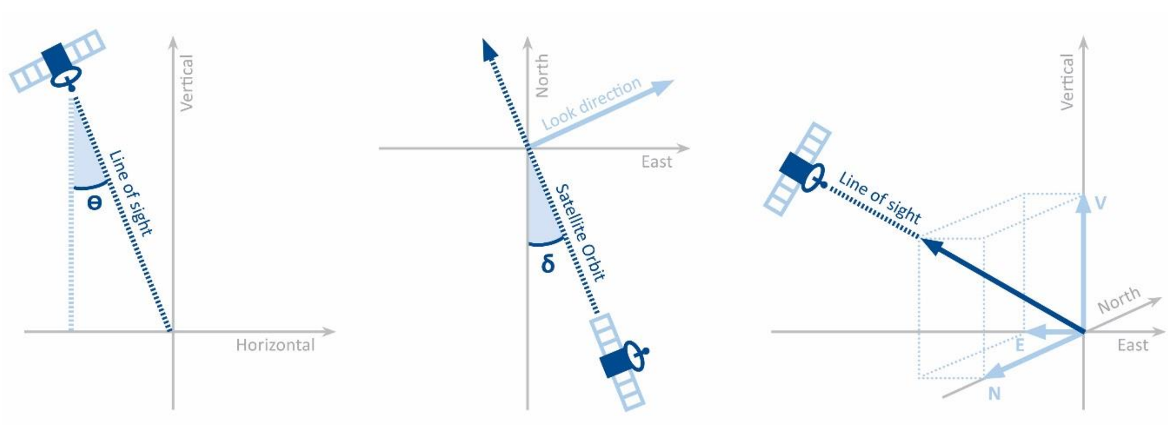


Figure 6: Geometry of the image acquisitions along the ascending orbit.

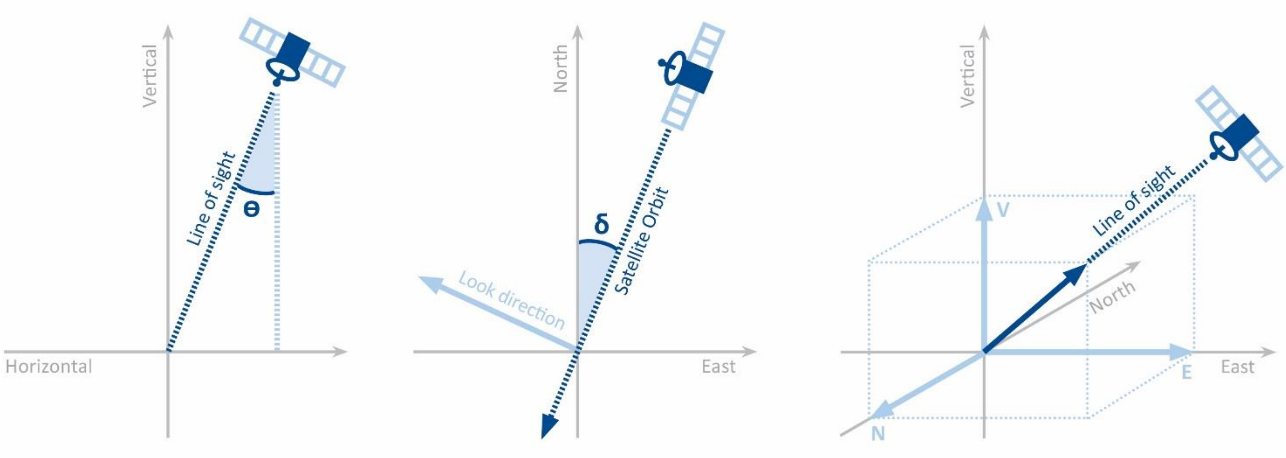


Figure 7: Geometry of the image acquisitions along the descending orbit.

Table 6 lists all the radar images used for the data processing.

Table 6: Radar images acquired over the site by the Sentinel satellite.

SENTINEL Ascending			SENTINEL Descending					
#	**Image Date**	**Frequency**	**#**	**Image Date**	**Frequency**	**#**	**Image Date**	**Frequency**
1	2018-05-28		1	2018-05-14		22	2019-02-02	12
2	2018-07-03	36	2	2018-05-26	12	23	2019-02-14	12
3	2018-07-27	24	3	2018-06-07	12	24	2019-02-26	12
4	2018-08-08	12	4	2018-06-19	12	25	2019-03-10	12
5	2018-09-13	36	5	2018-07-01	12	26	2019-03-22	12
6	2018-10-07	24	6	2018-07-13	12	27	2019-04-03	24
7	2018-10-19	12	7	2018-08-06	24	28	2019-04-15	12
8	2018-10-31	12	8	2018-08-18	12	29	2019-04-27	12
9	2018-12-06	36	9	2018-08-30	12	30	2019-05-09	12
10	2018-12-18	12	10	2018-09-11	12	31	2019-05-21	12
11	2018-12-30	12	11	2018-09-23	12	32	2019-06-02	12
12	2019-01-11	12	12	2018-10-05	12	33	2019-06-14	12
13	2019-02-16	36	13	2018-10-17	12	34	2019-06-26	12
14	2019-04-05	48	14	2018-10-29	12	35	2019-07-08	12
15	2019-04-17	12	15	2018-11-10	12	36	2019-07-20	12
16	2019-05-11	24	16	2018-11-22	12	37	2019-08-01	12
17	2019-05-23	12	17	2018-12-04	12	38	2019-08-13	12
18	2019-08-27	96	18	2018-12-16	12	39	2019-08-25	12
19	2019-09-20	24	19	2018-12-28	12	40	2019-09-06	12
			20	2019-01-09	12	41	2019-09-18	12
			21	2019-01-21	12	42	2019-09-30	12

Appendix 3: Technique Description

SqueeSAR Analysis

SqueeSAR® is an advanced multi-image InSAR algorithm patented by TRE ALTAMIRA that provides high precision measurements of ground deformation in the form of a point cloud. The algorithm identifies measurement points (MPs) from objects on the ground that display a stable return to the satellite in every image of an archive (at least 15 images) and tracks linear and non-linear ground movement. The MPs belong to two different classes (Figure 8):

- **Permanent Scatterers (PS)**: point-wise radar targets characterized by highly stable radar signal return (e.g. buildings, rocky outcrops, linear infrastructures, etc.)
- **Distributed Scatterers (DS)**: patches of ground exhibiting a lower but homogenous radar signal return (e.g. rangeland, debris fields, arid areas, etc.). DS therefore refer to small areas covering several pixels rather than to a single target or object on the ground. For clarity of presentation and ease of interpretation, DS are represented as individual points.

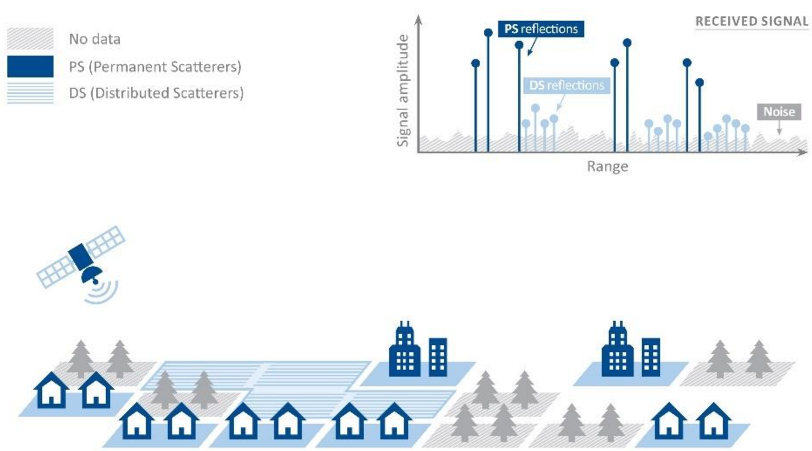


Figure 8: Schematic of PS and DS radar targets.

Each SqueeSAR MP provides the following information:

- Position and elevation estimated with respect to the WGS84 ellipsoid [m]
- Deformation time series (TS) representing the evolution of the deformation for each acquisition date [mm]
- Annual average deformation rate [mm/yr], calculated from a linear regression of the deformation time series over the analysis period.

The density and distribution of the MPs is related to the resolution of the imagery and the surface characteristics of the area. In general, MP density increases with satellite resolution and over areas with bare ground and man-made structures and decreases with the presence of vegetation and over areas with changes to the ground cover over time (e. g. snow, operational activities).

1-D Measurements

In InSAR analyses, all measurements are 1-D readings along the sensor's line-of-sight (LOS) as the true vector of deformation is projected onto the LOS. The same deformation will produce different readings when viewed from different angles (Figure 9). The LOS deformation rates are calculated from a linear regression of the ground movement measured over the entire period covered by the satellite images. Each measurement point corresponds to a Permanent Scatterer (PS) or a distributed scatterer (DS), and color-coded according to its annual rate of movement and direction:

- In a **descending** LOS analysis, negative values (red) indicate surface deformation away from the satellite (i.e. subsidence and/or westward movement), while positive values (blue) indicate surface deformation towards the satellite (i.e. uplift and/or eastward movement).
- In an **ascending** LOS analysis, negative values (red) indicate movement away from the satellite (i.e. subsidence and/or eastward movement) while positive values (blue) indicate movement towards the satellite (i.e. uplift and/or westward movement).

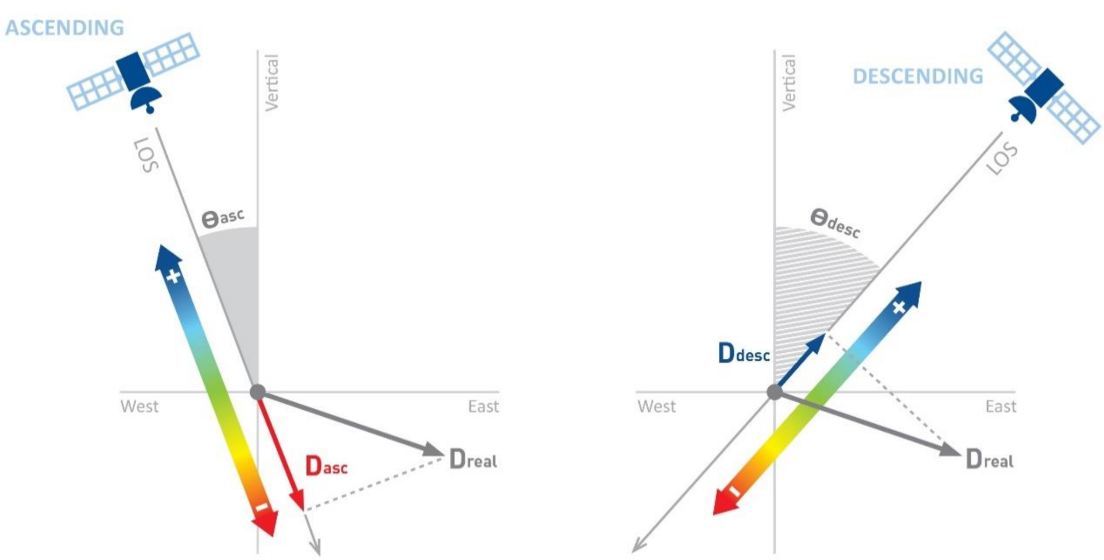


Figure 9: SqueeSAR measures the projection of real movement (D_{real}) along the LOS. The same real movement (D_{real}) will produce a different value from a different LOS (different inclination or different orbits).

2-D Measurements

The trigonometric combination of SqueeSAR results obtained from different orbits (i.e. ascending and descending), over the same area and overlapping period, produces 2-D (vertical and east-west) measurements of ground movement (Figure 10) in a gridded format, as different measurement points are identified from the two orbits. MPs contained within a same cell are averaged and a new unique, derived time series of deformation is obtained for each grid cell (Figure 11).

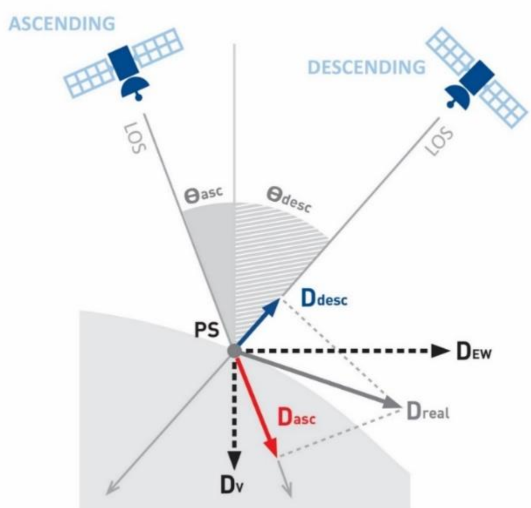


Figure 10: Example of motion decomposition combining ascending and descending orbits.

Figure 11: 2-D measurements are estimated by subsampling ascending and descending data on a common spatial grid. The measurements of all MPs contained within the same cell are averaged to produce 2-D measurement points located at the centre of the cell. The 2-D procedure only produces readings for cells containing MP from both orbits (red cells).

The estimation of the 2-D measurements requires the following steps and assumptions:

- Satellites from different orbits identify different radar targets on the ground, entailing that the 2-D procedure requires a spatial grid to capture MPs from both orbits within each cell. The assumption is that MPs belonging to a same cell are affected by the same motion. All MPs falling within a same cell are then averaged and referred to as synthetic measurement points (sMP). Depending on the satellite resolution, the site is divided into a common grid. Note that the 2-D cells do not represent specific radar targets on the ground but rather synthetic points located at the centre of the cells.
- The 2-D sMP time series of deformation are calculated by combining all ascending and descending time series using trigonometry. The 2-D procedure only produces measurements for cells that contain points from both input LOS data sets. The spatial coverage of the 2-D information is thus generally lower than the coverage of the individual LOS results.
- Since the images are acquired on different dates from each orbit, the LOS deformation time series must be re-sampled in time. The final output includes all ascending and descending acquisition dates and covers the period in common to the two datasets.
- North-south movement cannot be measured with InSAR because SAR satellites are not sensitive to movement parallel to their travel direction.
- Although 2-D measurements are easier to interpret than LOS data, 2-D data have a lower measurement point density, which means that detailed analysis of localized features may benefit from the use the LOS results.

As in the LOS analysis, average annual deformation rates in a 2-D analysis are calculated from a linear regression of the ground movement measured over the entire time interval covered by the analysis and all measurements are relative to a reference point chosen. Each point is color-coded according to the magnitude of movement:

- In a **vertical** data set, negative values (red) indicate downward surface deformation (e.g. subsidence), while positive values (blue) indicate upward surface deformation (e.g. uplift).
- In an **east-west** data set, negative values (red) indicate westward motion, while positive values (blue) indicate eastward motion.

Measurement Precision

SqueeSAR measurements are differential in space and time. Measurements are spatially related to the local reference point, and temporally to the date of the first available satellite image. The local reference point is assumed to be motionless and selected for its radar properties and motion behavior. SqueeSAR measurements contain two precision indices: the deformation rate standard deviation and the time series error bar.

The deformation rate standard deviation characterizes the error associated with the deformation rate with respect to the reference point. Given the standard deviation (σ), and assuming that the errors are normally distributed (Gaussian), 95% of the values tend to be included in a ±2σ range. The deformation rate standard deviation is inversely proportional to the number of processed images and the length of the interval covered by the imagery. This value is evaluated for both the 1-D and the 2-D measurements.

The deformation time series error bar indicates how well an analytical model fits the deformation time series. The model is selected individually for each measurement point with an advanced Model Order Selection technique that also considers the quality of the image archive (number of processed images, time span covered by the archive and possible gaps in the acquisitions). The lower the standard deviation, the lower the average residual with respect to the analytical model (i.e. the smaller the error bar of the time series). This parameter is evaluated only for 1-D measurements.

Table 7 provides a summary of the factors affecting the measurement precision and the geolocation (position in space) precision of the MPs estimated from the 1-D SqueeSAR analysis, as well as typical precision values.

Table 7: Factors affecting the measurement and geolocation precision of SqueeSAR points with typical values at mid-latitudes. Values are referred to a MP less than 1 km from the reference and a dataset of at least 30 radar images covering a 2-year period.

	Measurement Precision	Geolocation Precision
Factors	• Period of analysis • Temporal continuity of acquisitions • Number of images processed • Distance from the reference point (REF) • Measurement point density	• Satellite resolution • Satellite orbit accuracy (normal baseline) • Number of radar images (for z values) • Absolute accuracy of the REF
Typical Values	**Deformation Rate Standard Deviation:** <1 mm/yr **Time series Error Bar**: ±5 mm	**Sentinel** x = ± 12 m y = ± 8 m z = ± 8 m

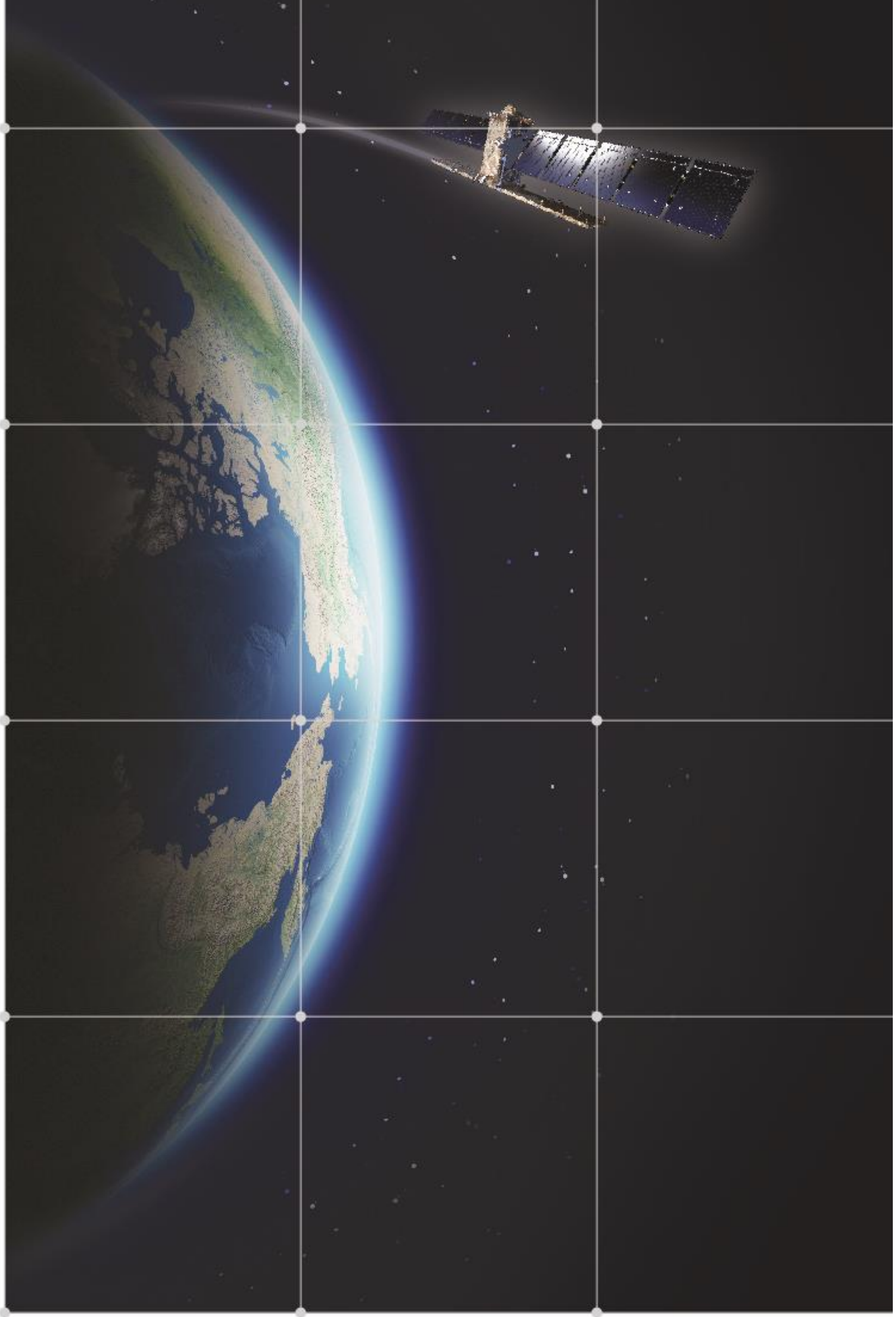
Temporary Coherent Scatterers

Temporary Coherent Scatterers (TCS) provide additional information based upon the extraction of temporary radar targets from an image stack. Compared to SqueeSAR, TCS represent points that are coherent within a subset of the image stack rather than within the entire archive. TCS:

- Provide an average deformation rate within the period of analysis for 1-D LOS measurements
- Have a raster format

- Do not provide deformation time series nor the exact period of coherence

The TCS analysis is effective in areas affected by strong coherence variations, for example, seasonal variations caused by snow and/or vegetation coverage. In those areas, TCS results typically leads to a greater spatial coverage of the results, including over areas where SqueeSAR measurement points cannot be identified. Thus, the combination of TCS and SqueeSAR data allow the maximum deformation information to be extracted and aid in the detection and delimitation of the deformation phenomena.



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APPENDIX C
CONCEPTUAL GEOTECHNICAL INVESTIGATION AND INSTRUMENTATION PLAN

Table 1. South Alignment Conceptual Geotechnical Investigation and Instrumentation Plan

BOREHOLE ID & LOCATION						SAMPLES			INSTRUMENTATION							Comments: Drill Type, Access, Instrumentation, Testing, etc.
Boring #	Type (bridge, landslide, earthwork)	Latitude (deg N)	Longitude (deg W)	Max. Hole Diameter (in)	Estimated Average Depth (ft)	SPT/2.5"/3" Split Barrel Sampling Intervals	Estimated No. of SPT/2.5"/3"	Possible Shelby Samples	Estimated Feet of Ream Casing (5 in I.D.)	VWP Dataloggers (each)	VWP (ft)	Thermister Dataloggers (each)	Thermistor String (ft)	3.34-inch SI pipe (ft)	1" Sch 40 PVC pipe w/ threaded joints for thermistor string (ft)	
PPS21-01	earthwork	63.55239787	-149.7744801	8	30	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	10	1	0	0		0			30	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-02	bridge	63.54777671	-149.7708818	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	0	0		0			100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-03	bridge	63.54397137	-149.7783191	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	15	0		1	100	100	100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-04	bridge	63.54170038	-149.7834445	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	15	0		1	100		100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-05	bridge	63.54559736	-149.7992511	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	0	0		0			100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-06	earthwork	63.53777119	-149.7846234	8	30	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	10	1	0	0		1	30		30	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-07	bridge	63.52997798	-149.7925959	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	15	0		1	100		100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-08	bridge	63.52993559	-149.8029352	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	1	0	0		0			100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-09	earthwork	63.52960308	-149.80983	8	30	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	10	0	0	0		1	30		30	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-10	bridge	63.52374914	-149.8207161	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	15	0		1	100		100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-11	earthwork	63.52267315	-149.8259548	8	50	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	14	0	0	0		0			50	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-12	bridge	63.52069152	-149.83643	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	1	0	0		0			100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-13	bridge	63.51899205	-149.845116	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	15	0		1	100		100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-14	earthwork	63.5201473	-149.860549	8	50	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	14	0	0	0		0			50	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-15	bridge	63.52055216	-149.866927	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	0	0		0			100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-16	earthwork	63.52118506	-149.8738146	8	30	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	10	0	0	0		0			30	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-17	bridge	63.52324221	-149.8810793	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	15	0		1	100		100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPS21-18	bridge	63.52427028	-149.8842654	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	0	0		0			100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
			TOTALS		**1420**		**356**	**4**	**90**	**0**	**0**	**8**	**660**	**100**	**1420**	

Note: This investigation scope also includes test pits at several different locations with material testing for gradation and aggregate suitability from each pit, and 8 locations for geophysical survey lines to extrapolate from borehole locations and characterize ground ice conditions.

Table 2. North Alignment Conceptual Geotechnical Investigation and Instrumentation Plan

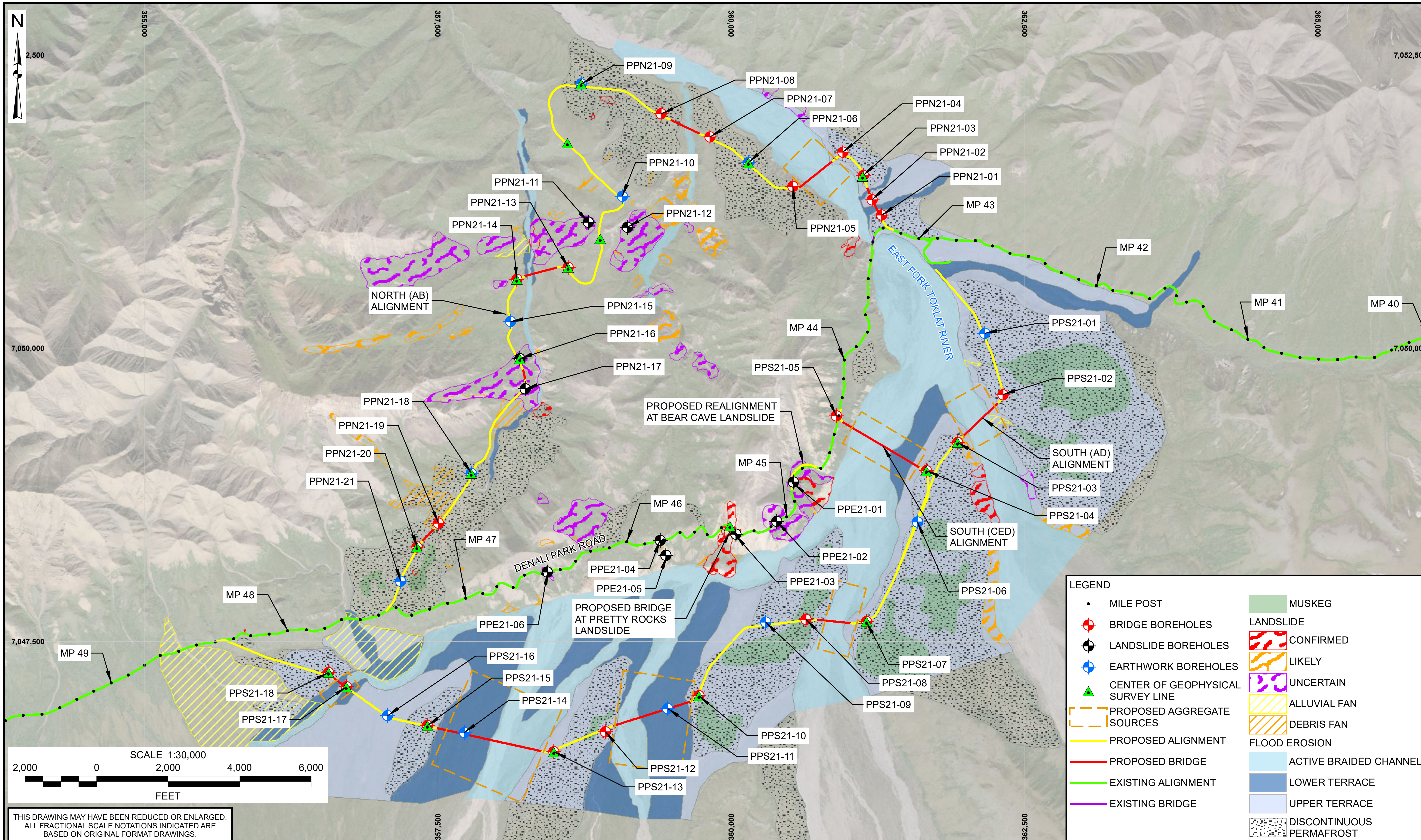
BOREHOLE ID & LOCATION						SAMPLES			INSTRUMENTATION							Comments: Drill Type, Access, Instrumentation, Testing, etc.
Boring #	Type (bridge, landslide, earthwork)	Latitude (deg N)	Longitude (deg W)	Max. Hole Diameter (in)	Estimated Average Depth (ft)	SPT/2.5"/3" Split Barrel Sampling Intervals	Estimated No. of SPT/2.5"/3"	Possible Shelby Samples	Estimated Feet of Ream Casing (5 in I.D.)	VWP Dataloggers (each)	VWP (ft)	Thermister Dataloggers (each)	Thermistor String (ft)	3.34-inch SI pipe (ft)	1" Sch 40 PVC pipe w/ threaded joints for thermistor string (ft)	
PPN21-01	bridge	63.56108443	-149.7930087	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	15	0		1	100		100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-02	bridge	63.56222639	-149.7947894	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	0	0		0			100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-03	bridge	63.56405179	-149.7964376	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	15	0		1	100		100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-04	bridge	63.56576609	-149.8001316	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	0	0		0			100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-05	bridge	63.5629859	-149.8084574	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	15	0		1	100		100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-06	earthwork	63.56464821	-149.8162418	8	30	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	10	1	0	0		0			30	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-07	bridge	63.56647975	-149.8230036	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	0	0		0			100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-08	bridge	63.56808521	-149.8315571	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	15	0		1	100		100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-09	earthwork	63.57009483	-149.8454776	8	30	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	10	1	0	0		1	30		30	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-10	earthwork	63.56165455	-149.8376008	8	30	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	10	0	0	0		0			30	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-11	landslide	63.55955291	-149.8432348	8	50	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	14	0	15	1	50	1	50	50	50	Lab testing: index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-12	landslide	63.55931748	-149.8364642	8	50	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	14	0	15	1	50	1	50	50	50	Lab testing: index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-13	bridge	63.5560241	-149.8463204	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	0	0		0			100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-14	bridge	63.55495071	-149.8550134	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	15	0		1	100		100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-15	earthwork	63.55170419	-149.8557963	8	30	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	10	0	0	0		0			30	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-16	landslide	63.54891073	-149.8538945	8	50	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	14	0	15	1	50	1	50	50	50	Lab testing: index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-17	landslide	63.54663075	-149.8527567	8	50	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	14	0	15	1	50	0		50	50	Lab testing: index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-18	earthwork	63.53997378	-149.861346	8	30	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	10	1	0	0		0			30	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-19	bridge	63.53604781	-149.8665995	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	15	0		1	100		100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-20	bridge	63.53417096	-149.8700551	8	100	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	24	0	0	0		0			100	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPN21-21	earthwork	63.53149565	-149.8726202	8	30	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	10	1	0	0		1	30		30	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
			TOTALS		1480		380	4	150	4	200	11	810	200	1480	

Note: This investigation scope also includes test pits at several different locations with material testing for gradation and aggregate suitability from each pit, and 10 locations for geophysical survey lines to extrapolate from borehole locations and characterize ground ice conditions.

Table 3. Existing Alignment Conceptual Geotechnical Investigation and Instrumentation Plan

BOREHOLE ID & LOCATION						SAMPLES			INSTRUMENTATION							Comments: Drill Type, Access, Instrumentation, Testing, etc.
Boring #	Type (bridge, landslide, earthwork)	Latitude (deg N)	Longitude (deg W)	Max. Hole Diameter (in)	Estimated Average Depth (ft)	SPT/2.5"/3" Split Barrel Sampling Intervals	Estimated No. of SPT/2.5"/3"	Possible Shelby Samples	Estimated Feet of Ream Casing (5 in I.D.)	VWP Dataloggers (each)	VWP (ft)	Thermister Dataloggers (each)	Thermistor String (ft)	3.34-inch SI pipe (ft)	1" Sch 40 PVC pipe w/ threaded joints for thermistor string (ft)	
PPE21-01	landslide	63.540402	-149.80611	8	50	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	14	1	15	1	50	1	50	50	50	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPE21-02	landslide	63.537332	-149.808666	8	50	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	14	1	15	1	50	1	50	50	50	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPE21-03	landslide	63.536215	-149.815571	8	50	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	14	0	15	1	50	1	50	50	50	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPE21-04	landslide	63.535493	-149.828495	8	50	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	14	0	15	1	50	1	50	50	50	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPE21-05	landslide	63.534378	-149.827433	8	150	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	34	1	15	1	150	1	150	150	150	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
PPE21-06	landslide	63.5327	-149.847599	8	50	SPT samples every 2.5 ft starting at the ground surface to a depth of 20 ft, then every 5 ft to the bottom of hole (BOH), or to competent bedrock	14	1	15	1	50	1	50	50	50	Lab testing: salinity, index testing (sieve, hydrometer), moisture content, volumetric ice content ("field logging +"), specific gravity
			TOTALS		**400**		**104**	**4**	**90**	**6**	**400**	**6**	**400**	**400**	**400**	

Note: This investigation scope also includes test pits at several different locations with material testing for gradation and aggregate suitability from each pit, and 1 location for geophysical survey lines to extrapolate from borehole locations and characterize ground ice conditions.



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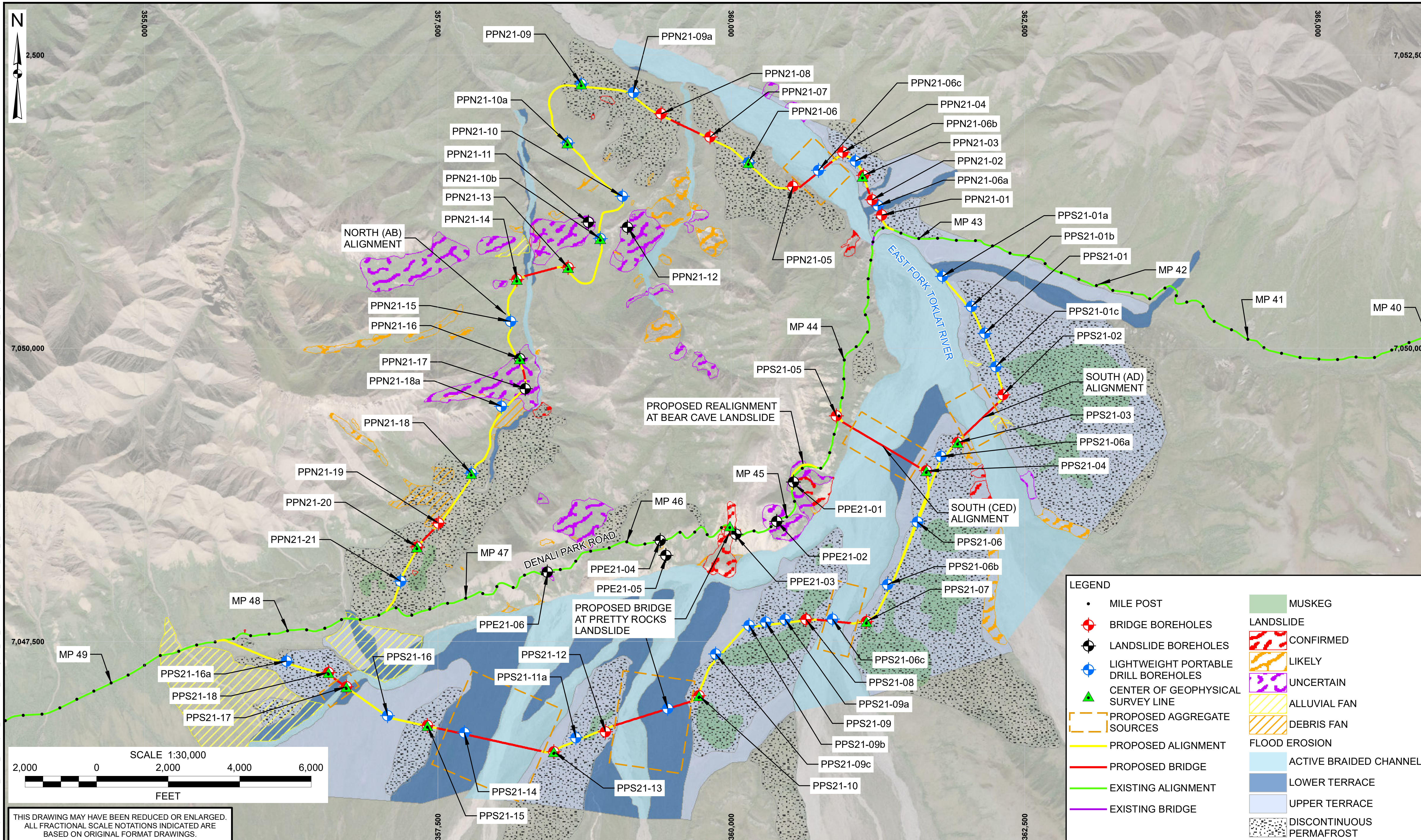
TITLE: PLAN A: CONCEPTUAL GEOTECHNICAL INVESTIGATION AND INSTRUMENTATION PLAN

PROJECT No.:	DWG No:
2000003	01



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3. EXISTING ALIGNMENT COURTESY OF THE U.S. GEOLOGICAL SURVEY. ALTERNATIVE ALIGNMENTS AND HAZARD MAPPING PROVIDED BY WESTERN FEDERAL LANDS HIGHWAY DIVISION AND DENALI NATIONAL PARK, FALL 2019.
4. IMAGERY PROVIDED BY ESRI WORLD IMAGERY, DATED JULY 30, 2014.
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SCALE:	1:30,000
DATE:	MAR 2020
DRAWN:	JVT
CHECKED:	CMC
APPROVED:	SAA

BGC ENGINEERING INC.
AN APPLIED EARTH SCIENCES COMPANY

CLIENT:
JACOBS ENGINEERING / FHWA - WESTERN FEDERAL LANDS HIGHWAY DIVISION

PROJECT:
INTERIM CONCEPTUAL GEOTECHNICAL INVESTIGATION AND INSTRUMENTATION PLAN

TITLE:
PLAN B: ALTERNATIVE CONCEPTUAL GEOTECHNICAL INVESTIGATION AND INSTRUMENTATION PLAN - LIGHTWEIGHT PORTABLE DRILL

PROJECT No.:	DWG No:
2000003	02

APPENDIX D

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Western Federal Lands Highway Division
610 E. Fifth Street
Vancouver, WA 98661-3801

TO:	Lotse Townsend Brandon Stokes Project File
FROM:	Denise Steele, Environmental Protection Specialist
DATE:	March 26, 2020
SUBJECT:	Environmental Feasibility Study AK NPS DENA 10(49), Polychrome Pass

Introduction
This memo presents a preliminary environmental review of four options to repair the Polychrome Pass located in the Denali National Park (NPS). This memo outlines the environmental considerations at the Mainline or existing alignment, the Northern Alignment, and the Southern Alignment.

Location
The project is located on the Denali Park Road within the Denali National Park. The existing road is within a 300 foot wide wilderness corridor.

NEPA
The National Environmental Policy Act (NEPA) analysis and documentation depends on the scope, funding source, and lead federal agency of the future construction project. It is likely that the NEPA document would be an Environmental Assessment or an Environmental Impact Statement. The level of NEPA will likely be influenced by the potential need to build a new road in a designated wilderness area which requires approval from Congress.

Purpose and Need
Landslide movement is increasingly impacting approximately 350 feet of the Denali Park Road at about MP 45.3. Since August 2019, the rate of road subsidence, as a result of the continued landslide movement of the Pretty Rocks Landslide has increased daily. Park Road closure at Polychrome Pass would cause widespread economic impacts to Alaska.

The Park Road services the Toklat Road Camp, and the Eielson Visitor Center. Historic views of Denali can be seen from Stony Overlook to Wonder Lake to Kantishna.

Options
There are three action options being reviewed at this time.

Option 1 or Mainline Alignment is on the existing alignment and the repair area is about 1.5-miles long.

Option 2 or Northern Alignment is 1.5-miles away from the Mainline at its farthest distance from the Mainline. This proposal traverses approximately 6.2-miles and includes eight bridges in pristine wilderness.
Option 3 or Southern Alignment is roughly 0.75-miles away from the Mainline at its farthest distance from the Mainline. This option will require two separate alignment reviews at the beginning of the option, Option 3a begins at East Fork Cabin and has eight bridges. Option 3b begins at milepost 44.3 and has five bridges. Options 3a and 3b traverse about 5.7 to 6.2-miles and include a turnaround on the east side of the East Fork Bridge at MP 43. This option is also in pristine wilderness.

Environmental Resources

For Options 2 and 3, the following environmental resources will need studies to determine impacts. **Land Use; Visual Quality; Floodplains; Hazardous Materials; Cultural and Historic Resources; Wetlands and Waters of the US; Tribal Coordination; Indirect Impacts; Public Involvement; Water Quality; Noise; Air Quality; Wild and Scenic Rivers; Scenic Route; Water Quality; Section 4(f) of the Department of Transportation Act; Section 6(f) of the Land and Water Conservation Act.**

The following applies to all three action options.

Traffic

The Denali National Park restricts access to the majority of the Park Road 89 miles. Traffic will likely not increase due to the existing limits regardless of the chosen option.

Environmental Justice

No Environmental Justice populations are expected to be impacted with any of the proposed alignments.

Recreational Resources

Recreational Resources may be reduced if Option 2 or 3 is chosen because these two options are in wilderness and it isn't clear whether hiking etc. will be allowed.

Biological Resources

- Listed species: None; see attached IPaC list.
- Sustenance
- Migratory bird treaty act; see attached IPaC list.
- Bald and Golden Eagle Act; an eagle survey will need to be completed on all action alignments.

Navigable Waters

There are no navigable waters along the three action options.

Property Acquisition or Right of Way

Wilderness acquisition is needed for the three action options.

List of stakeholders

PARK
USACE
National Park Services Wilderness department
Denver Services Center
Tribes, more specifically because of subsistence impacts
SHPO
Alaska Department of Fish and Game (ADF&G)

Federal Land Management Agency Consistency Determination will be needed for all options.

NPS existing Programmatic Agreements for the Mainline option will be used if appropriate.

Construction Impacts

Construction Compliance

Permits

1. EPA SWPPP permit
   Time to acquire: 2-weeks, once the design process is at least 70% complete.
2. 404 permit
   Time to acquire:
   i. NWP – 3-months, once the design process is at least 70% complete.
   ii. Individual – 12-months, once the design process is at least 70% complete.
3. Wilderness permit. Whether a wilderness permit is needed is unknown at this time.
   Time to acquire: UNKNOWN

Revegetation

Restoration Services Team (RST) with US Forest Service (USFS) may be able to revegetate after construction on the chosen alignment or on the decommissioned part of the Denali Park Road. Frequently the Nation Park Service takes on their own revegetation.

Waste, Storage and Staging facilities

Each option may have different waste, storage and staging facilities and these will be determined further along in the review and development of the project.

Consultation and Coordination:

Consultation and coordination will occur from now until the Notice of Termination of permits.

Estimated Costs:

Project Development

Included are the minimum hours for WFL Environment PE/total through permit closure.

80 hours: Congressional approval
40-80 hours: TO-Delineation, 4(f), 6(f), EA or EIS via consultant
20 hours: Wetlands purchase

40 hours: 404 permit
20 hours: NPDES permit
200 hours: general environmental coordination and additional tasks (ECS/SCR/reviews, PR's etc.)
Total 400-440 hours = ~$40,000 - $44,000

Consultant cost
TO = $60,000 (delineation only) to $400,000 with Delineation, 4(f), 6(f), EA or EIS via consultant. The EIS will be faster if done through a consultant rather than the PARK and/or WFL.

Wetlands purchase 1:1 ratio from Great land Trust non-profit= $85,731/acre purchase. This is a 1:1 ratio, if the ratio is higher the value will be higher.
Harmony Ranch may also have wetland credits and they may be a different price.
Revegetation through RST = $200,000.

United States Department of the Interior

FISH AND WILDLIFE SERVICE
Anchorage Fish And Wildlife Conservation Office
4700 Blm Road
Anchorage, AK 99507
Phone: (907) 271-2888 Fax: (907) 271-2786

In Reply Refer To: March 23, 2020
Consultation Code: 07CAAN00-2020-SLI-0158
Event Code: 07CAAN00-2020-E-00410
Project Name: Polychrome DENA 10(49)

Subject: List of threatened and endangered species that may occur in your proposed project location, and/or may be affected by your proposed project

To Whom It May Concern:

The enclosed species list identifies threatened, endangered, and proposed species, designated critical habitat, and some candidate species that may occur within the boundary of your proposed project and/or may be affected by your proposed project. The species list fulfills the requirements of the U.S. Fish and Wildlife Service (Service) under section 7(c) of the Endangered Species Act (Act) of 1973, as amended (16 U.S.C. 1531 *et seq.*). Please note that candidate species are not included on this list. We encourage you to visit the following website to learn more about candidate species in your area: http://www.fws.gov/alaska/fisheries/fieldoffice/anchorage/endangered/candidate_conservation.htm

New information based on updated surveys, changes in the abundance and distribution of species, changed habitat conditions, or other factors could change this list. Please feel free to contact us if you need more current information or assistance regarding the potential impacts to federally proposed, listed, and candidate species and federally designated and proposed critical habitat. Please note that under 50 CFR 402.12(e) of the regulations implementing section 7 of the Act, the accuracy of this species list should be verified after 90 days. This verification can be completed formally or informally as desired. The Service recommends that verification be completed by visiting the ECOS-IPaC website at regular intervals during project planning and implementation for updates to species lists and information. An updated list may be requested through the ECOS-IPaC system by completing the same process used to receive the enclosed list.

The purpose of the Act is to provide a means whereby threatened and endangered species and the ecosystems upon which they depend may be conserved. Under sections 7(a)(1) and 7(a)(2) of the Act and its implementing regulations (50 CFR 402 *et seq.*), Federal agencies are required to utilize their authorities to carry out programs for the conservation of threatened and endangered species and to determine whether projects may affect threatened and endangered species and/or designated critical habitat.

A Biological Assessment is required for construction projects (or other undertakings having similar physical impacts) that are major Federal actions significantly affecting the quality of the human environment as defined in the National Environmental Policy Act (42 U.S.C. 4332(2)(c)). For projects other than major construction activities, the Service suggests that a biological evaluation similar to a Biological Assessment be prepared to determine whether the project may affect listed or proposed species and/or designated or proposed critical habitat. Recommended contents of a Biological Assessment are described at 50 CFR 402.12.

If a Federal agency determines, based on the Biological Assessment or biological evaluation, that listed species and/or designated critical habitat may be affected by the proposed project, the agency is required to consult with the Service pursuant to 50 CFR 402. In addition, the Service recommends that candidate species, proposed species and proposed critical habitat be addressed within the consultation. More information on the regulations and procedures for section 7 consultation, including the role of permit or license applicants, can be found in the "Endangered Species Consultation Handbook" at:

http://www.fws.gov/endangered/esa-library/pdf/TOC-GLOS.PDF

Please be aware that bald and golden eagles are protected under the Bald and Golden Eagle Protection Act (16 U.S.C. 668 *et seq.*), and projects affecting these species may require development of an eagle conservation plan (http://www.fws.gov/windenergy/eagle_guidance.html). Additionally, wind energy projects should follow the wind energy guidelines (http://www.fws.gov/windenergy/) for minimizing impacts to migratory birds and bats.

Guidance for minimizing impacts to migratory birds for projects including communications towers (e.g., cellular, digital television, radio, and emergency broadcast) can be found at: http://www.fws.gov/migratorybirds/CurrentBirdIssues/Hazards/towers/towers.htm; http://www.towerkill.com; and http://www.fws.gov/migratorybirds/CurrentBirdIssues/Hazards/towers/comtow.html.

We appreciate your concern for threatened and endangered species. The Service encourages Federal agencies to include conservation of threatened and endangered species into their project planning to further the purposes of the Act. Please include the Consultation Tracking Number in the header of this letter with any request for consultation or correspondence about your project that you submit to our office.

Attachment(s):

- Official Species List

Official Species List

This list is provided pursuant to Section 7 of the Endangered Species Act, and fulfills the requirement for Federal agencies to "request of the Secretary of the Interior information whether any species which is listed or proposed to be listed may be present in the area of a proposed action".

This species list is provided by:

Anchorage Fish And Wildlife Conservation Office
4700 Blm Road
Anchorage, AK 99507
(907) 271-2888

This project's location is within the jurisdiction of multiple offices. Expect additional species list documents from the following office, and expect that the species and critical habitats in each document reflect only those that fall in the office's jurisdiction:

Fairbanks Fish And Wildlife Conservation Office
101 12th Avenue
Room 110
Fairbanks, AK 99701-6237
(907) 456-0203

Project Summary

Consultation Code: 07CAAN00-2020-SLI-0158

Event Code: 07CAAN00-2020-E-00410

Project Name: Polychrome DENA 10(49)

Project Type: TRANSPORTATION

Project Description: Wilderness N and S and Mainline

Project Location:

Approximate location of the project can be viewed in Google Maps: https://www.google.com/maps/place/62.94584537147049N150.56816313982128W

Counties: Denali, AK | Matanuska-Susitna, AK | Yukon-Koyukuk, AK

Endangered Species Act Species

There is a total of 0 threatened, endangered, or candidate species on this species list.

Species on this list should be considered in an effects analysis for your project and could include species that exist in another geographic area. For example, certain fish may appear on the species list because a project could affect downstream species.

IPaC does not display listed species or critical habitats under the sole jurisdiction of NOAA Fisheries[1], as USFWS does not have the authority to speak on behalf of NOAA and the Department of Commerce.

See the "Critical habitats" section below for those critical habitats that lie wholly or partially within your project area under this office's jurisdiction. Please contact the designated FWS office if you have questions.

---

1. NOAA Fisheries, also known as the National Marine Fisheries Service (NMFS), is an office of the National Oceanic and Atmospheric Administration within the Department of Commerce.

Critical habitats

THERE ARE NO CRITICAL HABITATS WITHIN YOUR PROJECT AREA UNDER THIS OFFICE'S JURISDICTION.

United States Department of the Interior

FISH AND WILDLIFE SERVICE
Fairbanks Fish And Wildlife Conservation Office
101 12th Avenue
Room 110
Fairbanks, AK 99701-6237
Phone: (907) 456-0203 Fax: (907) 456-0208

In Reply Refer To: March 23, 2020
Consultation Code: 07CAFB00-2020-SLI-0083
Event Code: 07CAFB00-2020-E-00234
Project Name: Polychrome DENA 10(49)

Subject: List of threatened and endangered species that may occur in your proposed project location, and/or may be affected by your proposed project

To Whom It May Concern:

The enclosed species list identifies threatened, endangered, proposed and candidate species, as well as proposed and final designated critical habitat, that may occur within the boundary of your proposed project and/or may be affected by your proposed project. The species list fulfills the requirements of the U.S. Fish and Wildlife Service (Service) under section 7(c) of the Endangered Species Act (Act) of 1973, as amended (16 U.S.C. 1531 *et seq.*).

New information based on updated surveys, changes in the abundance and distribution of species, changed habitat conditions, or other factors could change this list. Please feel free to contact us if you need more current information or assistance regarding the potential impacts to federally proposed, listed, and candidate species and federally designated and proposed critical habitat. Please note that under 50 CFR 402.12(e) of the regulations implementing section 7 of the Act, the accuracy of this species list should be verified after 90 days. This verification can be completed formally or informally as desired. The Service recommends that verification be completed by visiting the ECOS-IPaC website at regular intervals during project planning and implementation for updates to species lists and information. An updated list may be requested through the ECOS-IPaC system by completing the same process used to receive the enclosed list.

The purpose of the Act is to provide a means whereby threatened and endangered species and the ecosystems upon which they depend may be conserved. Under sections 7(a)(1) and 7(a)(2) of the Act and its implementing regulations (50 CFR 402 *et seq.*), Federal agencies are required to utilize their authorities to carry out programs for the conservation of threatened and endangered species and to determine whether projects may affect threatened and endangered species and/or designated critical habitat.

A Biological Assessment is required for construction projects (or other undertakings having similar physical impacts) that are major Federal actions significantly affecting the quality of the human environment as defined in the National Environmental Policy Act (42 U.S.C. 4332(2)(c)). For projects other than major construction activities, the Service suggests that a biological evaluation similar to a Biological Assessment be prepared to determine whether the project may affect listed or proposed species and/or designated or proposed critical habitat. Recommended contents of a Biological Assessment are described at 50 CFR 402.12.

If a Federal agency determines, based on the Biological Assessment or biological evaluation, that listed species and/or designated critical habitat may be affected by the proposed project, the agency is required to consult with the Service pursuant to 50 CFR 402. In addition, the Service recommends that candidate species, proposed species and proposed critical habitat be addressed within the consultation. More information on the regulations and procedures for section 7 consultation, including the role of permit or license applicants, can be found in the "Endangered Species Consultation Handbook" at:

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Guidance for minimizing impacts to migratory birds for projects including communications towers (e.g., cellular, digital television, radio, and emergency broadcast) can be found at: http://www.fws.gov/migratorybirds/CurrentBirdIssues/Hazards/towers/towers.htm; http://www.towerkill.com; and http://www.fws.gov/migratorybirds/CurrentBirdIssues/Hazards/towers/comtow.html.

We appreciate your concern for threatened and endangered species. The Service encourages Federal agencies to include conservation of threatened and endangered species into their project planning to further the purposes of the Act. Please include the Consultation Tracking Number in the header of this letter with any request for consultation or correspondence about your project that you submit to our office.

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- Official Species List

Official Species List

This list is provided pursuant to Section 7 of the Endangered Species Act, and fulfills the requirement for Federal agencies to "request of the Secretary of the Interior information whether any species which is listed or proposed to be listed may be present in the area of a proposed action".

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Anchorage Fish And Wildlife Conservation Office
4700 Blm Road
Anchorage, AK 99507
(907) 271-2888

Project Summary

Consultation Code: 07CAFB00-2020-SLI-0083

Event Code: 07CAFB00-2020-E-00234

Project Name: Polychrome DENA 10(49)

Project Type: TRANSPORTATION

Project Description: Wilderness N and S and Mainline

Project Location:

Approximate location of the project can be viewed in Google Maps: https://www.google.com/maps/place/62.94584537147049N150.56816313982128W

Counties: Denali, AK | Matanuska-Susitna, AK | Yukon-Koyukuk, AK

Endangered Species Act Species

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---

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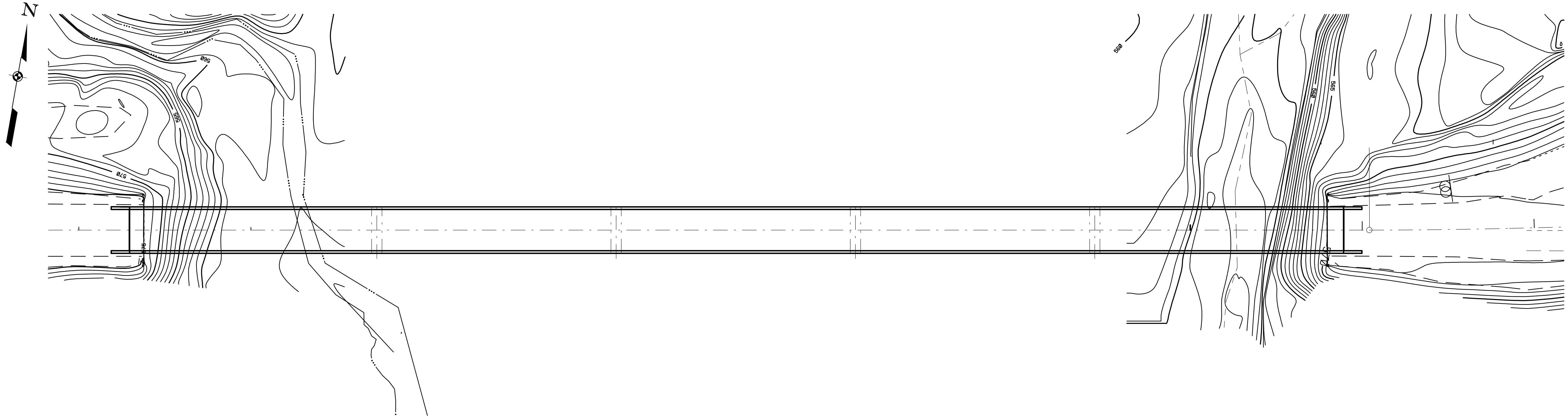
Critical habitats

THERE ARE NO CRITICAL HABITATS WITHIN YOUR PROJECT AREA UNDER THIS OFFICE'S JURISDICTION.

APPENDIX E



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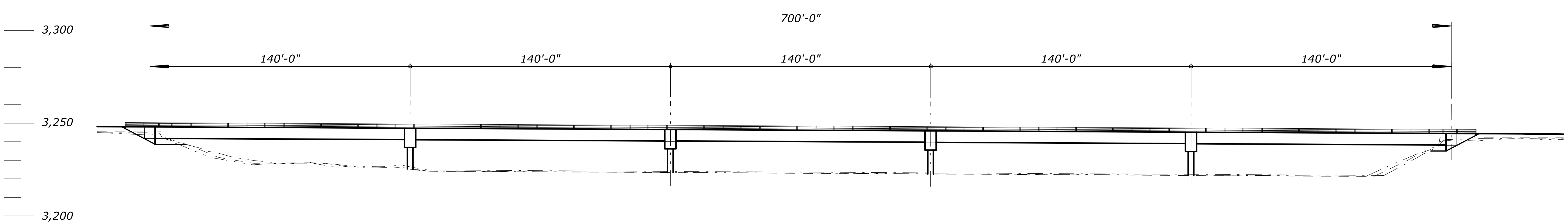


ELEVATION

NOTE - Survey and Mapping data simulated from similar crossing at another location

PRESTRESSED CONCRETE DECKED BULB-TEE GIRDER OPTION

ROUTE NAME AND NUMBER DENALI PARK ROAD
STATE AK COUNTY
ACCOUNT NO.
PREPARED BY B. OLTMANN

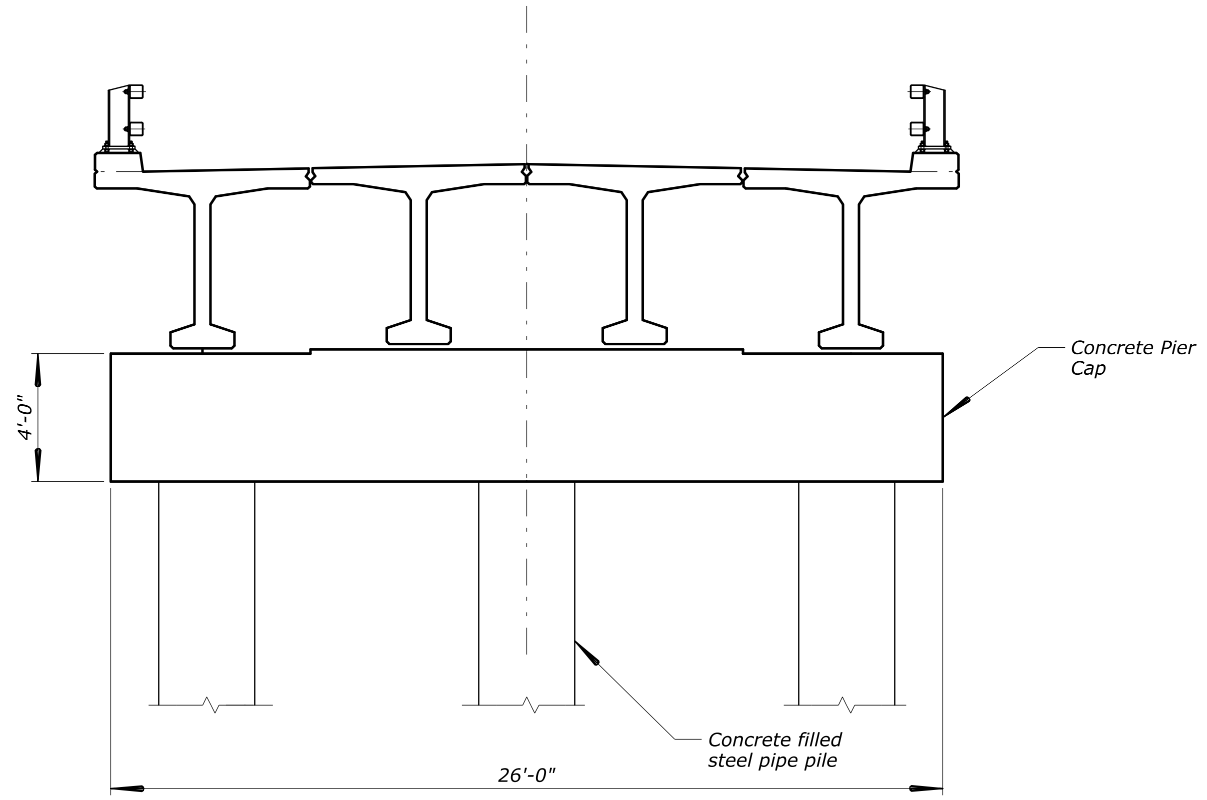
FOREST/PARK/OTHER DENALI NATIONAL PARK
STREAM NAME -
ESTIMATED STRUCTURE COST
SCALE: 1:30 DATE: MARCH 2020

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
WESTERN FEDERAL LANDS HIGHWAY DIVISION

BRIDGE NAME POLYCHROME PASS RE-ALIGNMENT BRIDGES

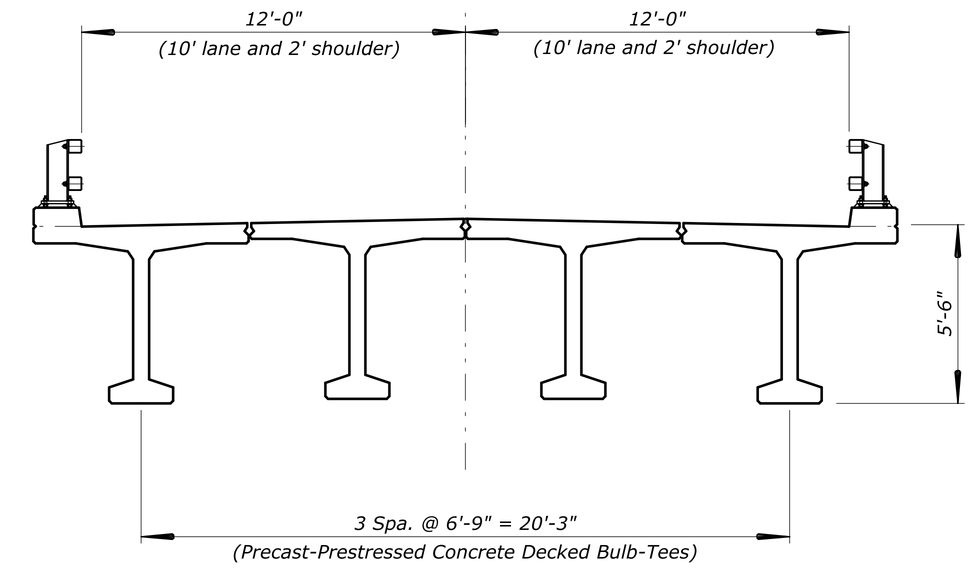
PRELIMINARY LAYOUT

GEOMETRICS



TYPICAL SECTION AT PIER

DESIGN DATA



TYPICAL BRIDGE SECTION

LIVE LOAD ____________ OVERLOAD ______________________ SPECIAL LOADS ______________________

SUPERSTRUCTURE ____

CONCRETE F'C ____

EPOXY REINFORCEMENT ____

RAIL ____________________________ TRANSITION ______________________________________

SIDEWALKS______________ DRAINS _______________ UTILITIES ____________________________

REMARKS____

HYDRAULICS

	Q	Vm	WS EL.
Q_2			
Q_{50}			
Q_{100}			
Q			

REMARKS ____

HYDRAULICS REPORT NO. ____ RECEIVED:________________

SCOUR

SLOPE PROTECTION TYPE: _______________ CLASS:_______________ DEPTH:_______________

TOP EL.: _______________ BOTTOM EL.:_______________ SLOPE:_______________

SCOUR ____

REMARKS ____

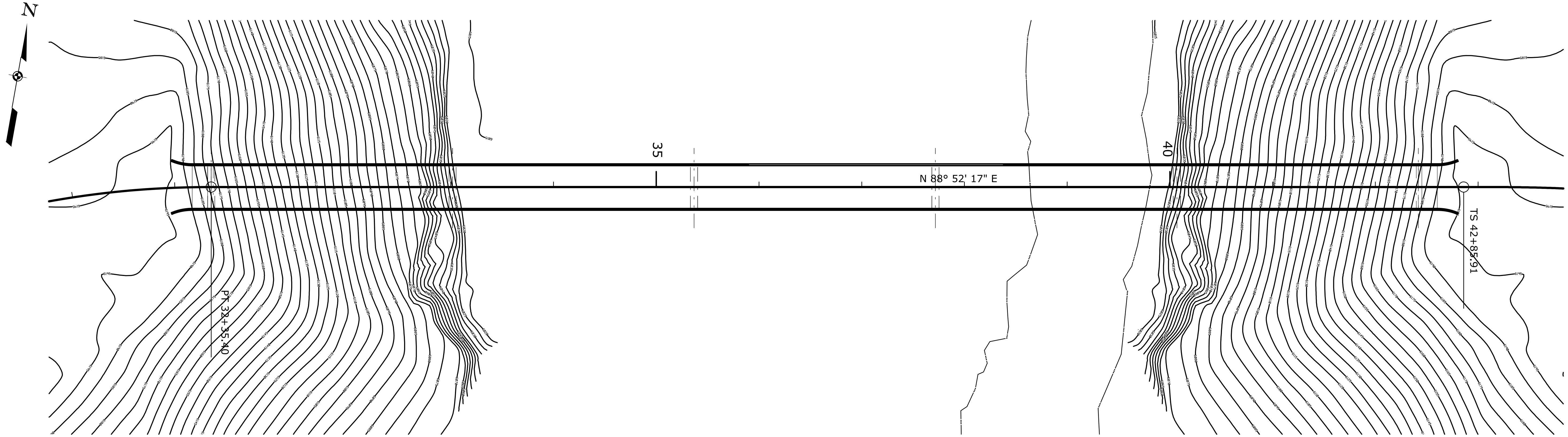
MATERIALS REPORT NO. ____ RECEIVED:_______________

PRESTRESSED CONCRETE DECKED BULB-TEE GIRDER OPTION

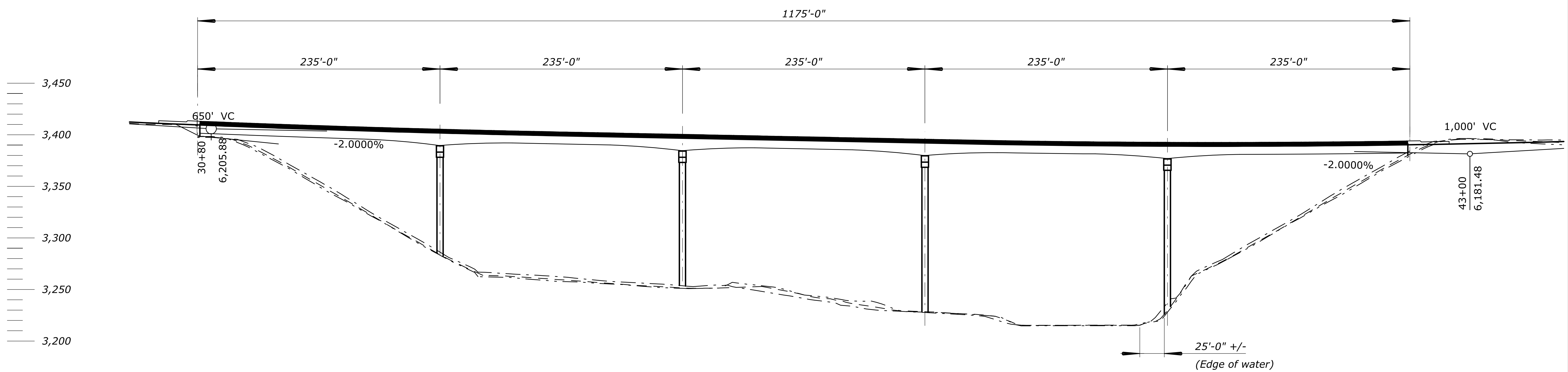
U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
WESTERN FEDERAL LANDS HIGHWAY DIVISION

BRIDGE NAME DENALI CONCRETE GIRDER OPTION

PRELIMINARY LAYOUT



PLAN



ELEVATION

NOTE - Survey and Mapping data simulated from similar crossing at another location

WEATHERING STEEL PLATE GIRDER OPTION

ROUTE NAME AND NUMBER DENALI PARK ROAD
STATE AK COUNTY
ACCOUNT NO.
PREPARED BY B. OLTMANN

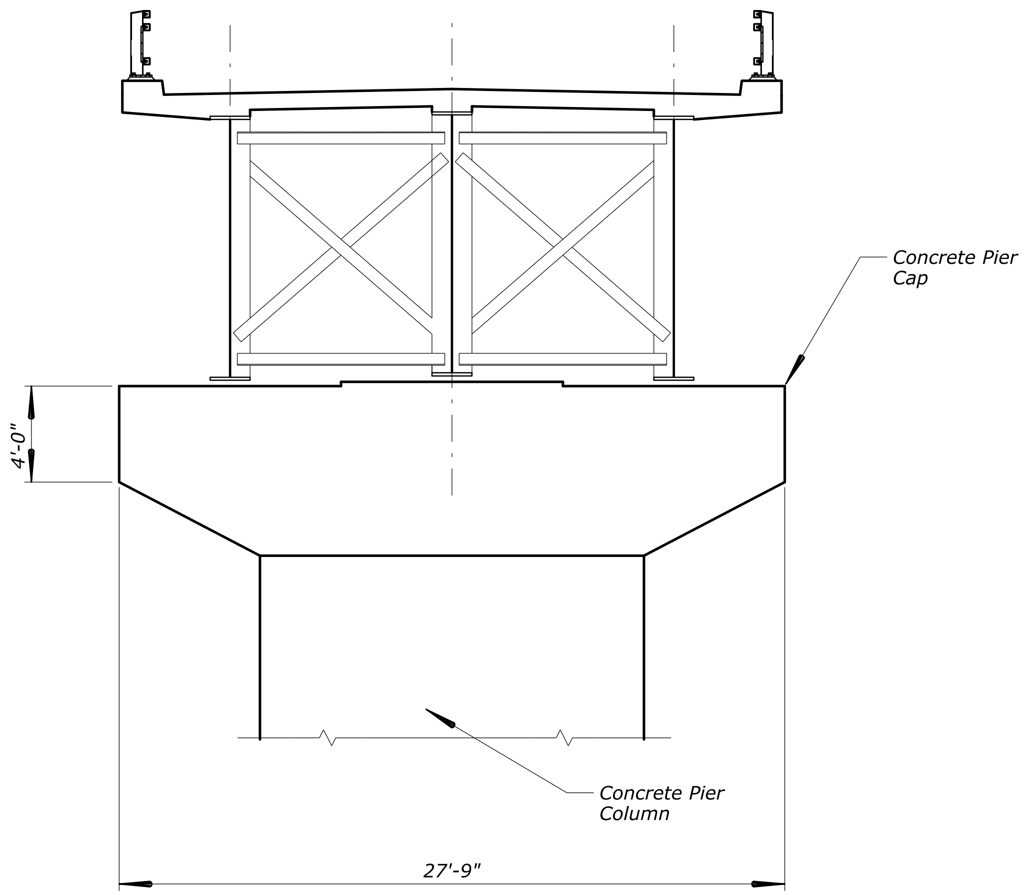
FOREST/PARK/OTHER DENALI NATIONAL PARK
STREAM NAME -
ESTIMATED STRUCTURE COST
SCALE: 1:50 DATE: MARCH 2020

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
WESTERN FEDERAL LANDS HIGHWAY DIVISION

BRIDGE NAME POLYCHROME PASS RE-ALIGNMENT BRIDGES

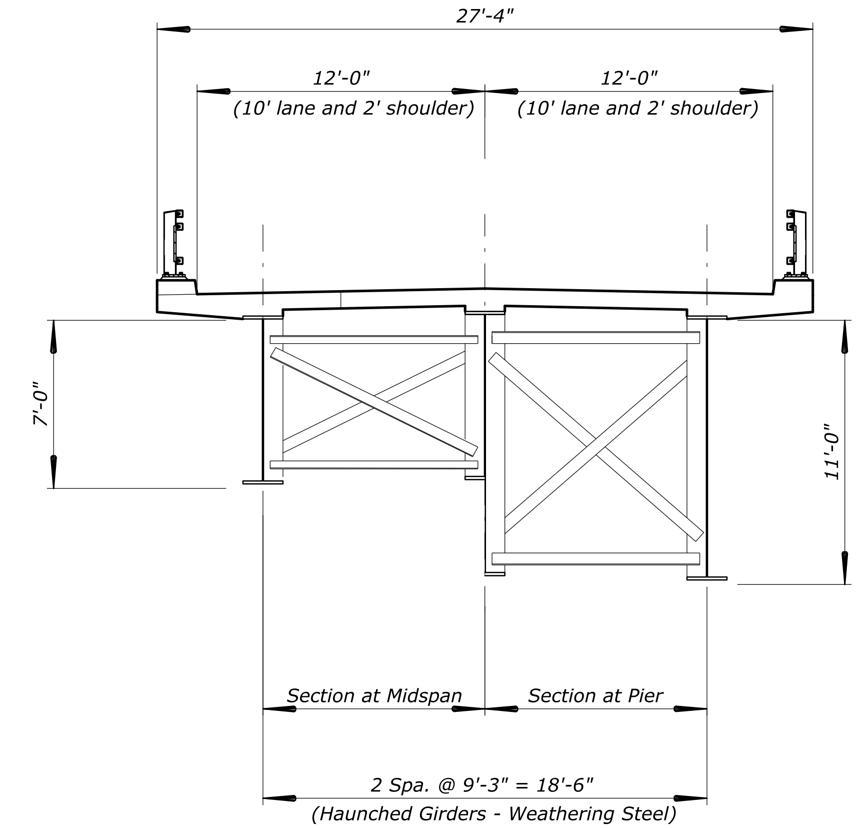
PRELIMINARY LAYOUT

GEOMETRICS



TYPICAL SECTION AT PIER

DESIGN DATA



TYPICAL BRIDGE SECTION

LIVE LOAD ____________ OVERLOAD ______________________ SPECIAL LOADS ______________________

SUPERSTRUCTURE ____

CONCRETE F'C ____

EPOXY REINFORCEMENT ____

RAIL ____________________________ TRANSITION ______________________________________

SIDEWALKS______________ DRAINS ______________ UTILITIES ______________________

REMARKS____

HYDRAULICS

	Q	Vm	WS EL.
Q_2			
Q_{50}			
Q_{100}			
Q			

REMARKS ____

HYDRAULICS REPORT NO. ____ RECEIVED:________________

SCOUR

SLOPE PROTECTION TYPE: ________________ CLASS:________________ DEPTH:________________

TOP EL.: ________________ BOTTOM EL.:________________ SLOPE:________________

SCOUR ____

REMARKS ____

MATERIALS REPORT NO. ____ RECEIVED:________________

WEATHERING STEEL PLATE GIRDER OPTION

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
WESTERN FEDERAL LANDS HIGHWAY DIVISION

BRIDGE NAME DENALI STEEL SPAN OPTION

PRELIMINARY LAYOUT

APPENDIX F

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Western Federal Lands Highway Division

610 E. Fifth Street

POLYCHROME PASS ALTERNATIVES ANALYSIS HYDRAULICS MEMO

To: Brandon Stokes, WFLHD Project Manager
From: Matthew Dillin, P.E., WFLHD Hydraulics Engineer
Date: March 30, 2020
Project: AK NPS DENA 10(49)

Background

The Denali Park Road crosses Polychrome Pass (the Pass) near Mile 45.5, within Denali National Park and Preserve (Figure 1). At the Pass, the roadway has experienced recurring large landslides over the past several years. The rate of movement has been increasing annually and Denali National Park (the Park) is being forced to spend more time each year maintaining the route. The Park is concerned that the movement could eventually force them to close the roadway. Due to the annual cost of maintenance and the risk of potential long term closer, the Park is actively pursuing alternative routes. Western Federal Lands Highway Division (WFLHD) has been asked to create a Project Delivery Plan (PDP) that could be used by the National Park Service (NPS) for requesting funds from Congress.

There are currently three alternatives being considered by the Park for crossing Polychrome Pass. WFLHD hydraulics group reviewed each alternative for fluvial geomorphic, hydrologic, and hydraulic considerations. The results of the review are presented in this memo.

Major Drainage Infrastructure

Alternative 1 (Mainline)

The mainline alternative largely maintains the existing Denali Park Road alignment with proposed improvements at high maintenance areas. Improvements include several locations with rock scaling of unstable slopes, a realignment along Bear Cave Sump, and a proposed bridge spanning the active Pretty Rocks slide (Figure 2). The drainage infrastructure in place for the existing roadway will largely be maintained with minor drainage improvements at the Bear Cave Sump realignment. The minimum low-chord height for the Pretty Rocks Slide crossing will be driven by the need to accommodate future anticipated slide debris.

Alternative 2 (North)

The Alternative 2 (north) reroutes the Denali Park Road to the north, pioneering roughly 6-miles of new roadway. The new roadway alignment requires one crossing of the East Fork Toklat River and several additional crossings of moderately sized drainageways (Figure 3). There are more drainage crossings in Alternative 2 (north) compared to Alternatives 3A & 3B (south); however, the additional drainage crossings are generally smaller, the channels are more confined, and the systems appear to be less dynamic. In order to avoid potential long term maintenance issues, it is recommended that proposed bridges over active channels completely span the active channel migration zone. It is recommended that the low-chord for the East Fork Toklat River crossing be set roughly 15-feet above the channel invert and the recommended minimum span between piers be 200-feet to accommodate maintenance, debris, and potential ice buildup. The low-chord heights for the smaller drainageway crossings will vary based on the characteristics of each drainageway; however, it is generally recommended that the low-chord be set a minimum of 10-feet above the channel invert to accommodate maintenance.

Alternative 3A, 3B (South)
Alternatives 3A and 3B are very similar, they both reroute the Denali Park Road south of the existing Polychrome Pass crossing. Alternative 3A reroutes the roadway sooner, while Alternative 3B reroutes the existing roadway roughly one mile further into the existing alignment (Figure 4). In this configuration, the new roadway must cross four large glacier fed tributaries near the headwaters of the East Fork Toklat River. These streams exhibit a braided channel geometry with multiple-channel watercourses separated by bare channel beds, as depicted in the figure below.


The view from near Polychrome Overlook, looking south toward Polychrome Glaciers (NPS Photo)

This braided geometry is created by the presence of high energy, high sediment loads, and unstable channel banks. In these systems, flow can be frequently diverted from one channel bed into another channel. The change is driven by a complex sequence of erosion and deposition that varies with stage. These systems are generally dynamic and unstable; however, Google Earth aerial imagery and the presence of vegetative cover within the channel migration zone indicates that the location of the active channel has been constant over the past several years. In order to avoid potential long term maintenance issues, it would be a general recommendation to avoid placing structures within these dynamic systems. The initial layout of bridge crossing for the two alternatives has mitigated this issue by proposing bridges that span the entire active channel migration zone. This is a costly solution as the bridge structures become quite large. In order to accommodate maintenance, debris, and ice buildup it is recommended that the low-chord be set roughly 15-feet above the channel invert. To reduce future maintenance it is recommended that piers within the active channel migration zone be spaced 200-feet minimum.

Minor Drainage Infrastructure

Alternative 1 (Mainline)
The mainline alternative generally maintains the existing Denali Park Road with improvements at select high maintenance areas, such as the proposed bridge spanning the Pretty Rocks slide and the realignment at Bear Cave Sump. The existing roadway drainage will be maintained and minor drainage improvements are anticipated to be relatively small. Bear Cave Sump has known drainage issues which will need to be addressed with the realignment.

Alternative 2 (North)
Alternatives 2 also requires a significant amount of new roadway (6-miles); therefore, it too will require the installation of additional minor drainage infrastructure. For this alternative, the roadway alignment will require the installation of roughly 60 additional 24-inch culverts. Given the unique climate and terrain in the Park, more culverts and larger diameter culverts may be required for the pioneered roadway to mitigate for debris, aufeis, and potential aggradation.

Alternative 3A, 3B (South)
Alternative 3A and 3B require the construction of 6-miles and 5-miles of new roadway, respectively. Additional minor drainage infrastructure will be needed to support the newly pioneered road alignments. On the typical project, WFLHD recommends the installation of one 24-inch cross-drain culvert for every 500-feet of roadway. For these two alternatives, the roadway alignments would require roughly 50 to 60 additional culverts. Given the unique climate and terrain in the Park, more culverts and larger diameter culverts may be required for the pioneered roadway to mitigate for debris, aufeis, and potential aggradation.

Construction

Alternative 1 (Mainline)
The mainline option will have the least amount of additional drainage infrastructure as much of the existing infrastructure is to be maintained. However, construction on the mainline will be difficult due to the presence of traffic and the exposure to steep slopes.

Alternative 2 (North)
Alternative 2 has one crossing of the East Fork Toklat River plus several additional crossings for smaller more confined streams. The East Fork Toklat River is fed by glacier meltwater, therefore high flows are likely to occur during the construction season. Flow rates for the East Fork Toklat River and the additional drainage crossings will likely be manageable during the construction season using diversions and dewatering techniques. However, due the dynamic nature of the braided systems the diversion of the East Fork Toklat River crossing may require more frequent monitoring and maintenance.

Alternative 3A, 3B (South)
Since the tributaries to the East Fork Toklat River are fed by glacier meltwater, the high flows are likely to occur during the construction season. Braided systems generally have low discharges comparable to their width so flow rates will likely be manageable during construction using diversion and dewatering techniques. However, due to the dynamic nature of the braided systems diversions required within the braided glacier fed streams may require more frequent monitoring and maintenance.

Operation and Maintenance

Alternative 1 (Mainline)
Alternative 1 (mainline) will generally rely on the existing infrastructure already in use. Compared to the other alternatives, the history and maintenance requirements for Alternative 1 is relatively well-known. WFLHD Hydraulics Group has not visited the site to assess the condition of the existing drainage infrastructure; however, it is assumed that the existing drainage is functioning outside of the Pretty Rocks Slide and Bear Cave Sump areas.

Alternative 2 (North)
Alternative 2 (north) will cross the East Fork Toklat River and several moderately sized drainageways. The proposed East Fork Toklat River crossing spans the entire active channel migration zone, limiting the potential maintenance associated with crossing such a dynamic system. Debris will need to be periodically cleared from the piers located within the active channel migration zone. The several smaller drainage crossings have little information on annual flows, sediment transport, or instability and could become

potential maintenance issues once the structures are in place. Alternative 2 also requires a relatively large amount of new minor drainage infrastructure that will need to be maintained. Issues such as channel debris, debris flows, ice jams, aufeis, and long term aggradation/degradation can occur within the Park.

Alternative 3A, 3B (South)
Alternatives 3A & 3B (south) cross four glacial headwater tributaries to the East Fork Toklat River. These alternatives propose spanning the entire active channel migration zone at each crossing, limiting the potential maintenance associated with crossing such dynamic systems. Debris will need to be periodically cleared from the piers located within active channel migration zone. Due to the amount of new roadway associated with these alternatives, they will require a relatively large amount of new drainage infrastructure that will need to be maintained. Issues such as channel debris, debris flows, ice jams, aufeis, and long term aggradation/degradation can occur within the Park.

Floodplain and Flood-Rise Impacts

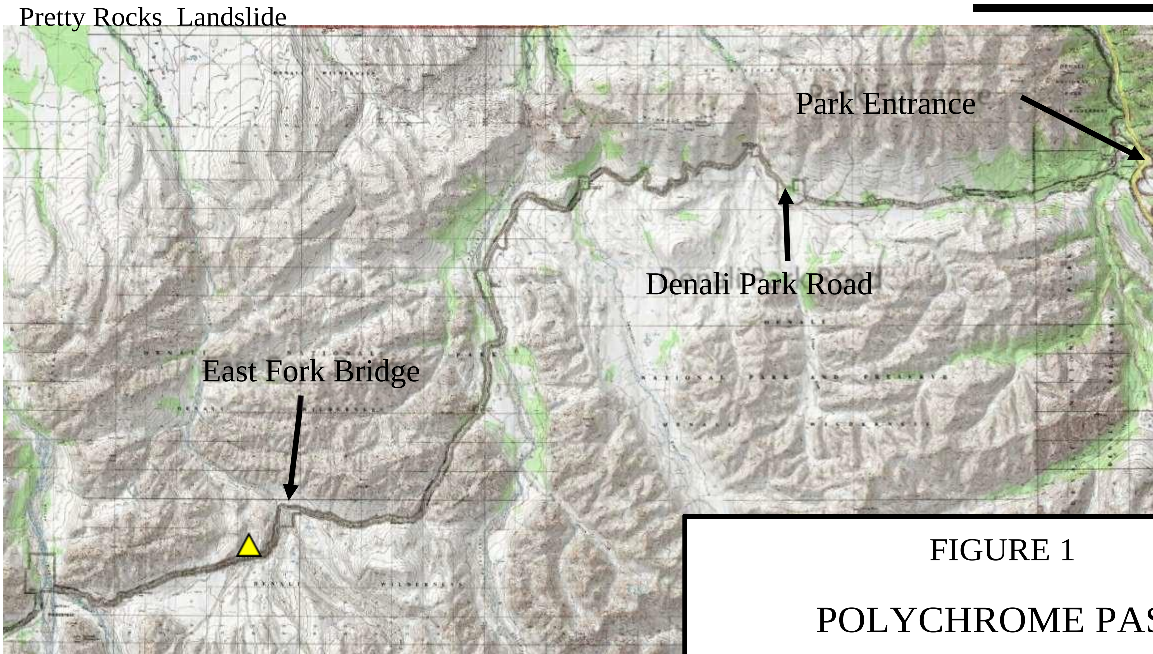
There are no regulatory floodplains mapped within the proposed project limits. Given the remote location of the crossings, none of the alternatives are anticipated to negatively impact any existing insurable structures.

attachments: Figure 1 – Site Location
Figure 2 – Alternatives 1 (Mainline)
Figure 3 – Alternative 2 (North)
Figure 4 – Alternative 3 (South)

East Fork Bridge
Pretty Rocks Landslide

0 5000 feet

N



Map from Google Earth , image date 7/29/2014.

FIGURE 1

POLYCHROME PASS

Site Location

N

0 1 mile

FIGURE 2

POLYCHROME PASS

Alternative 1 - Mainline

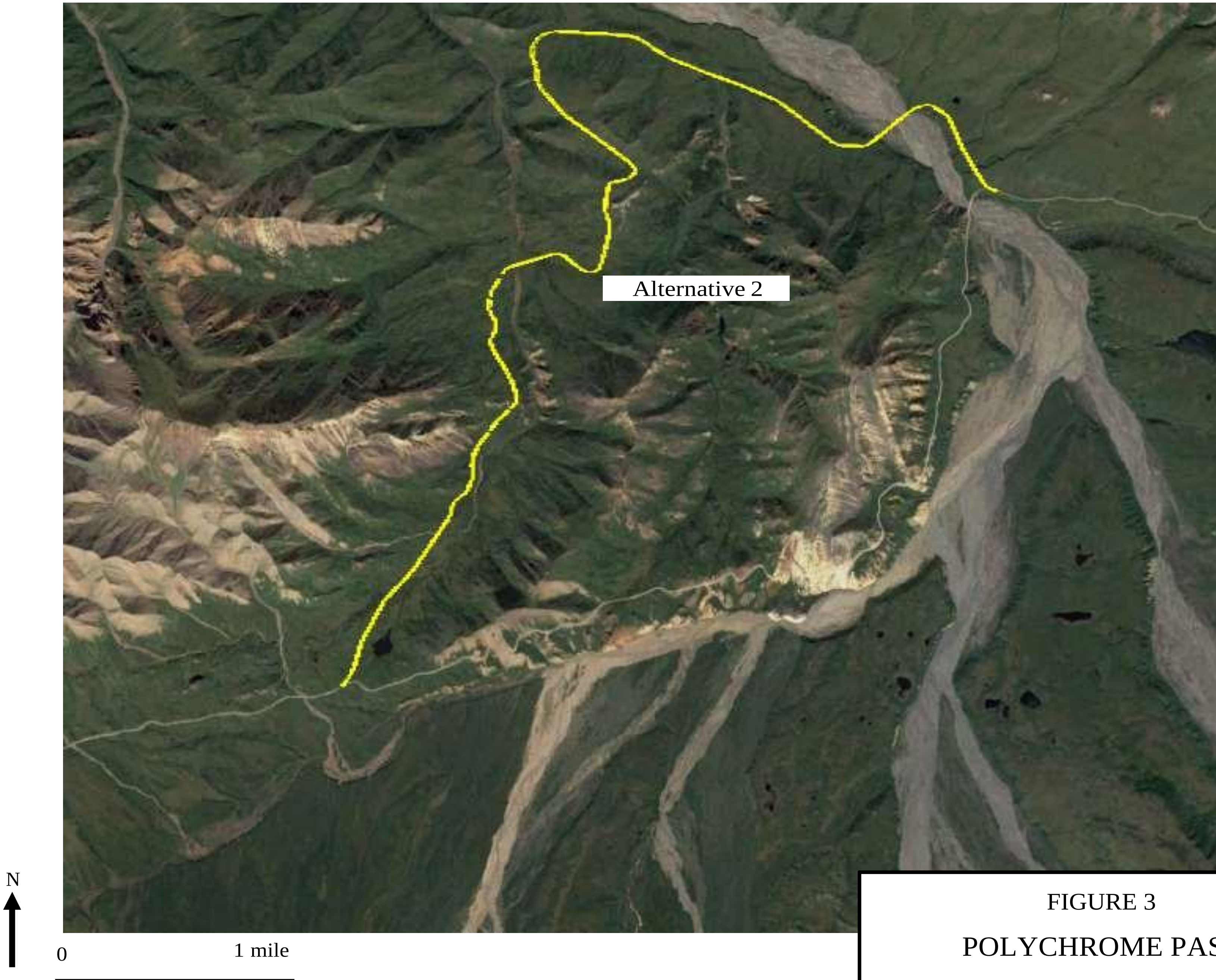


FIGURE 3

POLYCHROME PASS

Alternative 2 - North

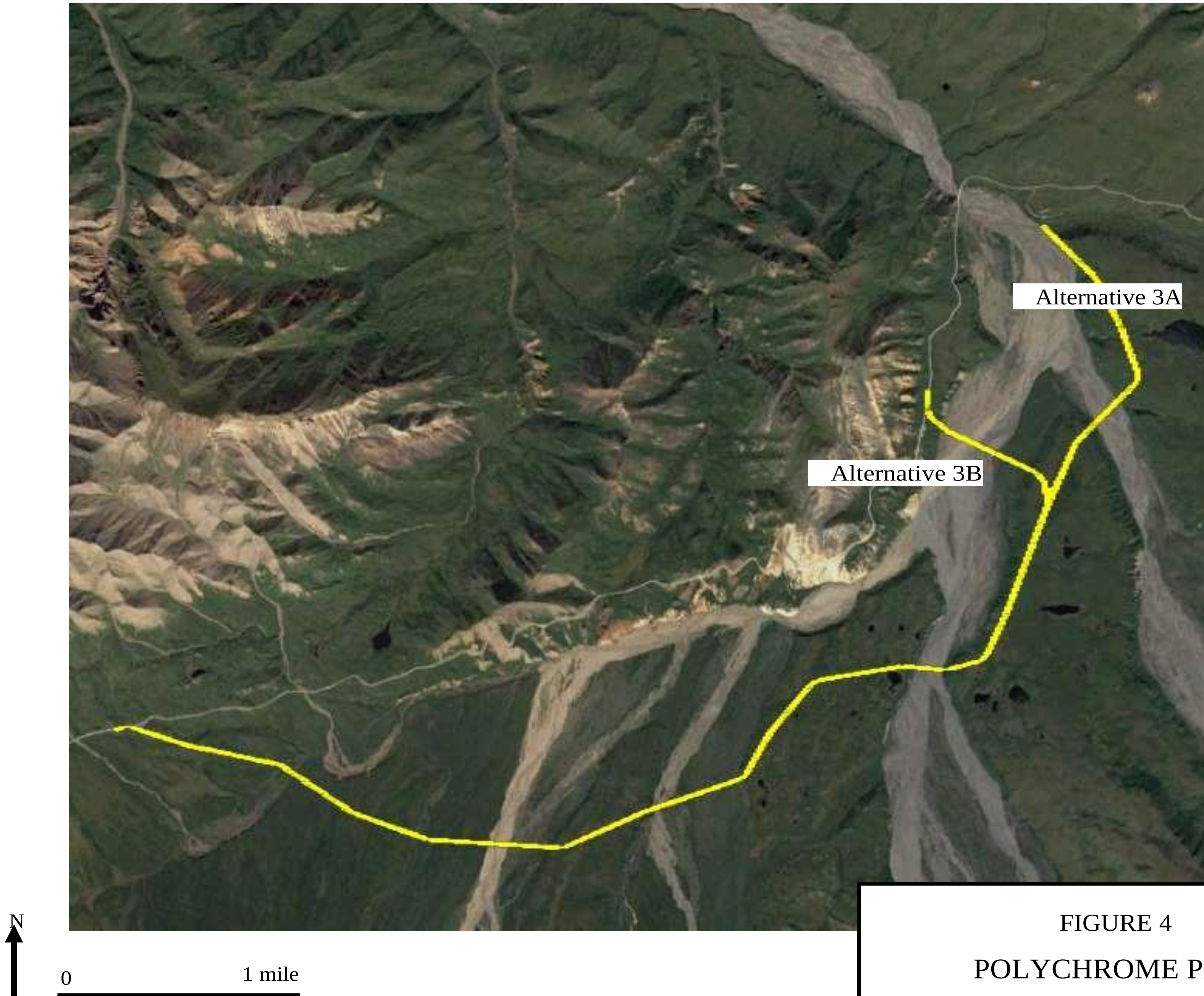


FIGURE 4

POLYCHROME PASS

Alternatives 3 - South

APPENDIX G

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Western Federal Lands Highway Division
610 E. Fifth Street
Vancouver, WA 98661

Date:	August 17, 2020
From:	Sean Kilmartin, P.E. Highway Safety Engineer
To:	Brandon Stokes, P.E. Project Manager
Subject:	AK NPS DENA 10(49) Polychrome Pass Feasibility Study – Safety and Traffic Assessment

Introduction

As part of the feasibility study for repairing or realigning Denali Park Road near Polychrome Pass (Milepost 45.4) in Denali National Park, Alaska, the Western Federal Lands Highway Division (WFLHD) Highway Safety Team has conducted an analysis of three potential options (one option, to the south of the existing Mainline, has two potential starting points) with respect to safety and operational concerns. This safety analysis uses the Interactive Highway Safety Design Model (IHSDM) software to identify locations of concern throughout the corridor. The IHSDM software considers the proposed roadway horizontal and vertical alignments, templates, cross sections, roadside design features, roadside hazards, anticipated driver behavior and other elements of design. The IHDSM software provides a prediction of roadway performance and safety over its design life based on these design elements.

Three alignment options are proposed in the Polychrome Pass Feasibility Study. Proposed work on mainline alignment (Option 1) calls for a bridge constructed outside of the Pretty Rocks landslide, a quarter mile long reconstruction of Bear Cave to fix unstable slopes and drainage issues, and the improvement of several sites along the corridor for unstable slopes. The approximation of the existing mainline alignment, roughly bounded by where Option 3A joins the existing alignment, was developed using aerial imagery and lidar data. Option 2 reroutes Denali Park Road to the north, pioneering approximately 6 miles of new roadway and Option 3A and 3B reroutes to the south. Option 3A begins before existing East Fork Toklat River bridge and 3B begins past existing East Fork Toklat River bridge. Option 3A was adjusted following the results and issuing of the EBRA report.

The overall goal for this study is to provide a high-level comparison among the project options to approximate the safety performance over the design life. For this study, the results from this high-level analysis are essentially a measure of the exposure to the traveling public for each alignment. An increase in exposure, such as increased alignment length, additional horizontal curves, steep grades, roadside hazards, etc., is correlated with an increase in crashes. While most motorists in this section of the Park will be familiar with the road and its conditions, differences in exposure will still correlate with expected crashes over time. The results of the analysis are best viewed in comparison amongst the options, rather than an absolute measure of safety for any particular option.

Crash and Traffic Data

Existing crash data was not available for this analysis. See the Safety and Traffic Assessment Appendix for discussion on how IHSDM uses geometric data to predict crashes. Traffic data used for the IHSDM analysis was taken from the title sheet for a separate project along Denali Park Road (AK NPS DENA 10(36) Replace Ghiglione Bridge).

IHSDM Description

The Interactive Highway Safety Design Model is a suite of software analysis tools for evaluating safety and operational effects of geometric design in the highway project development process. The IHSDM contains six modules that can be used to evaluate nominal and substantive safety performance. For this study, WFLHD Safety used the Crash Prediction, Policy Review, Design Consistency, Traffic Analysis and Driver/Vehicle Modules to evaluate the Polychrome Pass alignment options.

Some key assumptions and further description of the use of IHSDM and these five modules for this project are listed in the Safety and Traffic Assessment Appendix. Please refer to this section to further understand the context of the model for this project.

One key input for IHSDM is an assigned roadside hazard rating for each option, or from station to station within each option. The roadside hazard rating captures combined features such as clear zone, foreslopes, obstacles such as tree lines or utility poles, or cliff or rock cuts. For a unique location such as Denali Park Road, it also captures the geologic hazard conditions.

IHSDM Output Data Analysis

The IHDSM software divides the roadway into segments based on changes in roadway geometry, such as lane width, shoulder width, cross slope, or roadside hazard rating, as well as changes in traffic data or behavior. All four alignments were analyzed with respect to their entire length rather than specific locations within each alignment. The Traffic Analysis and Driver/Vehicle Modules helped identify errors in data input, areas of severe opposing speed differentials and higher risk regions within the corridor. The Policy Review Module helped identify geometric deficiencies for each alignment. The Driver/Vehicle and Design Consistency Modules were used to help examine expected speeds throughout the corridor. IHSDM runs a speed model through the geometry, taking into account horizontal curves and vertical grades in order to determine the effects that geometry has on speed (e.g. faster on steeper downgrades, slower on steeper upgrades). The results are sensitive to the Desired Speed input, which is estimated here in absence of formal speed data. For a design speed such as 20 mph, proposed for use on this project, an estimate of 25 mph was used considering both the steep downhill grades and the experienced shuttle bus drivers. The speed model helps the project team with locating higher discrepancies between expected speed and design speed of individual geometric elements such as horizontal and vertical curves. This can help to identify areas of elevated risk.

The primary module used to compare the alignment options was the Crash Prediction Module. Predicted Crashes are calculated for each alignment option. This is the default model for crash prediction within IHSDM and relies on roadway geometry, roadside features and other program inputs. The data from the Crash Prediction Module used to evaluate the alignment options was the number of predicted crashes for the entire corridor over a 20-year period (as will be shown on the construction plan title sheet). Additionally, the Policy Review Module, the Design Consistency Module, the Traffic Analysis Module and the Driver/Vehicle Module (for both increasing and decreasing stations) were completed for each alignment options. Results from these four modules were used subjectively, in addition to the predicted crash data, to compare all of the options.

Data Analysis

The predicted crash type distribution, over a 20-year period, for each alignment option is shown in Tables 1-4 below:

Table 1: Design Life Predicted Crash Type Distribution for the Mainline

Crash Type	Fatal and Injury		Property Damage Only		Total	
	Crashes	Crashes (%)	Crashes	Crashes (%)	Crashes	Crashes (%)
Collision with Animal	0.23	1.20	2.32	12.50	2.25	12.10
Collision with Bicycle	0.02	0.10	0.01	0.10	0.04	0.20
Other Single-vehicle Collision	0.04	0.20	0.36	2.00	0.39	2.10
Overturned	0.22	1.20	0.19	1.00	0.46	2.50
Collision with Pedestrian	0.04	0.20	0.01	0.10	0.06	0.30
Run Off Road	3.25	17.50	6.36	34.30	9.67	52.10
Total Single Vehicle Crashes	**3.80**	**20.50**	**9.26**	**49.90**	**12.86**	**69.30**
Right-Angle Collision	0.60	3.20	0.91	4.90	1.58	8.50
Head-on Collision	0.20	1.10	0.04	0.20	0.30	1.60
Other Multi-vehicle Collision	0.15	0.80	0.38	2.00	0.50	2.70
Rear-end Collision	0.98	5.30	1.54	8.30	2.63	14.20
Sideswipe	0.23	1.20	0.48	2.60	0.69	3.70
Total Multiple Vehicle Crashes	**2.17**	**11.70**	**3.34**	**18.00**	**5.70**	**30.70**
Total Crashes	**5.97**	**32.20**	**12.60**	**67.90**	**18.55**	**100.00**

Table 2: Design Life Predicted Crash Type Distribution for Option 2 (Northern Route)

Crash Type	Fatal and Injury		Property Damage Only		Total	
	Crashes	Crashes (%)	Crashes	Crashes (%)	Crashes	Crashes (%)
Collision with Animal	0.14	1.20	1.41	12.50	1.36	12.10
Collision with Bicycle	0.01	0.10	0.01	0.10	0.02	0.20
Other Single-vehicle Collision	0.03	0.20	0.22	2.00	0.24	2.10
Overturned	0.13	1.20	0.12	1.00	0.28	2.50
Collision with Pedestrian	0.03	0.20	0.01	0.10	0.03	0.30
Run Off Road	1.97	17.50	3.87	34.30	5.87	52.10
Total Single Vehicle Crashes	**2.31**	**20.50**	**5.63**	**49.90**	**7.81**	**69.30**
Right-Angle Collision	0.37	3.20	0.55	4.90	0.96	8.50
Head-on Collision	0.12	1.10	0.02	0.20	0.18	1.60
Other Multi-vehicle Collision	0.09	0.80	0.23	2.00	0.30	2.70
Rear-end Collision	0.60	5.30	0.93	8.30	1.60	14.20
Sideswipe	0.14	1.20	0.29	2.60	0.42	3.70
Total Multiple Vehicle Crashes	**1.32**	**11.70**	**2.03**	**18.00**	**3.46**	**30.70**
Total Crashes	**3.63**	**32.20**	**7.66**	**67.90**	**11.27**	**100.00**

Table 3: Design Life Predicted Crash Type Distribution for Option 3A (Southern Route)

Crash Type	Fatal and Injury		Property Damage Only		Total	
	Crashes	Crashes (%)	Crashes	Crashes (%)	Crashes	Crashes (%)
Collision with Animal	0.09	1.20	0.91	12.50	0.88	12.10
Collision with Bicycle	0.01	0.10	0.01	0.10	0.01	0.20
Other Single-vehicle Collision	0.02	0.20	0.14	2.00	0.15	2.10
Overturned	0.09	1.20	0.07	1.00	0.18	2.50
Collision with Pedestrian	0.02	0.20	0.01	0.10	0.02	0.30
Run Off Road	1.27	17.50	2.50	34.30	3.80	52.10
Total Single Vehicle Crashes	**1.49**	**20.50**	**3.64**	**49.90**	**5.05**	**69.30**
Right-Angle Collision	0.24	3.20	0.36	4.90	0.62	8.50
Head-on Collision	0.08	1.10	0.01	0.20	0.12	1.60
Other Multi-vehicle Collision	0.06	0.80	0.15	2.00	0.20	2.70
Rear-end Collision	0.39	5.30	0.60	8.30	1.03	14.20
Sideswipe	0.09	1.20	0.19	2.60	0.27	3.70
Total Multiple Vehicle Crashes	**0.85**	**11.70**	**1.31**	**18.00**	**2.24**	**30.70**
Total Crashes	**2.35**	**32.20**	**4.95**	**67.90**	**7.29**	**100.00**

Table 4: Design Life Predicted Crash Type Distribution for Option 3B (Southern Route)

Crash Type	Fatal and Injury		Property Damage Only		Total	
	Crashes	Crashes (%)	Crashes	Crashes (%)	Crashes	Crashes (%)
Collision with Animal	0.12	1.20	1.22	12.50	1.18	12.10
Collision with Bicycle	0.01	0.10	0.01	0.10	0.02	0.20
Other Single-vehicle Collision	0.02	0.20	0.19	2.00	0.20	2.10
Overturned	0.12	1.20	0.10	1.00	0.24	2.50
Collision with Pedestrian	0.02	0.20	0.01	0.10	0.03	0.30
Run Off Road	1.71	17.50	3.35	34.30	5.08	52.10
Total Single Vehicle Crashes	**2.00**	**20.50**	**4.87**	**49.90**	**6.76**	**69.30**
Right-Angle Collision	0.32	3.20	0.48	4.90	0.83	8.50
Head-on Collision	0.11	1.10	0.02	0.20	0.16	1.60
Other Multi-vehicle Collision	0.08	0.80	0.20	2.00	0.26	2.70
Rear-end Collision	0.52	5.30	0.81	8.30	1.39	14.20
Sideswipe	0.12	1.20	0.25	2.60	0.36	3.70
Total Multiple Vehicle Crashes	**1.14**	**11.70**	**1.76**	**18.00**	**3.00**	**30.70**
Total Crashes	**3.14**	**32.20**	**6.63**	**67.90**	**9.76**	**100.00**

As shown, the percentages for each crash type are nearly identical for each alignment option. This is due to assumptions made in IHSDM data input, including but not limited to side slopes, clear zones, roadside hazard rating, sight distances and cross slopes. Additionally, the number of predicted crashes for each alignment option may be somewhat overstated. First, the model uses 365-day traffic when Denali Park Road is shut down for part of the year. Second, vehicles on Denali Park Road are almost entirely shuttle bus traffic with experienced drivers who have a high level of familiarity with the road. The exception to the second point is five days per year when Denali Park Road traffic consists of the winners of the Denali Park Road lottery or veterans participating in Military Appreciation Day. Still, the IHSDM Crash Prediction Module provides value in assessing the relative risk for each alignment option.

For a relative comparison: Option 3A can be predicted to have a probability of approximately 61% fewer crashes as compared to the Mainline (Option 1). Option 3B can be predicted to have a probability of approximately 47% fewer crashes as compared to the Mainline (Option 1). Option 2 can be predicted to have a probability of approximately 39% fewer crashes as compared to the Mainline (Option 1). It seems likely that the existing alignment (Option 1, Mainline) was designed to minimize earth disturbance on the side of the mountain as much as possible. As a result, the existing alignment (Option 1, Mainline) consists of a number of low radius horizontal curves in combination with vertical crest or sag curves. Because of this, it is understandable that a new alignment could be preferable to the existing alignment from a safety perspective as it would be possible to design a new alignment to have fewer of these severe design features.

No concerns for significant speed differentials for traffic in opposite lanes were identified for any of the alignment option. The Driver/Vehicle Module could not be completed for decreasing stations for Option 3A. This seems to be due to areas with significant grade combined with curves at the western end of the alignment causing a simulated vehicle to have difficulty staying with its lane. All potential realignments had grade issues in the CADD software that would have to be mitigated if any were to be selected as the preferred option.

Conclusion

Based on the geometry data available at this time for Options 1, 2, 3A and 3B, Option 3A is preferable from a safety perspective. Options 2 and 3B are also feasible from a safety perspective. Given the safety history of the road, a rehabilitated mainline is also feasible and additional safety considerations could be given if this is the chosen alternative. If any of the realignment options are selected as the preferred option, it is recommended that this safety memorandum as well as the IHSDM model be revisited once the design has progressed and the proposed roadway geometry and roadside design are better understood. WFL Safety can develop a more detailed version of the IHSDM analysis to help recommend context-sensitive mitigation solutions in higher risk areas or in areas where it is difficult to meet design criteria.

If there are any questions on the content of this memorandum, please contact Sean Kilmartin at 360-619-7686 or sean.kilmartin@dot.gov.

Safety and Traffic Assessment Appendix

IHSDM Discussion

Crash Prediction Module

The IHSDM Crash Prediction Module estimates the frequency of crashes on a highway using geometry design and traffic characteristics. It is an implementation of the crash prediction methods documented in part C of the American Association of State Highway and Transportation Officials' (AASHTO) First Edition Highway Safety Manual (HSM)—includes capabilities to evaluate rural two-lane highways, rural multilane highways, urban/suburban arterials, freeway segments, and freeway ramps/interchanges (including ramps, collector-distributor (C-D) roads, and ramp terminals). The algorithms for estimating crash frequency combine statistical Safety Performance Functions (SPFs)—i.e., base models—and crash modification factors (CMFs). SPFs are available for roadway segments, many types of intersections, freeway ramps, C-D roads, and ramp terminals. The Crash Prediction Module was run for this project for the years of 2020 through 2040. No site-specific historical crash data was available for this analysis. 3% normal cross slopes for the length of each alignment alternative based on a provided sample typical section. No superelevation data was available for any alignment alternatives. Shoulder widths and lane widths were taken from the sample typical section. The Annual Average Daily Traffic data were developed as discussed in the 'Crash and Traffic Data' of the report. Design speed, driveway density and roadside hazard rating are other inputs for the Crash Prediction Module. Roadside hazard rating is a 1 to 7 scale for the roadside that estimates the risk of a road departure. Roadside hazard ratings were developed for the entire corridor by using aerial imagery.

Design Consistency Module

The IHSDM Design Consistency Module helps diagnose safety concerns at horizontal curves. Crashes on two-lane rural highways are over-represented at horizontal curves, and speed inconsistencies are a common contributing factor to crashes on curves. This module provides estimates of the magnitude of potential speed inconsistencies. The DCM uses a speed-profile model that estimates 85th percentile, free-flow, passenger vehicle speeds at each point along a roadway. The speed-profile model combines estimated 85th percentile speeds on curves (horizontal, vertical, and horizontal-vertical combinations), desired speeds on long tangents, acceleration and deceleration rates exiting and entering curves, and an algorithm for estimating speeds on vertical grades. Speeds entering or exiting the corridor at the western and eastern ends of the project were estimated to be 20 MPH at either end.

Policy Review Module

The Policy Review Module checks roadway-segment design elements for compliance with relevant highway geometric design policies. The module provides electronic files replicating quantitative policy values specified by the American Association of State Highway and Transportation Officials (AASHTO) in the 1990, 1994, 2001, 2004, and 2011 editions of "*A Policy on Geometric Design of Highways and Streets"* and automates checks of design values against those policy values. The Interactive Highway Safety Design Model (IHSDM) also provides a tool for inputting policy tables from other agencies' design policies. The module,

IHSDM Discussion Reference:
https://cms7.fhwa.dot.gov/research/interactive-highway-safety-design-model/modules/modules

which is applicable to rural two-lane and rural multilane highways, organizes checks into four categories: cross section, horizontal alignment, vertical alignment, and sight distance. Cross-section checks include through-traveled way width, auxiliary lane width, shoulder width and type, cross slope rollover on curves, bridge width, bike lane width, and (on rural multilane highways only) median width. Horizontal alignment checks include radius of curvature, superelevation rate, length of horizontal curve, and compound curve ratio. Vertical alignment checks include tangent grade and vertical curve length. The Policy Review Module can also check stopping, passing (on rural two-lane highways), and decision sight distance.

Traffic Analysis Module

The Traffic Analysis Module uses the TWOPAS traffic simulation model to estimate traffic quality-of-service measures for an existing or proposed design under current or projected future traffic flows. The traffic analysis module facilitates use of TWOPAS by feeding it the roadway geometry data stored by IHSDM. TWOPAS is the microscopic traffic simulation model that was previously used to develop the two-lane highway chapter of the Transportation Research Board's (TRB) "*Highway Capacity Manual*." TWOPAS produces measures including average speed and percentage of time spent following other vehicles. TWOPAS has the capability to simulate any combination of grades, curves, sight restrictions, no passing zones, and passing and climbing lanes. It is particularly useful for understanding variable traffic speeds throughout the corridor. 'Steep Grade' was selected to describe the alignment for both increasing and decreasing stations. The vehicle flow rate used was the Design Hourly Volume (Design Year ADT*0.15 – K Value selected for rural roadway).

Driver/Vehicle Module

The objective of the Driver/Vehicle Module is to permit the user to evaluate how a driver would operate a vehicle (e.g., passenger car or tractor-trailer) within the context of a roadway design and to identify whether conditions exist in a given design that could result in loss of vehicle control (e.g., skidding or rollover). The Driver/Vehicle Module consists of a Driver Performance Model linked to a Vehicle Dynamics Model. Driver performance is influenced by cues from the roadway/vehicle system (i.e., drivers modify their behavior based on feedback from the vehicle and the roadway). Vehicle performance is, in turn, affected by driver behavior/performance. The Driver Performance Model estimates a driver's speed and path along a two-lane rural highway in the absence of other traffic. The resulting estimates serve as input to the Vehicle Dynamics Model, which estimates measures including lateral acceleration, friction demand, and rolling moment. The driver type selected was 'Nominal'. The path decision selected was 'Center'. The vehicle type selected was 'Passenger Car' (the module could not be completed for any alignment alternatives when using 'Truck'). The road familiarity selected was 'Long Tangent'. The free speed used was 25 MPH, as it is assumed vehicles will travel higher than the design speed for certain stretches due to significant downhill grades.

IHSDM Discussion Reference:
https://cms7.fhwa.dot.gov/research/interactive-highway-safety-design-model/modules/modules

APPENDIX H

U.S. Department
of Transportation
**Federal Highway
Administration**

Memorandum

Western Federal Lands Highway Division
610 E. Fifth Street
Vancouver, WA 98661-3801

TO: Brandon Stokes, PM

In Reply Refer to: HFL-17

FROM: Megan Chatfield, Materials Engineer

DATE: March 19, 2020

SUBJECT: <u>INFORMATION:</u>
Preliminary Structural Pavement Recommendation
Polychrome Pass Alternatives Analysis
AK NPS DENA 10(49)

This memorandum provides the preliminary pavement structure recommendations for the Polychrome Pass Alternatives Analysis. Should new information develop that impacts the information and assumptions made as a part of this recommendation, the pavement design should be reevaluated.

The roadway is assumed to be constructed from imported materials with a resilient modulus of 16,500 psi, which will be used in the calculations to determine the aggregate design thickness. The relative quality of the roadbed soil is assumed to be "very good."

The AADT for this section of the road is approximately 60, with 80 percent of that being passenger buses Due to the low projected ESAL value, a low-volume road catalog design will be assumed.

The ESAL and specified layer design was determined from the 1993 AASHTO Pavement Design Guide, Low-Volume Road Design. The climatic region was assumed to be Region III (Wet, hard freeze, spring thaw) and the relative quality of the roadbed soil classification is "very good." Taking into consideration constructability and future maintenance impacts, a maintenance design was used to supplement the structural template for aggregate surfacing.

The following is recommended for the roadway structure:

8 inches – Roadway Aggregate, Method 2 (Section 302, estimated @ 1.97 tons/cuyd)

cc: Materials/Pavement File

APPENDIX I

U.S. Department
of Transportation
**Federal Highway
Administration**

MEMORANDUM

Western Federal Lands Highway Division
610 E. Fifth Street
Vancouver, WA 98661

DATE: April 20, 2020 In Reply Refer to: HFL-19

TO: Brandon Stokes
Project Manager

FROM: Douglas A. Anderson, Engineering Geologist
Mike Baron, Construction Operations Engineer
Tyler Yeoman, Design Engineer

- DAA for
All AUTHORS
4/20/2020

SUBJECT: **Geotechnical Memorandum 14-20**
Preliminary Pretty Rocks Landslide Earthwork Feasibility and Constructability
Pretty Rocks Landslide Investigation
AK NPS DENA 10(45)

INTRODUCTION

At the request of the Park, we have evaluated the feasibility and constructability of the earthwork option following the installation of one additional test boring (PR19-11 in September 2019), laboratory testing completed in November 2019, and measurement and acquisition of the subsurface borehole instrumentation data from late-September to early November 2019 (Figure 1). Due to safety concerns by Denali National Park, the lower test boring has not been visited to acquire subsurface data following November 6, 2019. Therefore, we have based our groundwater and subsurface ground temperature interpretations on these limited measurements for this evaluation.

ADDITIONAL TEST BORING AND INSTRUMENTATION

To evaluate the feasibility of the earthwork option, we planned two test borings (PR19-10 and PR19-11) at the base of the slope to gather additional information critical to the development of the landslide stability model beyond the area proximal to the roadway, where 2018 test borings were generally concentrated to characterize the landslide. Only test boring PR19-11 was installed before unsafe, wintry conditions shut down drilling operations in late-September 2019. This test boring helped us determine the subsurface geology, presence and temperature of ice rich soils, groundwater conditions, and depth of landslide movement.

In an effort to discuss the information collected and the importance of the additional test borings information to our understanding of the landslide, we have presented the available instrumentation information from test boring PR19-11 in Figure 2, 3, and 4. We also present our observations from the test boring and instrumentation in Table 1. This table provides the ranges of depth for similar subsurface materials encountered during drilling and provides a general material name and material descriptions with instrumentation data observations and comments. The groundwater conditions in the lower slope area is discussed following Table 1 below.

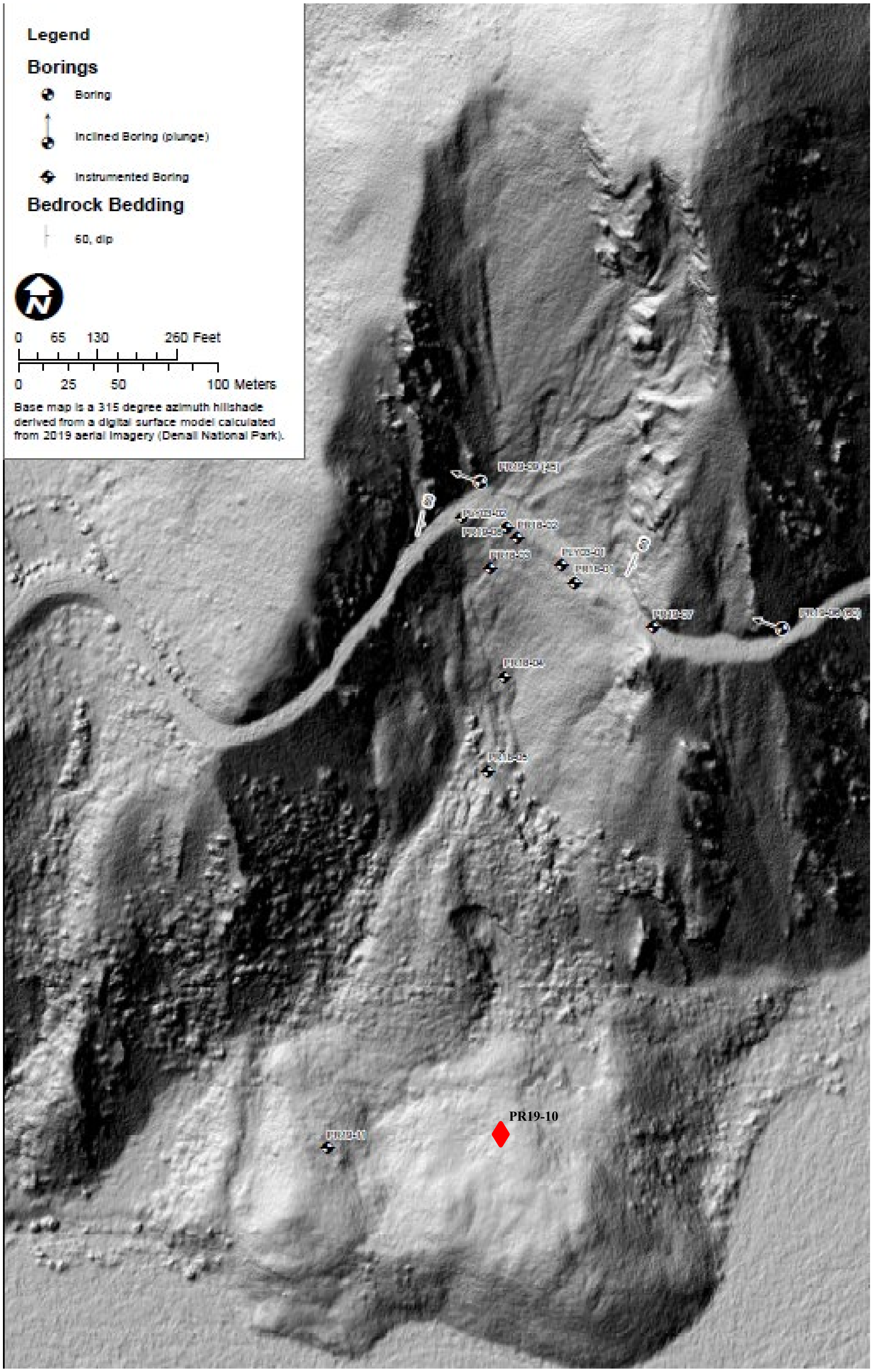


Figure 1. Test boring locations for Pretty Rocks Landslide Investigation. PR19-10 was not drilled.

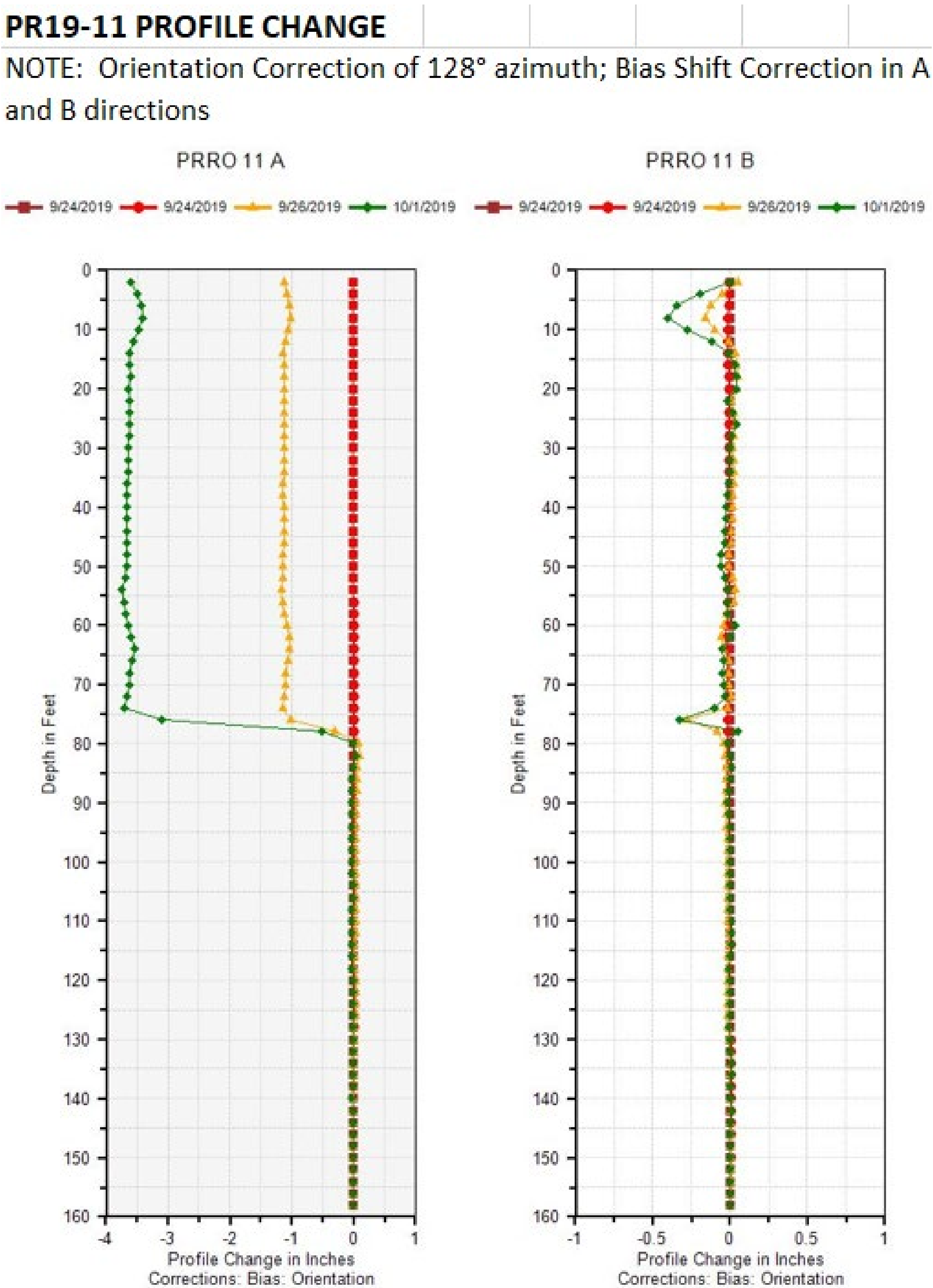


Figure 2. Slope inclinometer readings in the downslope direction (A) and from side to side (B) from September 24, 2019 to October 1, 2019. The slope inclinometer casing has sheared and is no longer usable for measurements. The rate of landslide movement indicated by these measurements is about 1.6 to 1.8 inches/day.

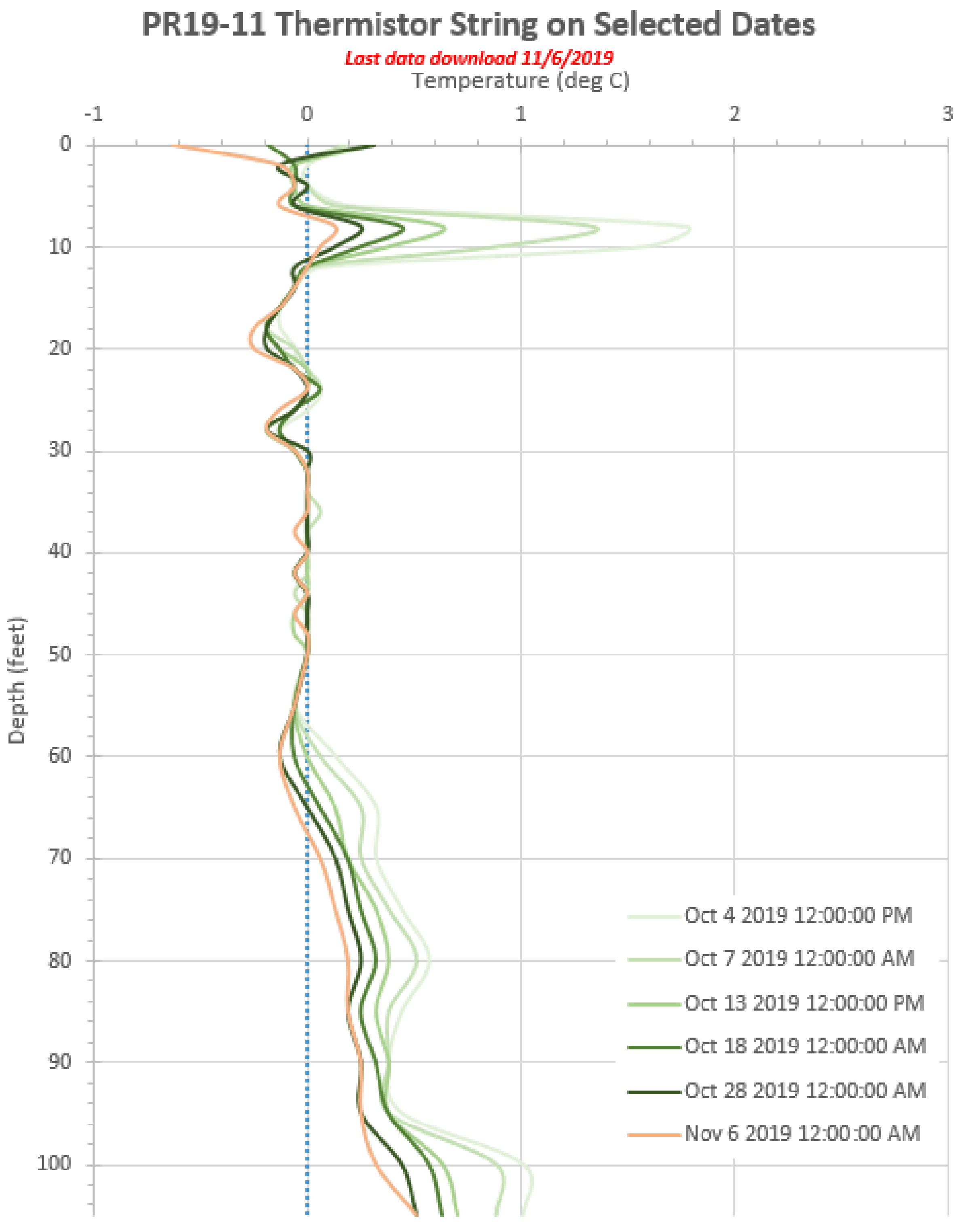


Figure 3. Thermistor (ground temperature) readings in degrees C. The temperature readings are graphed around freezing at 0 degrees C. Each colored, wavy line represents a snapshot in time. The trend indicates the temperature during drilling temporarily increased the ground temperature but it fairly quickly adjusts back to its temperature after the drilling and instrumentation has been installed.

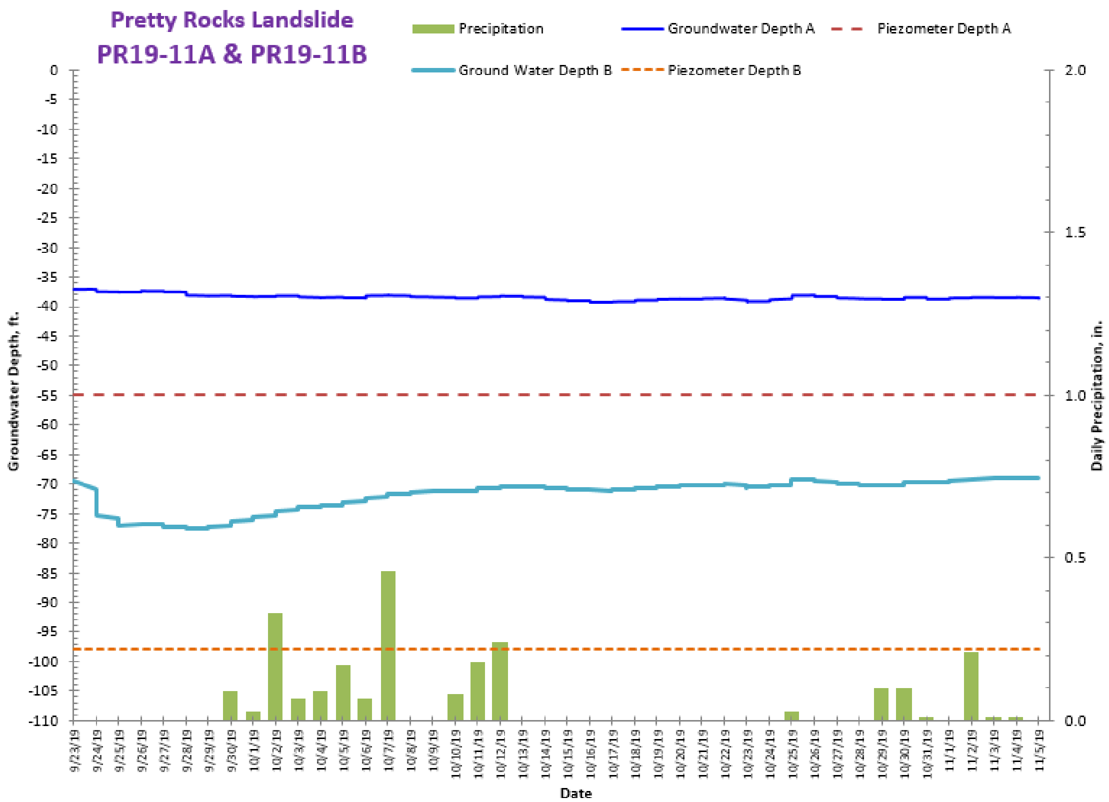


Figure 4. Groundwater data collected on top of a thick clay layer at 55 feet (A) and atop very weak bedrock at 98 feet (B), as observed in test boring PR19-11. Daily precipitation is shown at the base of the graph in green columns to compare precipitations impacts on groundwater measured at the base of the landslide.

Table 1. Test Boring Material Descriptions and Instrumentation Observations

Approximate Depth (ft)*	Material Name	Description with Observations
0 to 34	Landslide Deposit	Frozen, Elastic Silt with Silty Sand and angular, Sandy Gravel with thick cobble layers
34 to 44.5	Landslide Deposit	Hard Ice
44.5 to 55	Landslide Deposit	Frozen, Disrupted, Silty Sand with gravel; Figure 2 displays evidence of potential upper failure surface in landslide near 55 ft.
55 to 62.3	Landslide Deposit	Very stiff, Fat Clay; Figure 3 thermistor indicates below freezing temperatures.
62.3 to 80	Landslide Deposit	Angular, Silty Sand with gravel, Sandy Gravel with clay, Silty Gravel with sand and disrupted Silty Sand from 75 to 80 feet; Figure 3 thermistor data indicates transition from below to above freezing near 68 feet. Figure 2 shows basal landslide failure plane between 76 and 80 feet.
80 to 85	Landslide Deposit	Very stiff, Fat Clay; Figure 3 indicates above freezing temperatures.
85 to 100	Alluvial Deposit	Subrounded to rounded, Silty Sand with gravel to Silty Gravel with sand; this is likely the old flood plain surface before landsliding began; Figure 3 indicates above freezing temperatures.
100 to 157.1	Teklanika Formation	Extremely weak rhyolitic tuff to rhyolitic rock in a highly altered to completely weathered state to depth.

*Depth is measured below the ground surface.

In addition to the information presented in Table 1, the groundwater data, although limited, has been critical in developing an updated geologic model for stability analyses. At the base of the slope in test boring PR19-11, groundwater measurements in Figure 4 indicate the presence of two independent groundwater tables. The upper groundwater measuring device (vibrating wire piezometer (vwp)) is located at 55 feet below the ground surface on top of an impermeable fat clay layer that is thought to be laterally continuous. The upper groundwater table (A) shows consistent groundwater elevations between 37 and 38 feet below the ground surface, and it is assumed that it is under pressure and confined based on the frozen landslide debris and hard ice that lies above this fat clay layer. The lower vwp is located at 98 feet, just above the extremely weak and altered rhyolitic rock materials. This lower groundwater table (B) is less consistent and shows some variation between 69 and 77 feet below the ground surface. It is assumed that it is under pressure and confined by the overlying, lower fat clay unit in the landslide debris from 80 to 85 feet below the ground surface. The variation in the lower groundwater table may be more closely tied to the river fluctuations on the East Fork of the Toklat River, where a river gauge is scheduled for installation at the downstream East Fork Bridge crossing this summer.

This groundwater regime at the base of the slope was not observed in previous test borings and instrumentation that was installed closer to the roadway, in the upper portion of the landslide.

PRELIMINARY STABILITY ANALYSES

Stability analyses for this landslide is complex and further complicated by the presence of appreciable permafrost and ice rich soils that may be driving the movement of the landslide through dynamic fluid mechanic processes studied in glaciers, different from traditional landslide slope stability modeling using limit equilibrium methods.

The Pretty Rocks Landslide site closely matches the recent studies of rock glaciers and frozen debris lobes being studied in cold climate areas that are experiencing warming climatic conditions, similar to Denali National Park. Figure 5 illustrates a rock glacier formation that closely matches the situation at the Pretty Rocks Landslide (Mueller, et.al, 2016), and coincidentally may explain some of the difficulties we are having with limit equilibrium modeling. Limit equilibrium modeling has different input parameters than a dynamic fluid mechanic problem for glaciers. We are unaware of research into frozen debris lobes and rock glacier modeling that is appropriate to apply to this site. For these reasons, the limit equilibrium methods being utilized for the stability analyses of this slope should be considered relative, not absolute, providing trends of improvement or worsening of the slopes stability.

As discussed above, installation and monitoring of instrumentation in test boring PR19-11 and conducting additional laboratory testing has updated our understanding of the landslide geometry, extent of movement down slope of the road, and the groundwater regime. The stability model from the 2018 preliminary stability analysis was updated with this new information in the 2019-2020 back-analysis using Rocscience SLIDE 2018 (Version 8.008)(Figure 6). The 2019-2020 back-analyzed factor of safety (FOS) of 0.56 to 0.60 is consistent with landslide horizontal movement rates of about 14 inches per week, as published by Cornforth and Vessely in 1992. Figure 7 shows the upper landslide material removed from the slope and the new road placed on the very weak and altered bedrock materials. The landslide

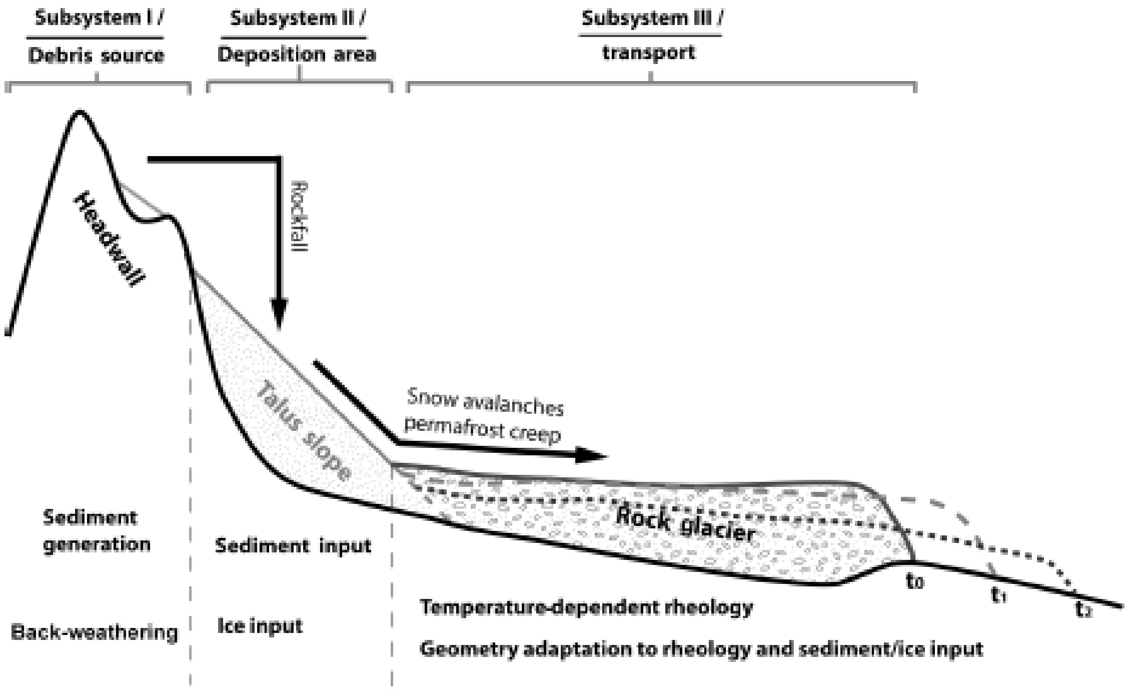


Figure 5 Conceptual model of the dynamic evolution of a rock glacier system (adapted from Fig. 1 in Müller et al., 2014a). Black arrows show the sediment transport. t_0, t_1 and t_2 show the rock glacier surface geometries at different time steps resulting from variations in environmental factors such as warming and a decrease of sediment–ice input.

material was removed until near equilibrium at a FOS of 1 to indicate the approximate volume of material that could be moved from the upper landslide to the road elevation area and sidecast before the slope becomes unstable. Based on the slope stability model shown in Figure 7, the horizontal distance of excavated material that can be sidecast at the road elevation is approximately 50 feet. This is important to understanding when discussing the ability to simply sidecast the upper landslide material over the side of the road for disposal. *The modeling suggests that if more than about 50 horizontal feet of material is sidecast at the road, the landslide will become increasingly unstable and could possibly fail rapidly, putting construction workers at risk if they are present, and pushing excavated material over the edge. For this reason, we recommend that the construction staging of excavated material at the road elevation be minimized, and excavated material be moved to the base of the landslide to add resisting (force) weight to the landslide, like a counterbalance.* The slope stability modeling suggests that up to a 75 foot thick, uniform layer of excavated material can be placed on top of the landslide as shown in Figure 8. This will minimize the likelihood of rapid instability during construction and provide safer access for the construction workers below the road elevation. Figure 9 indicates that the toe of the landslide, after being loaded by excavated material, should be globally,

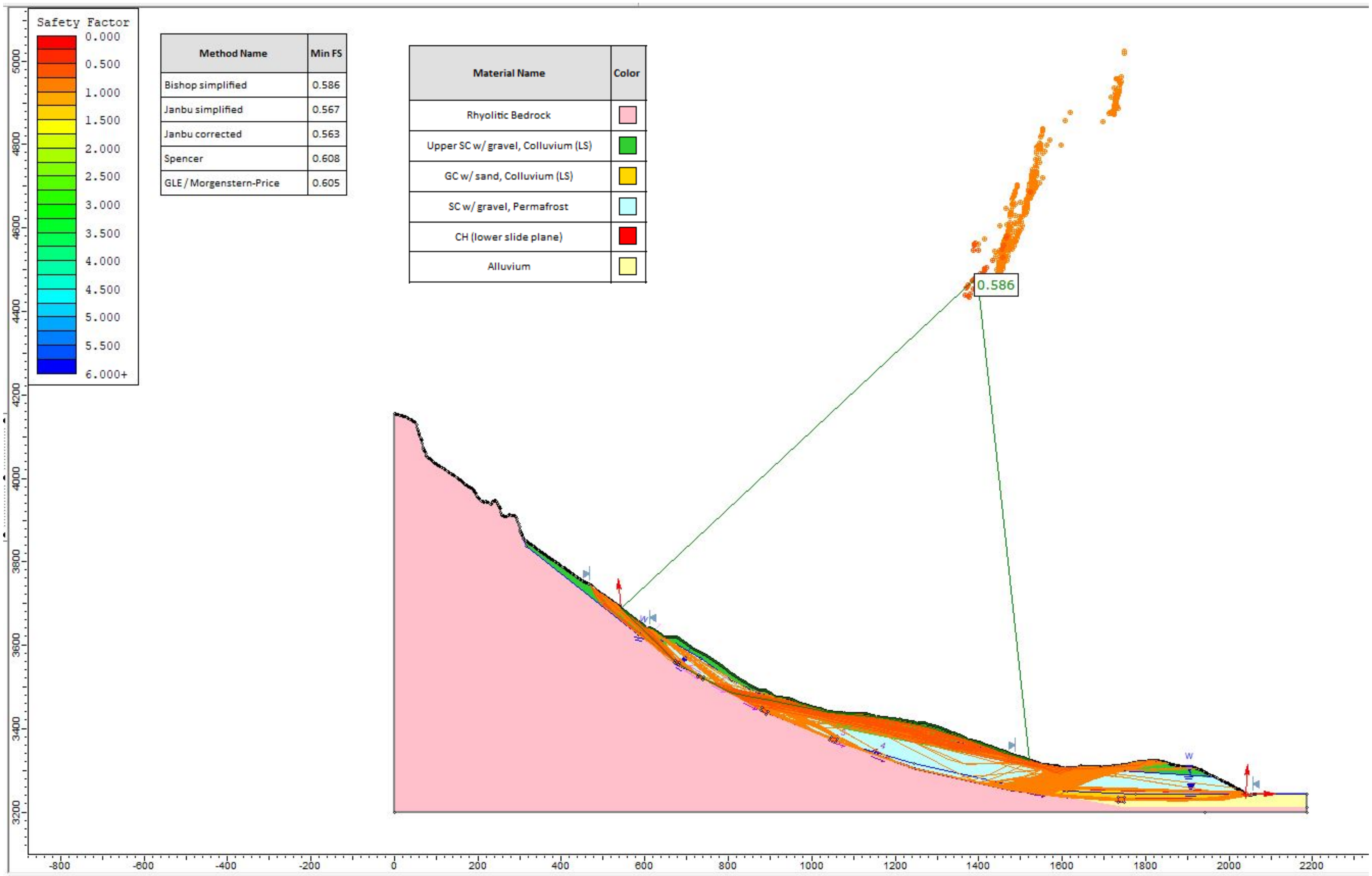
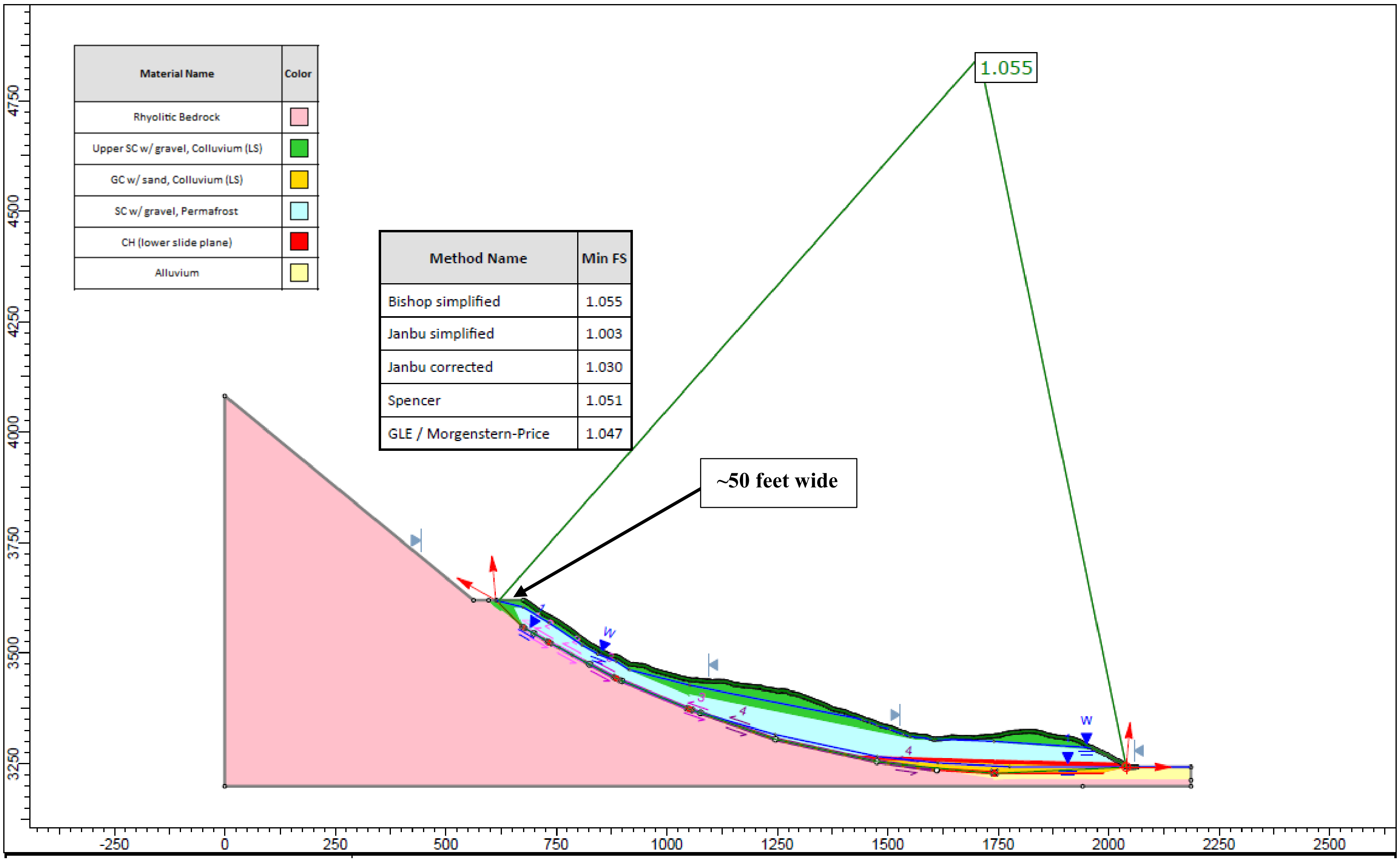


Figure 6. Revised 2019-2020 slope stability back analyses model following additional information from test boring PR19-11 information.



Method Name	Min FS
Bishop simplified	1.055
Janbu simplified	1.003
Janbu corrected	1.030
Spencer	1.051
GLE / Morgenstern-Price	1.047

Figure 7. Slope stability analysis of the upper landslide material removed with the road shifted, and founded upon the very weak and altered bedrock. No material has been sidecast in this stability model.

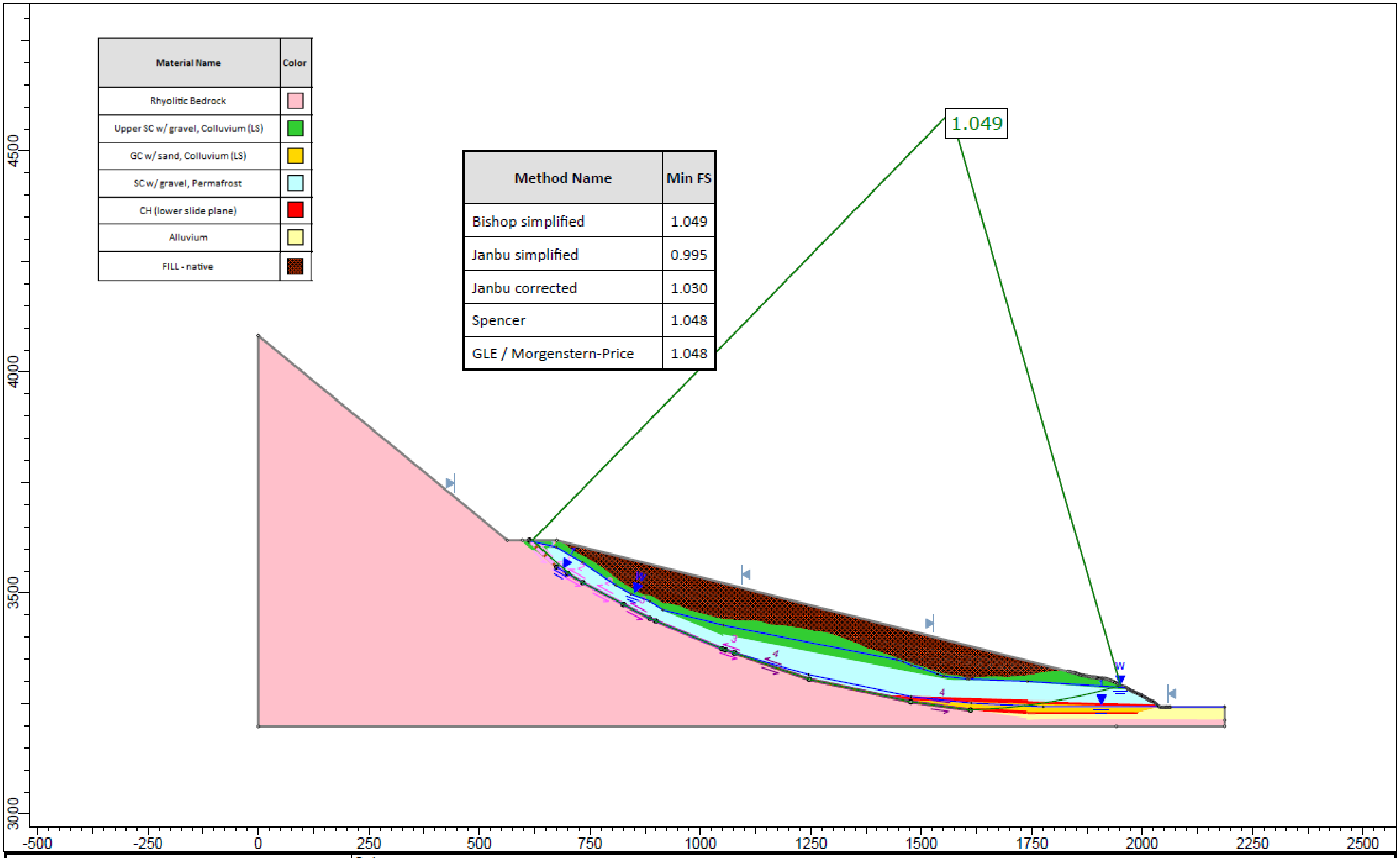


Figure 8. Slope stability analysis of the upper landslide material removed and placed below the road (brown) in a uniform (load) slope that provides some excavated material storage on the landslide, while attempting to minimize the instability during construction and maintain access to the lower landslide for work.

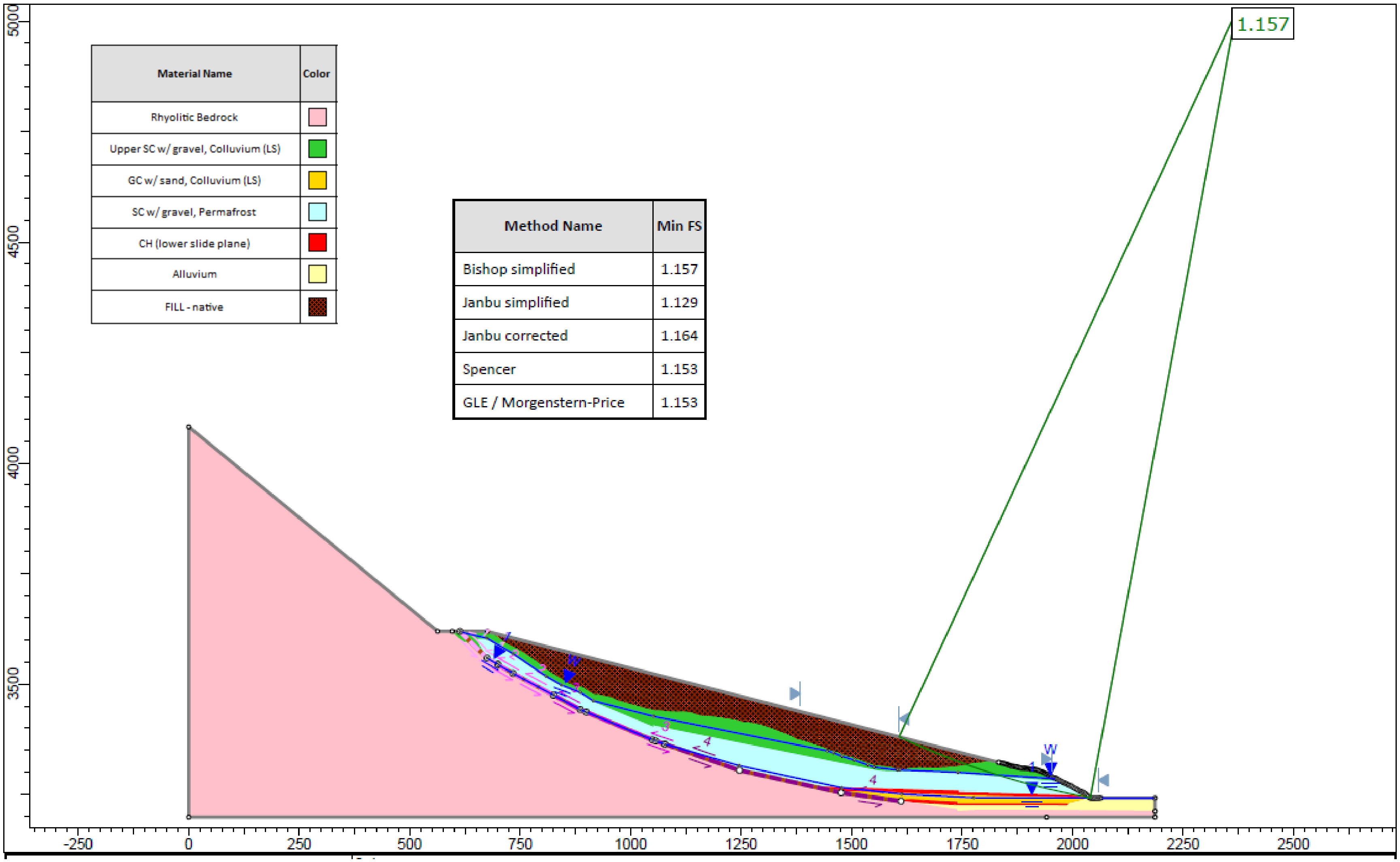


Figure 9. Slope stability analysis of the landslide toe area to determine if the load of the excavated material at the base of the slope will destabilize the lower landslide. The modeling suggests that it is globally, marginally stable.

marginally stable, but will certainly be subject to erosion and shallow failures on the surface.

It should be noted that the very weak, altered bedrock beneath the landslide material is sensitive to moisture, highly erodible and subject to strength loss, or weakening when exposed to surface elements. With this in mind, we should anticipate that the cut slope will differentially weather in the differing geologic units, causing heavy erosion and shallow failures. Based on our limited experience with this material and annual observations during Spring Road Opening assistance visits, we estimate about 10 to 15 years for these very weak volcanic materials to potentially become a maintenance nuisance.

EARTHWORK OPTIONS AND CONSTRUCTION CONSIDERATIONS

The earthwork option generally proposes removing the upper landslide and placing the excavated material at the base of the slope. The roadway will be shifted into the hillside onto the freshly exposed, very weak and altered volcanic rock with a 24-foot wide roadway section. The proposed cut slope in the limits of the landslide is 1V:1.5H (35 degrees), and is anticipated to be within the very weak and altered volcanic rock. The cut slopes proposed to the west and east of the landslide are 1V:1H (45 degrees) in the basaltic rock to the west and the rhyolite rock to the east. Blasting will likely be required for the excavation of the rock materials to the west and east of the landslide. *In total, approximately 1.1 million cubic yards (CY) of excavation is anticipated* (Figure 10 and 11). As noted above, erosion and shallow failures of the newly exposed cut slope is anticipated, so erosion control techniques and methods should be considered.

Removal of the upper landslide material and sidecasting it near the roadway elevation within the landslide is not advisable for the slope stability and safety reasons provided in the Preliminary Stability Analysis section above. However, strategically placing the excavated, upper landslide material below the road elevation within the limits of the landslide is feasible (Figure 12). Material must be uniformly placed along the lower slope area up to 75 feet thick from the base of the landslide to the road elevation. Staging of excavated material at the road elevation, at the top of the remaining landslide material, should be minimized to reduce the risk of instability. *Approximately 300,000 CY of material can be strategically placed and wasted within the lower portion of the landslide.*

Assuming about a 20% swell of the excavated material being hauled and placed at the base of the slope, the proposed waste area will need to be roughly 1.3 million CY in size. An additional waste area downstream and to the east of the landslide on the valley floor has been identified in Figure 10. Figure 13 provides a cross-section for visualizing the anticipated depths of waste materials outside the landslide limits on the valley floor. It is important to note that outside the landslide limits on the existing roadway to the east, excavated material can be sidecast to expedite wasting of the excavated material, and placement in the area downstream, and immediately east of the landslide, as shown in Figure 10 and 13. Rock excavation in the basalts on the west can be sidecast if being controlled and moved into the lower landslide area as described previously. *The additional waste site area has been sized to accommodate about 1 million CY of excavated waste, and it can be refined by shaping and contouring to match the landscape better than depicted in Figure 9 if this option is selected to move forward in design.*

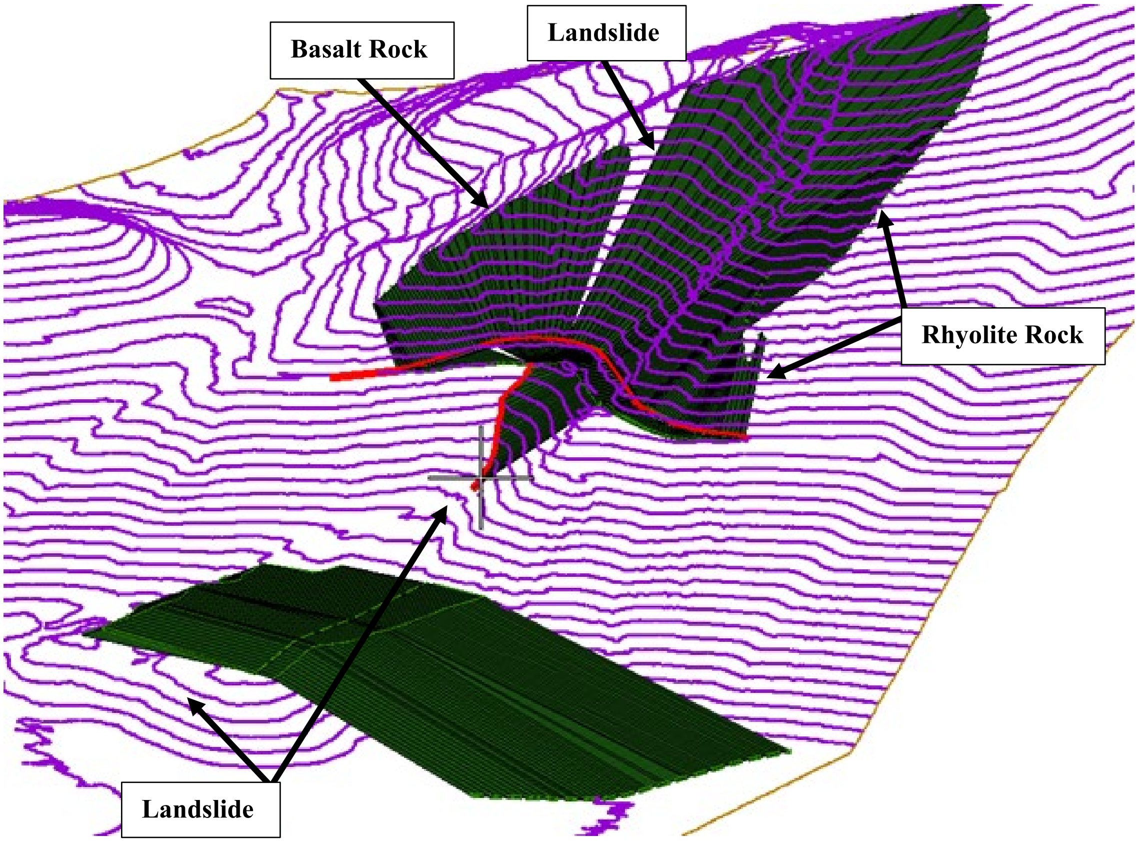


Figure 10. Plan view map showing the anticipated excavation area in the upper landslide area, shifting the roadway into the hillside, and the footprint of waste material at the base of the landslide, and to the east on the valley floor.

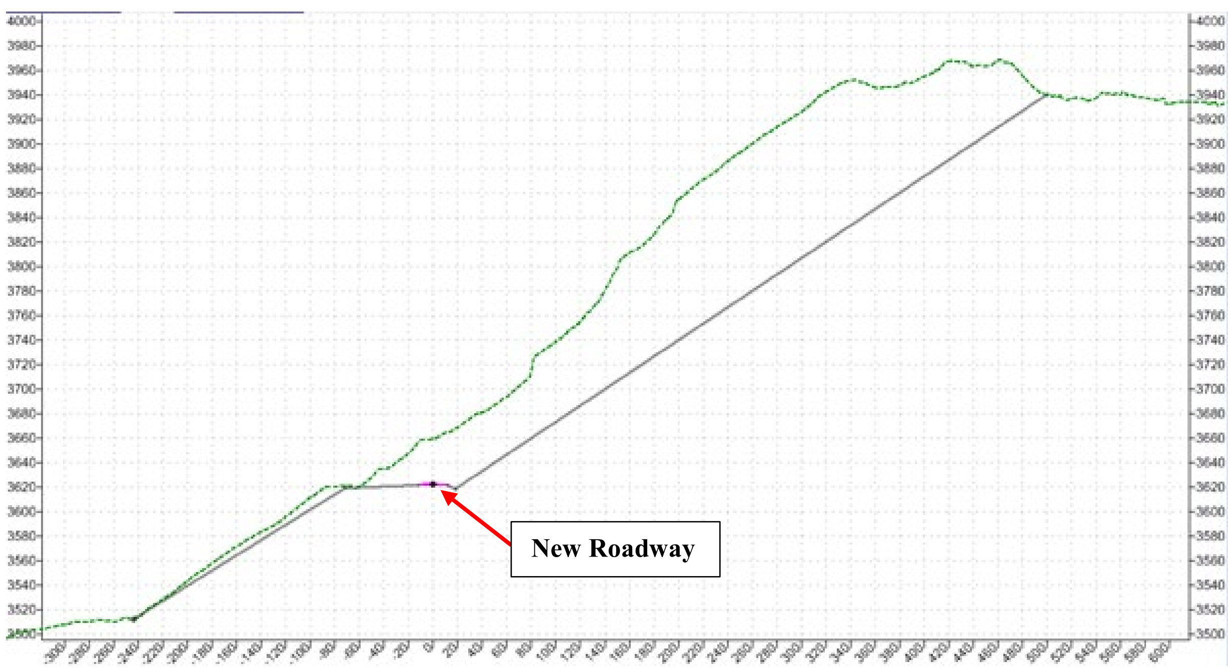


Figure 11. Cross section in the middle of the upper landslide showing proposed excavation and road shift into the hillside. Existing ground is represented by the green dashed line.

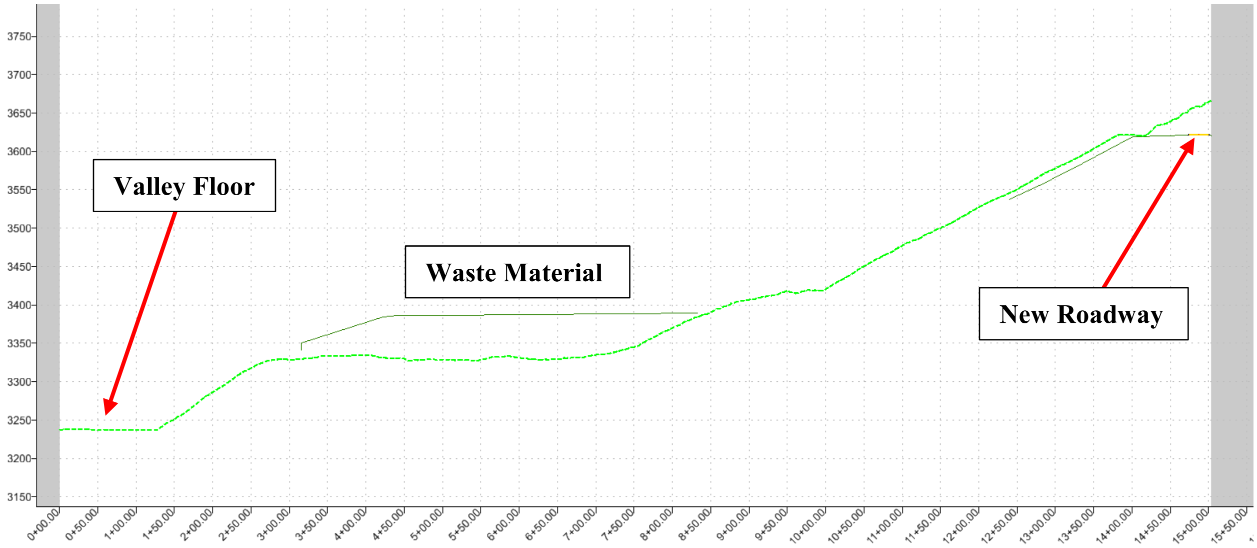


Figure 12. Cross section in the middle of the lower landslide showing proposed waste material that can be placed on the lower landslide. Existing ground is represented by the green dashed line.

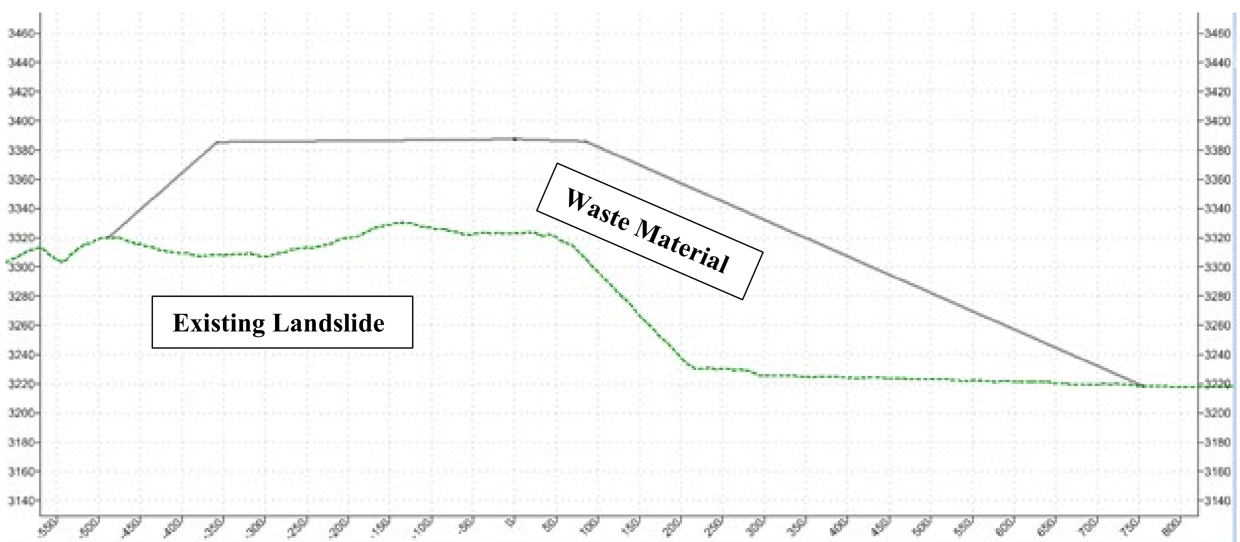


Figure 13. Cross section through the middle of the waste area, oriented from west to east. Existing ground is represented by the green dashed line.

Although the rock and soil excavation operations are relatively simple in principal, there are a number of challenges associated with the work that will have a significant impact on anticipated production rates. The existing soil slope within the limits of the slide is steep, dozers will have to work from the top down and cannot traverse the existing slope. Further complicating the excavation, is the limitation on how much material can be placed below the road on the active landslide (Figure 8). This requires completing the excavation in stages to allow the material below the road to be placed along, and adjacent to, the toe of the landslide as the work progresses to avoid further destabilizing the slope. In addition, the soil within the limits of the landslide contains weak, ice rich soils. The high water content when frozen will require more time to excavate when frozen (or allowed time to thaw) and may not support equipment as the ice melts and saturates the silts and clays, and will likely add moisture to already present moisture sensitive soils and further complicate construction activities and localized instabilities. The extreme weather in Denali will limit construction to the months of March through October. Weather during the months of March and October are marginal, so it's not uncommon to encounter temperatures or snow fall that would stop production work. *With these challenges in mind, it's anticipated the work will require three seasons to complete. This duration assumes all material can be disposed of on-site.*

TRAFFIC IMPACTS

Based on the feasible and constructible earthwork option provided, and the need to consistently move excavated material from the upper landslide area to waste it in the lower landslide area, public access through the site cannot be accommodated until the landslide material above the road has been removed and the new roadway template has been established. The primary issue for allowing public access is the slope above the road will constantly be a source of shallow landslides and rockfall during the construction activities until the upper slope is excavated and the new roadway is established. *For these reasons, we believe the earthwork option will require a full roadway closure during the landslide removal operations and it may be possible to stage road openings-until the rock excavation on both sides of the landslide are completed.*

CONCLUSION

If this earthwork option is selected to move forward, the following would be required for careful consideration in preparing the contract and contractual risks associated with the earthwork activities described above:

- A drainage plan to shed water from the upper landslide through the existing roadway elevation will be required to efficiently move ice rich soils and permafrost melt-water from the active excavation in the landslide section.
- A staged, delay for the upper landslide excavation once permafrost is exposed to allow for melting, similar to the original Park Road building descriptions and conditions provided by the Alaska Road Commission, will likely be required (Bryant, 2011). It will likely be too dangerous to work on the solid ice on a steep slope with equipment. Other work can continue outside the landslide limits during this delay of removing the upper landslide material.
- Melting of the permafrost and ice rich soils in the upper landslide may create delays as the soils de-water so they can support heavy equipment to continue excavation removal operations.

LIMITATIONS

This memorandum has been prepared to assist the National Park Service in evaluating the feasibility of the earthwork option for the Pretty Rocks Landslide. It should not be used, in part or in whole for other purposes without contacting the Western Federal Lands Highway Division

(WFL) for a review of the applicability of such reuse. These data are not to be used for other purposes.

The conclusions and recommendations contained in this report are based on WFL's understanding of the project at the time that the memorandum was written and onsite conditions that existed at time of the field observations and subsurface exploration. If significant changes to the nature, configuration, or scope of the project occur, WFL should be consulted to determine the impact of such changes on the preliminary Pretty Rocks Landslide bridge option feasibility and constructability analyses and conclusions presented in this memorandum.

REFERENCES

Bryant, J. 2011. Snapshots from the Past – A roadside history of Denali National Park and Preserve. US Department of Interior, Denali Nationl Park Preserve, Center for Resources, Science, & Learning, pp 204.

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Muller, J., Vieli, A., and Gartner-Roer, I. 2016. *Rock glaciers on the run – understanding rock glacier landform evolution and recent changes from numerical flow modeling.* Dept of Geography, University of Zurich, Switzerland. Published in The Cryosphere, 10, p 2865-2886.

CLOSING

If you have any questions or concerns regarding the information contained in this memorandum, please contact Brandon Stokes at 360-619-7813.

CC: Michael Madar, Highway Design Manager
Eric Lim, Acting Geotechnical Functional Manager
Orion George, Engineering Geologist
Geotechnical File