U.S. Department of Transportation
Federal Highway Administration

Technical Memorandum

Date: May 28, 2026

To: Justin Henwood, P.E., Project Manager/COE, CFLHD, Lakewood, Colorado

From: Aaron Baukus, Geotechnical Engineer, CFLHD, Lakewood, Colorado

Through: Devin Dixon, P.E., Geotechnical Team Lead, CFLHD, Lakewood, Colorado

Subject: CO ERFO FS SNJUAN 2026-1(1) Geotechnical Recommendations

1.0 PROJECT BACKGROUND INFORMATION

This geotechnical memorandum presents the field evaluation results, analysis, and recommendations for the reconstruction of five sites along various roadways within the San Juan National Forest, Colorado. The sites were damaged during heavy rainfall and flooding in the Pagosa Springs area from October 10-14th, 2025. Repairs are planned as part of the Emergency Relief for Federally Owned Roads (ERFO) program.

The Fall Creek Road project sites are in Mineral and Archuleta Counties. All sites are located within a 20 mile radius of Pagosa Springs and lie along various Forest Service Roads that connect US Route 160 to recreation areas and private cabins. Damage assessments confirm that erosion due to flooding caused roadway and structure damage.

A project vicinity map is presented in Figure 1 and a summary of the damaged sites is presented in Table 1. This summary includes the site name, location, damage description, roadway width, surfacing, ADT (Average Daily Traffic), and accessibility status. DDIRs (Detailed Damage Inspection Report) for these sites were prepared by the Forest Service and Central Federal Lands Highway Division (CFLHD) prior to initiating design.

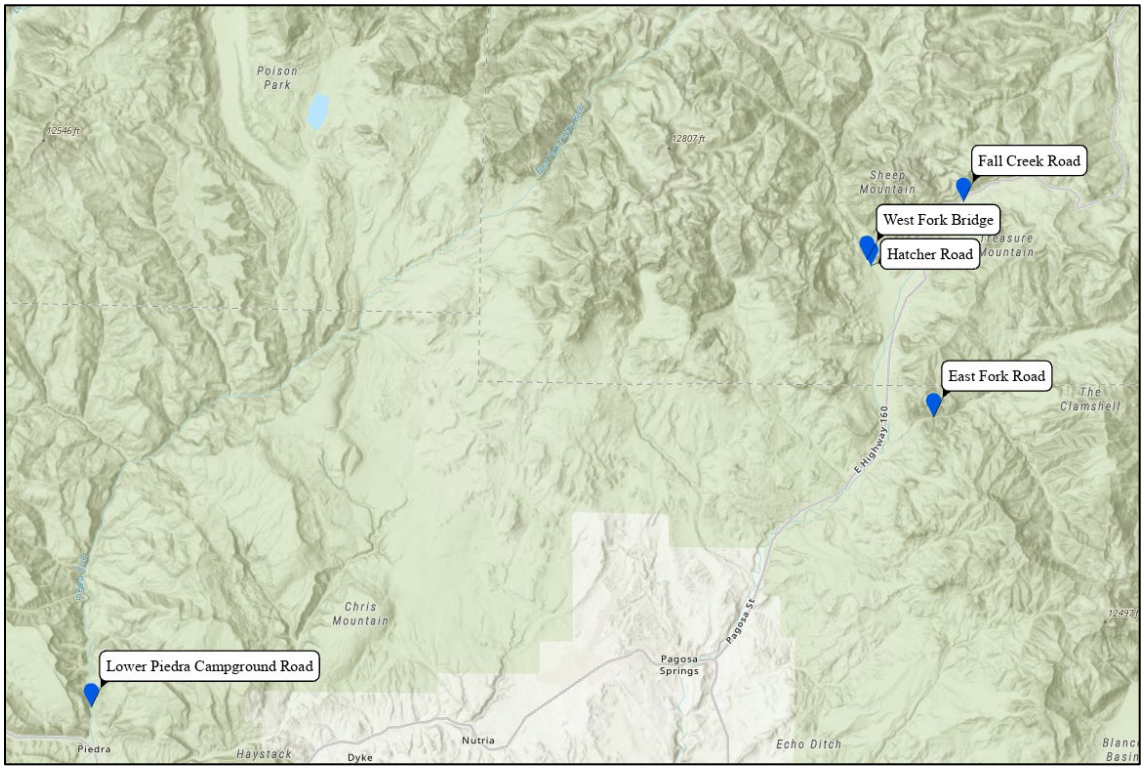


Figure 1: Project Vicinity.

Table 1: Summary of Damaged Sites.

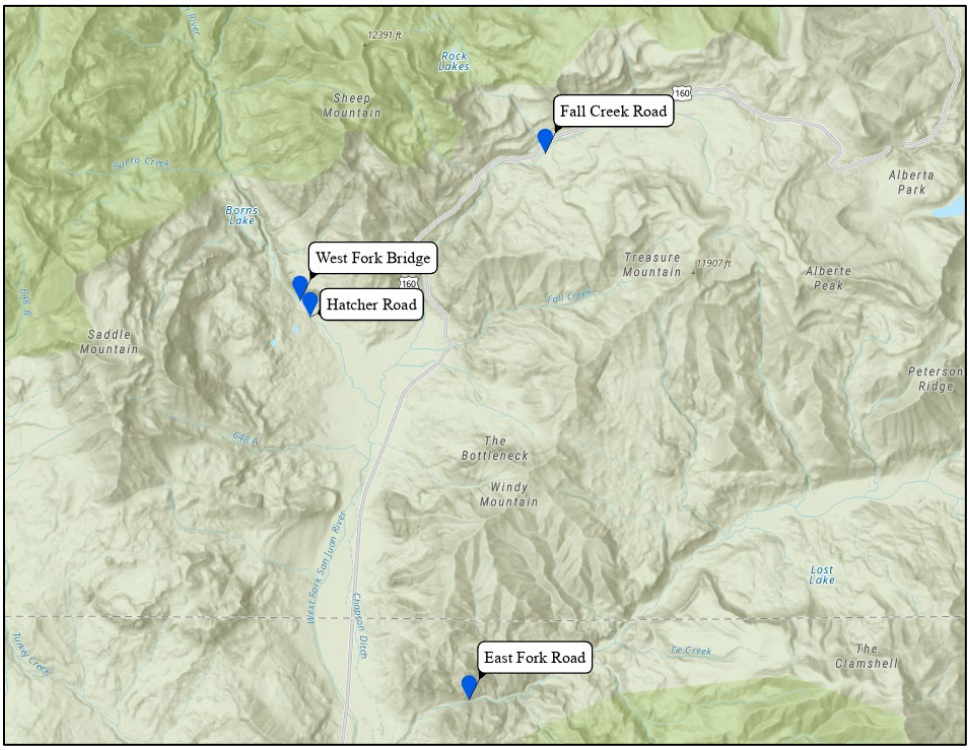
Site	Latitude (deg)	Longitude (deg)	Damage Description	Roadway Width (ft)	Surfacing[1]	ADT[1]	Accessibility
033 Hatcher Road	37.44704399	-106.908865	Erosion induced embankment failure resulting in loss of 128 ft of roadway.	14	Native Material	Low Volume	No Access
667 East Fork Road	37.379706	-106.872357	Erosion induced embankment failure resulting in partial road collapse. Temporary repair performed by Xcel Energy.	18	Gravel	Low Volume	Limited Access
039 Fall Creek Road	37.45863472	-106.8588993	Culvert failure resulting in loss of roadway.	18	Aggregate	Low Volume	No Access
648 West Fork Bridge	37.4498965	-106.9111342	Severe scour resulting from high flow event. Emergency repair performed by Forest Service.	14	Monolithic Concrete	Low Volume	Limited Access
621 Lower Piedra Road	37.24306696	-107.342937	Embankment erosion resulting in partial loss of roadway.	14	Gravel	Low Volume	Limited Access

(1) As described in DDIR.

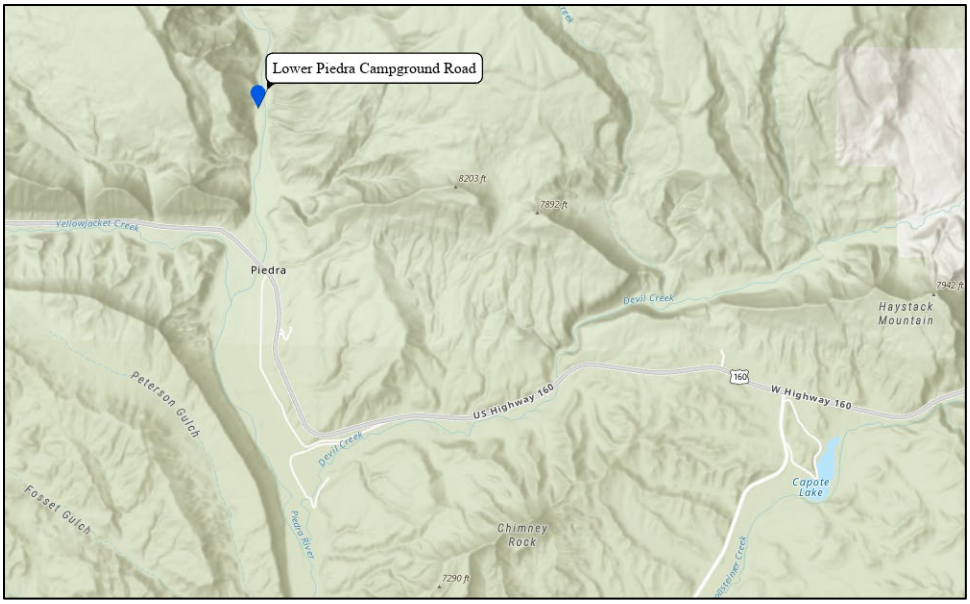
Proposed repairs generally consist of embankment reconstruction, culvert replacement, and bridge abutment repair. Embankment reconstruction and stabilization are proposed to restore the lost roadway width at Hatcher Road, East Fork Road, and Lower Piedra Road. A concrete box culvert (CBC) is proposed to replace the failed corrugated metal pipes (CMPs) at Fall Creek Road. Scour repairs and armoring are proposed to repair the west abutment of West Fork Bridge.

2.0 SITE CHARACTERIZATION

Magnified topographic maps of the damage sites are presented in Figure 2 below. These sites all sit along mountainous tributaries, four of which flow into the San Juan River (Figure 2 (A)), and one into the Piedra River (Figure 2 (B)). The sites at Hatcher Road and West Fork Bridge are located along the West Fork San Juan River, the East Fork Road site is adjacent to the East Fork San Juan River, and the Fall Creek Road site lies along Wolf Creek. Lastly, the Lower Piedra Road site is located adjacent to the Piedra River.



(A)



(B)

Figure 2: Location of Damage Sites.

2.1 Historical Climate Summary

Figure 3 presents the daily precipitation recorded at 31 NOAA weather stations within 20 miles of Pagosa Springs, CO. The daily precipitation was recorded over a three-month period spanning from September 1, 2025, to November 30th, 2025. The rainfall accumulation corresponding to the flooding from October 10th – 14th, 2025 is highlighted in red. The precipitation peaked on October 14th where areas within the project vicinity received 2.8 inches of rainfall in one day (NOAA, 2026).

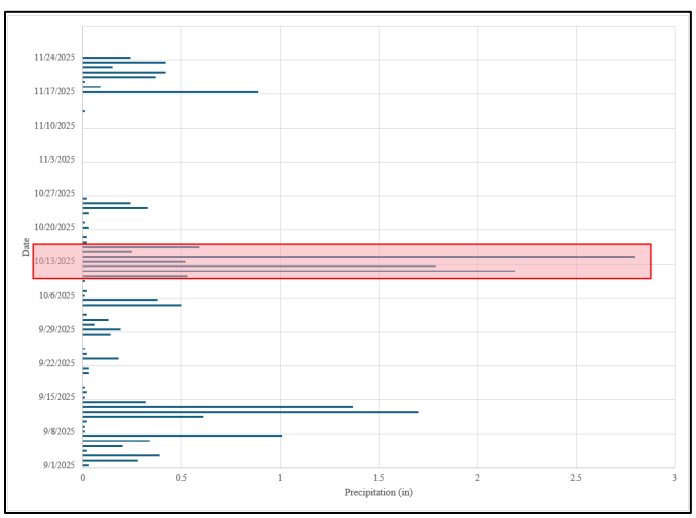


Figure 3: Daily Precipitation at Pagosa Springs Weather Stations (9/1/2025 – 11/30/2025) (NOAA, 2026).

Figure 4 displays USGS hydrograph data recorded along the San Juan River at Pagosa Springs. The graph captures the water levels over a three-month period spanning from September 1, 2025, to November 30th, 2025. The heavy rainfall on October 14th produced peak flows that elevated water levels to 12.77 feet, placing the river in a major flood stage (U.S. Geological Survey (1), 2026).

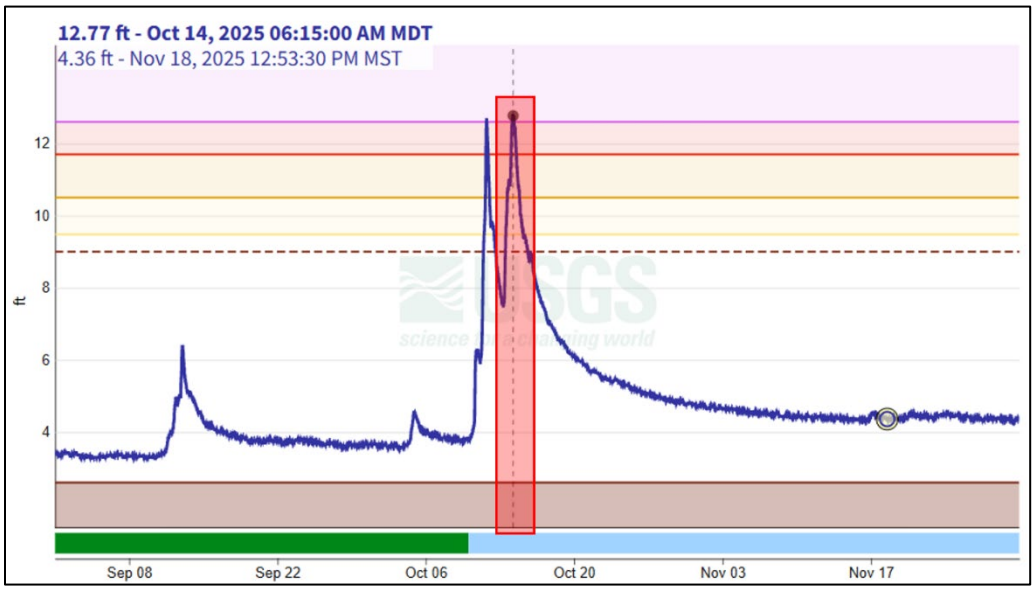
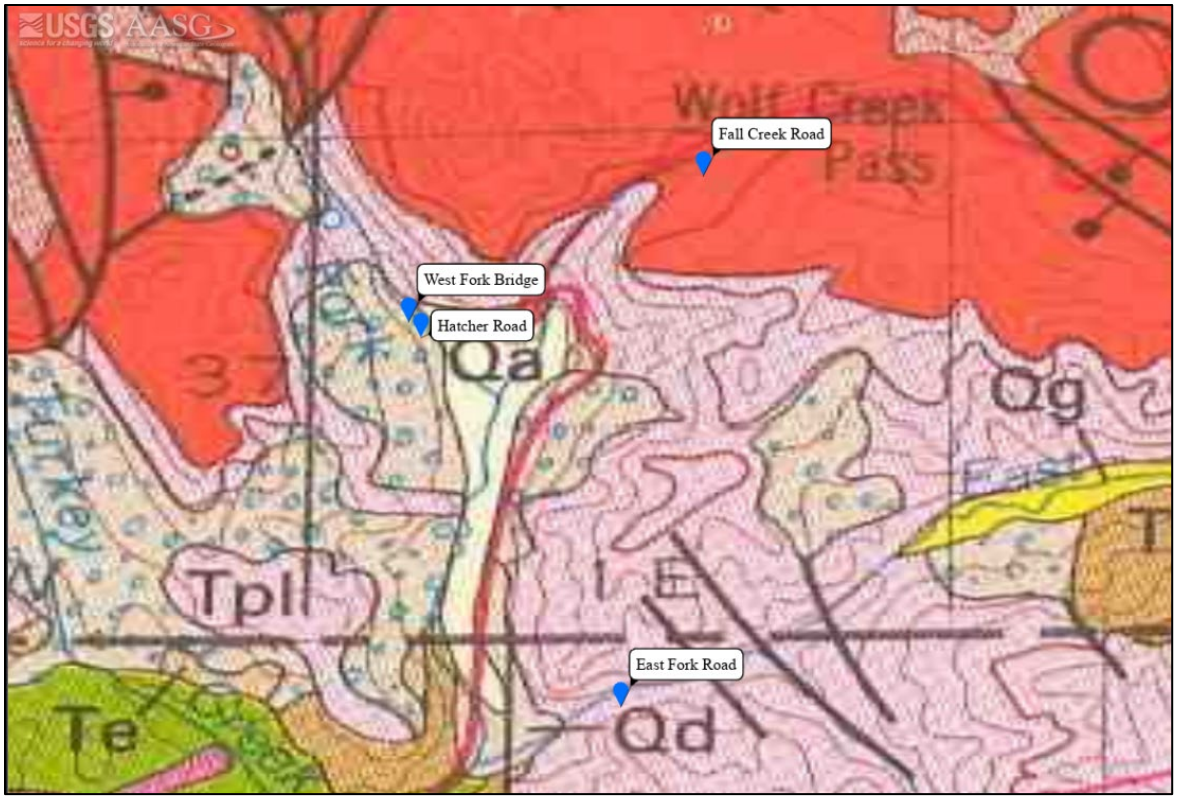


Figure 4: San Juan River at Pagosa Springs Stream Guage (U.S. Geological Survey (1), 2026).

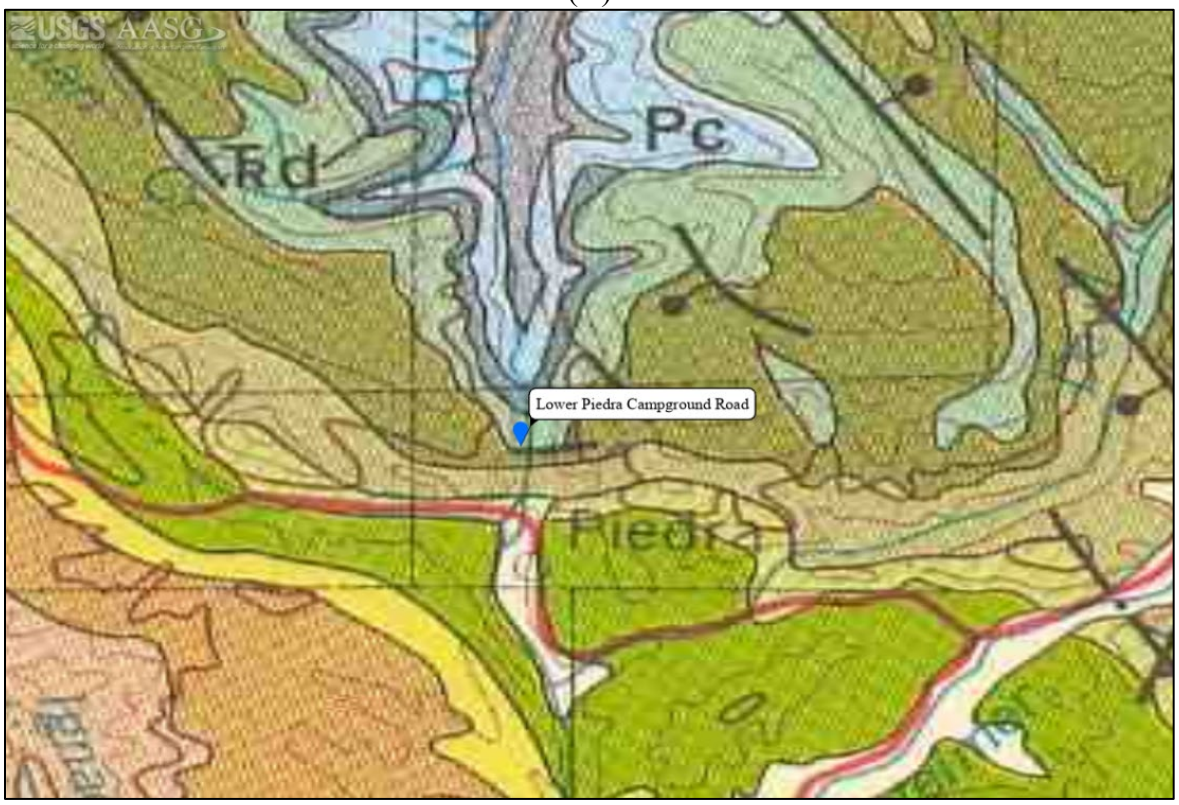
Figure 3 and Figure 4 illustrate the regional severity of the October 10th – 14th storm event. The direct correlation between the precipitation and peak river stages confirms the extreme hydrologic conditions present during the flooding. These elevated water levels and high velocity flows were the primary mechanisms driving the embankment erosion, culvert washouts, and scour observed at the project sites. This context informs the subsequent geotechnical analysis and repair recommendations.

2.2 Regional Geology

Geologic mapping in the vicinity of the damage sites is presented in Figure 5 and the geologic units corresponding to the damage sites are summarized in Table 2. The summary includes the site name, geologic figure reference, geologic unit, unit age, and geologic unit description. The soils at the damage sites predominantly consist of colluvial material.



(A)



(B)

Figure 5: Regional Geology (Tweto, 1979).

Table 2: Geology Units at Damage Sites.

Site	Figure Reference	Geologic Unit	Unit Age	Geologic Unit Description
033 Hatcher Road	Figure 5 (A)	Ql	Quaternary	Landslide Deposits - Locally includes talus, rock-glacier, and thick colluvial deposits.
667 East Fork Road	Figure 5 (A)	Tpl	Tertiary	Pre-ash-flow andesitic lavas, breccias, tuffs, and conglomerates (General age 30-35 M.Y.).
039 Fall Creek Road	Figure 5 (A)	Taf	Tertiary	Ash-flow tuff of main volcanic sequence (Age in San Juan Mountains 26-30 M.Y.).
648 West Fork Bridge	Figure 5 (A)	Ql	Quaternary	Landslide Deposits - Locally includes talus, rock-glacier, and thick colluvial deposits.
621 Lower Piedra Road	Figure 5 (B)	Kdb	Cretaceous	Dakota Sandstone and Burro Canyon Formation - Sandstone, Shale, and Conglomerate.

2.3 Regional Seismicity

No known seismic source faults are mapped within approximately 40 miles of the project area (U.S. Geological Survey (2), 2026). Therefore, the potential for surface fault rupture occurring within the project vicinity is low.

2.4 Seismic Design Parameters

Recommended seismic response parameters for the project sites are based on the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications, 9th Edition, 2020. These parameters represent horizontal peak ground acceleration (PGA) with 7 percent probability of exceedance in 75 years (approximate 1,000-year return period). The 1,000-year return period uniform hazard spectrum for the project sites (see Table 1 for site coordinates) was obtained in accordance with the AASHTO ground motion maps.

The USGS vs30 online map view gave VS30 values ranging from 1,980 to 2,760 ft/s. Based on the site class definitions in AASHTO Table 3.10.3.1-1, the sites range from **Class B** to **Class C** (U.S. Geological Survey (3), 2026). Table 3 summarizes the recommended spectral acceleration coefficient values for probabilistic design.

Table 3: Summary of Seismic Parameters Corrected for Site Class.

	Hatcher Road	East Fork Road	Fall Creek Road	West Fork Bridge	Lower Piedra Road
Site Class	**B**	**B**	**C**	**B**	**C**
Horizontal Peak Ground Acceleration, (PGA)	0.086	0.083	0.086	0.086	0.074
Horizontal Response Spectral Acceleration at Period of 0.2 sec, (S_s)	0.196	0.191	0.196	0.197	0.167
Horizontal Response Spectral Acceleration at Period of 1.0 sec, (S_1)	0.057	0.057	0.058	0.057	0.05
Site Factor at Zero-Period of Acceleration Spectrum, (F_{pga})	1	1	1.2	1	1.2
Site Factor at Short-Period Range of Acceleration Spectrum, (F_a)	1	1	1.2	1	1.2
Site Factor at Long-Period Range of Acceleration Spectrum, (F_v)	1	1	1.7	1	1.7
Factored Horizontal Peak Ground Acceleration, (A_s)	0.086	0.083	0.103	0.086	0.089
Factored Horizontal Response Spectral Acceleration at Period of 0.2 sec, (SD_s)	0.196	0.191	0.235	0.197	0.200
Factored Horizontal Response Spectral Acceleration at Period of 1.0 sec, (SD_1)	0.057	0.057	0.098	0.057	0.084
Seismic Zone	**Zone 1**	**Zone 1**	**Zone 1**	**Zone 1**	**Zone 1**

Based on the long acceleration coefficient S_{D1} values (0.057 – 0.098), the sites are assigned to Seismic Hazard Zone 1, in accordance with Table 3.10.6-1 of AASHTO.

2.5 Geologic Hazards

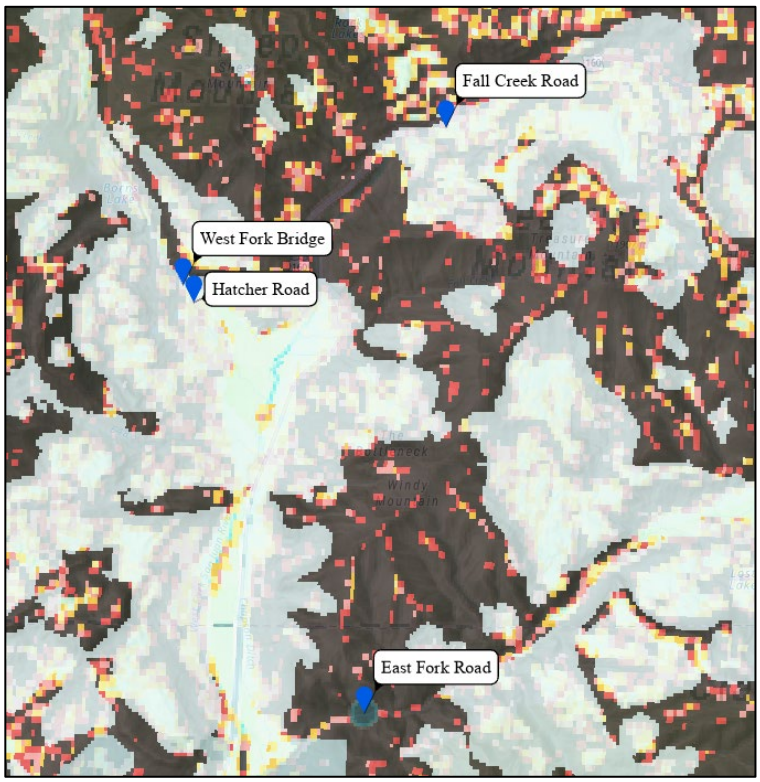
Potential geological hazards within the project vicinity include those related to ground subsidence, expansive soils, and abandoned mining operations (Colorado Geological Survey, 2026). These regional characteristics are not anticipated at the project sites.

High erosion potential of soils also exists due to the dry climate and high intensity precipitation events. Additionally, steep terrain and colluvial material increase the risk of mass movements. Other natural hazards, such as drought and wildfire, have also been reported in the project region. In general, these hazards represent landscape scale processes that are outside the scope of this project to mitigate.

Embankment armoring is recommended to reduce the potential for erosion at West Fork Bridge, Hatcher Road, East Fork Road, and Lower Piedra Road. Hydraulic countermeasures and

appropriate surface drainage capacity should also be provided as directed by the CFLHD Hydraulics Engineer.

Figure 6 displays mapped landslide susceptibility near the project sites sourced from the U.S. Geological Survey Landslide Hazard Program. The Landslide Hazards Program produces maps indicating historical landslide locations and potential future landslide risks. The landslide susceptibility models analyze different terrain features to show where landslides are more or less likely to occur (U.S. Geological Survey (5), 2026). Figure 6 (A) indicates that East Fork Road is flanked by slopes with high landslide susceptibility (grey/dark red). Hatcher Road, West Fork Bridge, and Fall Creek Road all lie in areas of moderate (yellow) to high (red) landslide susceptibility. Figure 6 (B) indicates that Lower Piedra Road is located at the base of a slope with high landslide susceptibility (grey/dark red). Addressing these mapped landside susceptibilities is outside the scope of this project.



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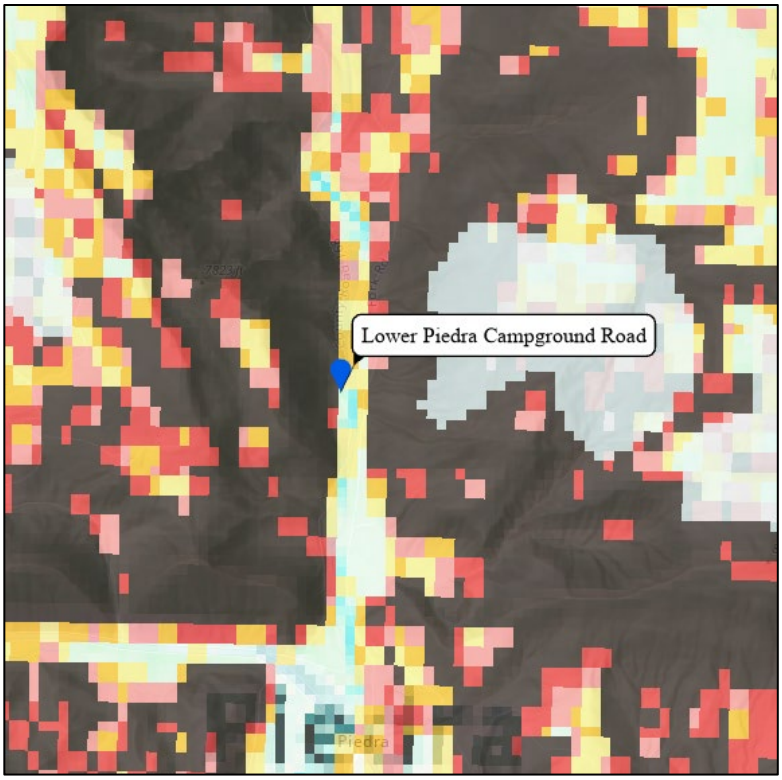

(B)

Figure 6: Landslide Susceptibility (U.S. Geological Survey (5), 2026).

3.0 FIELD EVALUATION

To evaluate the field conditions, a CFLHD team visited the project site and conducted limited field measurements and observations on 12/2/2025. This team included Justin Henwood (Project Manager), Luis Calderon (Hydraulic Engineer), and Kelly Keele (Design Engineer). The CFLHD team met with Forest Service personnel Allen McCaw and Kim Bristow.

Field observations focused on evaluating the existing roadway conditions and soil properties. Evaluations and recommendations presented in this memorandum are based on visual observations and the limited measurements taken during the site visit. Due to ERFO project time and budget constraints to expedite project delivery and reestablish roadway access, no subsurface investigations were conducted.

3.1 033 Hatcher Road

The flooding triggered high flows within the West Fork San Juan River that severely eroded the riverbank supporting the roadway. The damage spans approximately 250 feet along the roadway. The northernmost 125 feet of erosion resulted in a complete loss of roadway whereas the remaining 125 feet of damage to the south affected the native slope but left the road intact. The erosion extends approximately 28 vertical feet from the roadway prism down to the stream channel. Based

on site observations, a colluvial terrace deposit formed the natural slopes beneath the roadway. The native soil consists of silty sand mixed with gravel, cobbles, and boulders. The outboard slope failure at this site can be seen in Figure 7.

Figure 7: Hatcher Road Slope Failure.

3.2 667 East Fork Road

The riverbank supporting the roadway experienced severe erosion due to high flows within the East Fork San Juan River. The erosional damage extends approximately 66 feet along the riverbank, encroaches 8 feet into the travel way, and drops 8 vertical feet from the roadway prism to the streambed. The native soil profile consists of silty sand mixed with gravel, cobbles, and boulders. Figure 8 presents photos of the damage.

(A)

(B)

Figure 8: East Fork Road.

Xcel Energy performed emergency repairs at this site to establish temporary access and allow for the removal of seasonal equipment. Crews removed a log jam within the East Fork San Juan River and partially reconstructed the embankment using native material. These temporary repairs restored 4 feet of the travel way, bringing the current roadway width to 14 feet at this location. Logs and debris were placed at the toe of the repaired embankment to mitigate further erosion. A photo of the temporary repair is presented in Figure 9.

Figure 9: Temporary Repair of East Fork Road.

3.3 039 Fall Creek Road

High flows within Wolf Creek caused the failure of a double 72-inch by 60-foot culvert structure and eroded the adjacent roadway embankment. The erosional damage extends approximately 48 feet along the road centerline and 21 feet deep from the roadway to the streambed. Native materials in this area consist of silty sand with gravel, cobbles, and boulders. The embankment is composed of native material with 1V:3H side slopes. Figure 10 displays photos of the site damage.

(A)

(B)

Figure 10: Fall Creek Road.

3.4 648 West Fork Bridge

High flows within the West Fork San Juan River overtopped West Fork Bridge, deposited debris on and around the structure, eroded the west bridge approach, and scoured approximately 50 cubic yards from the west abutment spread footing. The flooding has redirected the channel to strike the west abutment. The Forest Service salvaged streambed material for use as emergency rip rap to protect the scoured abutment. Figure 10 presents a photo of the damage. The emergency riprap placed around the west abutment is highlighted in red.

Figure 11: West Fork Bridge.

3.5 621 Lower Piedra Road

The outboard slope at this location experienced severe erosion due to high flows within the Piedra River. The damage spans approximately 600 feet along the center line of the roadway, encroaches 8 feet into the travel way, and drops 10 vertical feet from the roadway prism to the stream bed. The native soil profile consists of silty sand mixed with gravel, cobbles, and boulders. Figure 12 presents photos of the damage.

(A) (B)

Figure 12: Lower Piedra Road.

4.0 ANALYSIS

This section presents the analysis for the repairs at Hatcher Road and Fall Creek Road. The proposed repairs for East Fork Road and Lower Piedra Road are discussed in Section 5.0. Scour repair recommendations for West Fork Bridge are detailed in the Final Hydraulics Report dated 5/28/2026.

4.1 Slope Stability Back Analysis

Slope stability analysis was completed for the failed embankment at Hatcher Road. Back-calculation analyses for slope conditions at failure were first conducted using the Slide 2 computer program developed by RocScience. Slide2 uses limit equilibrium methods to determine the factors of safety for slope stability. The original condition cross sections used in the back analysis were developed from one-meter accuracy LiDAR scans of the project area (U.S. Geological Survey (4), 2026).

Presumptive soil properties were estimated based on visual classification and experience with similar soil materials for initial input in the back analysis. An incipient static factor of safety of 1.0 was used to refine the estimated soil parameters.

A phreatic surface at an elevation of 7,899 feet was incorporated into the back analyzed profile to reflect the water level of the West Fork San Juan River. This elevation was selected based on the maximum modeled depth of 5.62 feet for the 25-year storm event, as detailed in the Hydraulics report. For this analysis, the 25-year water level was correlated to an assumed water depth of 5 feet at the toe of the critical (steepest) cross section. The phreatic surface was modeled to extend horizontally across the entire length of the cross section. This is a conservative assumption for slope stability analysis and does not necessarily reflect the actual phreatic surface.

The back analyzed and assumed soil and rock parameters were used in further analyses to examine the effectiveness of repair options and are summarized in Table 4 below. Results of the slope stability analysis are attached to this memorandum.

Table 4: Material Properties Used in Slope Stability Design.

Soil Type	**Unit Weight**	**Friction Angle**	**Cohesion**
	(*pcf*)	**(*degrees*)**	**(*psf*)**
Native Soil (SM w/ gravel)	125	34	80
RSS Backfill	125	32	0
SRE	150	48	0

4.2 Reinforced Soil Slope

Based on the observed conditions and to comply with the ERFO objectives to restore the roadway to acceptable and safe conditions within budget, a Reinforced Soil Slope (RSS) founded on a special rock embankment (SRE) was analyzed as the preferred repair alternative for Hatcher Road. SRE recommendations are discussed in Section 5.2 and RSS recommendations are discussed in Section 5.3.

Target minimum factors of safety used in RSS design are presented in Table 5 below. These values were adopted based on design guidelines provided in FHWA GEC011 – Volume II (U.S. Department of Transportation Federal Highway Administration, 2009).

Table 5: RSS Performance Requirements.

Performance Condition	FS Requirement
Global Stability	≥ 1.3
Internal Stability	≥ 1.3
Sliding Resistance	≥ 1.3
Local Bearing	≥ 1.3
Seismic Stability	≥ 1.1

Global, internal, and external stability analysis were performed for the proposed RSS. Slide 2 was used to analyze global stability whereas ReSSA+ was used to evaluate internal and external stability. The ReSSA+ program uses limit equilibrium methods to determine factors of safety for circular, two-part wedge, and three-part wedge failure surfaces.

The RSS analysis assumed the use of Type I Reinforcement Geosynthetic (FP-24, 714.04) with a long-term design strength of 1,000 lb/ft at 1-foot vertical spacing. The outboard slope ratio of the RSS was limited to 1(V):1(H) to reduce the potential for surficial sloughing and promote revegetation. The maximum design heights were determined from cross sections prepared by the CFLHD Design Engineer. The analysis assumed that drainage would be provided to the reinforced fill and did not include hydrostatic pressures. Reinforcement lengths were varied incrementally to produce satisfactory factors of safety for the proposed RSS.

Seismic design requirements for the RSS were adapted from the MSE wall seismic design requirements in AASHTO 11.5.4.2 due to the lack of explicit seismic guidance for RSS structures in the TGM and NHI manuals. Seismic design is not mandatory for walls within Seismic Zones 1 through 3; therefore, internal and external seismic analysis for the RSS was waived since the project is in Seismic Zone 1. However, a seismic global stability analysis was conducted for the RSS due to the steep slope face and site's close (3 mile) proximity to the highway. Results of the RSS analyses are summarized in Table 6 below and are attached to this memorandum.

Table 6. Results of Global Stability and RSS Analyses

Site	Max. Design Height	Range of Reinforcement Length[(1)]	Factors of Safety			
	(*ft*)	(*ft*)	Static			Seismic
			Global	Two-Part Wedge	Three-Part Wedge	Global
Hatcher Road	20	12 - 20	1.35	1.30	1.58	1.25

(1) Reinforcement extended to back of RSS excavation.

4.3 Box Culvert Design

A new box culvert is proposed to repair the culvert and embankment failure at Fall Creek Road. The proposed box culvert will be 16 feet wide, 8 feet tall, and 76 feet long. The box culvert is assumed to be founded on silty sand with gravel, cobbles, and boulders.

4.3.1 Bearing Resistance and Settlement

Ultimate bearing resistance of the culvert is dependent on both the length and width of the foundation elements. A foundation length of 76 feet was used in the analysis. The proposed culvert length was provided by the Hydraulics Engineer. Based on knowledge of the geologic conditions near the CBC site, the presumptive bearing resistance can be estimated to be between 8 – 14 ksf with a recommended value of use of 10 ksf, not to be exceeded, according to Table C10.6.2.5.1-1 of AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020. The bearing resistance values presented in this table are based on a maximum settlement of one inch and only apply at the service limit state (resistance factor of 1.0). Bearing resistance for strength, extreme event, and service limit states are included in the attachments. The bearing resistances provided are nominal values and should be modified by the appropriate AASHTO LRFD resistance factors. It is recommended that the base of the footings be embedded below the local 42-inch frost depth (Town of Pagosa Springs, 2026). Additionally, the footings should be protected from scour, as recommended by the Hydraulics Engineer.

4.3.2 Lateral Loads

The culvert should be designed to resist lateral loads based on the soil parameters presented in Table 7 below. These properties were estimated based on typical design values shown in the NAVFAC Design Manual 7.01. The values are unfactored loads and assume that the surface of

the soil slope behind the wall is horizontal. The equivalent fluid densities do not include any surcharge for sloping backfill surfaces or other loads. These equivalent fluid densities do not include load factors or factors of safety; the designer should apply appropriate factors based on their design methodology. Lateral loads imposed on the structures are resisted by the development of friction between the base of the structure and the supporting soils. The nominal friction factors presented in Table 7 should be used for the design of the culvert.

Table 7: Lateral Load Design Parameters.

Material Type	Assumed Material Properties	Case	Unfactored Equivalent Fluid Density (PCF) (1)	Nominal Friction Factor
Structural Backfill	c = 0 psf	Active	35	0.67
	ϕ = 34 deg.			
	γ = 125 pcf	At-Rest	55	0.67
GC w/ Sand (Native Soil)	c = 0 psf	Active	35	0.65
	ϕ = 33 deg.			
	γ = 119 pcf	At-Rest	54	0.65

(1) Values are unfactored loads and assume that the surface of the slope behind the wall is horizontal.

The AASHTO LRFD resistance factors for various limit states are presented in Table 8 below.

Table 8: Resistance Factors (ϕ) for Bearing and Sliding Limit States.

Limit State	Bearing	Shear Resistance to Sliding (Cast in Place)	Shear Resistance to Sliding (Precast)	Passive Pressure Resistance to Sliding
Strength I	0.45	0.80	0.90	0.50
Service I & Extreme Event I, II	1.00	1.00	1.00	1.00

4.4 Wing Wall Design

Box culvert wingwalls should be designed to resist lateral earth pressures and other applicable lateral loads in accordance with AASHTO. Lateral earth pressure is influenced by the strength of

the backfill, the presence or absence of water, and the ability of the wall to move in response to lateral loads. Other loads, such as live loads, construction loads, and soil compaction loads should also be considered in the design.

4.4.1 Bearing Resistance

Ultimate bearing resistance of the box culvert wingwall foundations is dependent on the length and width of the foundation elements. For these calculations, a foundation length of 12 feet was selected, based on the proposed culvert size and the proposed embankment side slopes. The ultimate bearing capacity of the foundation soils for various foundation widths is presented in the attachments. The appropriate resistance factor should be applied to the ultimate capacity to determine the factored capacity. Resistance factors for various limit states are presented in Table 8 above.

4.4.2 Lateral Loads

Retaining structures should be designed to resist lateral pressures depending on the restraint condition. Structures that can deflect (active condition) should be designed to resist loads based on the active equivalent fluid density presented in Table 7. Walls that are restrained (at-rest condition) should be designed based on the at-rest equivalent fluid density. If structural backfill is placed within the active or at-rest zones behind the retaining structures, reduced equivalent fluid densities may be used. The equivalent fluid densities do not include any surcharge for sloping backfill surfaces or other loads. The equivalent fluid densities do not include load factors or factors of safety; the designer should apply appropriate factors based on their design methodology. Below the mean water level, design the structures for the full hydrostatic condition.

Lateral loads imposed on the structures are resisted by development of friction between the base of the structure footing and the supporting soils. The nominal friction factors presented above in Table 7 may be used for the design of cast-in-place footings established on native soil.

5.0 RECOMMENDATIONS

5.1 Rip Rap Armoring

Rip Rap armoring is proposed at Hatcher Road, East Fork Road, West Fork Bridge, and Lower Piedra Road. Place rip rap armor at a maximum slope of 1V:1.5H. Conform to Hydraulics recommendations for rip rap gradation.

5.2 Special Rock Embankments

Table 9 summarizes the proposed SREs. Construct the SRE as shown on the plan set and according to Section 252 of the FP-24. Conform to SCRs for separation geotextile.

Table 9: SRE Recommendations

Site	Description	Max Slope	Minimum Gradation Requirements (FP-24)	Note
033 Hatcher Road	RSS Foundation	1V:1.5H	Class 6 Rip Rap (Table 705-1)	See Hydraulics recommendations for scour embedment.
667 East Fork Road	Embankment Repair	1V:1.5H	Class 6 Rip Rap (Table 705-1)	See Hydraulics recommendations for scour embedment.

The temporary repairs performed by Xcel Energy at East Fork Road must be removed prior to construction of the SRE. Excavate adequate material to achieve the appropriate base width for SRE as shown on the plan set.

5.3 RSS

5.3.1 Reinforcement

Furnish Type I Reinforcement Geosynthetic (FP-24, 714.04) with a long-term design strength of 1,000 lb/ft. Install reinforcement at a 1-foot vertical spacing. Extend reinforcement to the back of the excavation bench.

5.3.2 Backfill Material

It is assumed that material will be imported for RSS backfill. Furnish RSS Backfill in accordance with Table 704-1 of the FP-24.

5.3.3 Erosion Control

Erosion control best management practices should be implemented to stabilize the slopes. A type 5.A turf reinforcement mat (FP-24, Table 713-4) is recommended to assist with post-construction revegetation. Install turf reinforcement mat as shown on the plan set.

6.0 DISCLAIMERS/LIMITATIONS CLAUSE

Subsurface explorations were not performed as a part of this scope of work. Interpretation of surface and subsurface conditions is based on limited field reconnaissance and surface mapping of soil and rock outcrops. The Recommendations section of this report includes interpretations and recommendations developed by the Government in the process of preparing the design. These interpretations are not intended as a substitute for the personal investigation, independent interpretation, and judgement of the Contractor.

Attachments (2)

Hatcher Road – Slope Stability Analysis

Fall Creek Road – Bearing Capacity Analysis

Prepared by:

AARON ALGIRDAS BAUKUS

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Reviewed by:

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Brendan L. McGarity

Geotechnical Engineer, CFLHD

Approved by:

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Devin T. Dixon, P.E.

Geotechnical Team Leader, CFLHD

7.0 REFERENCES

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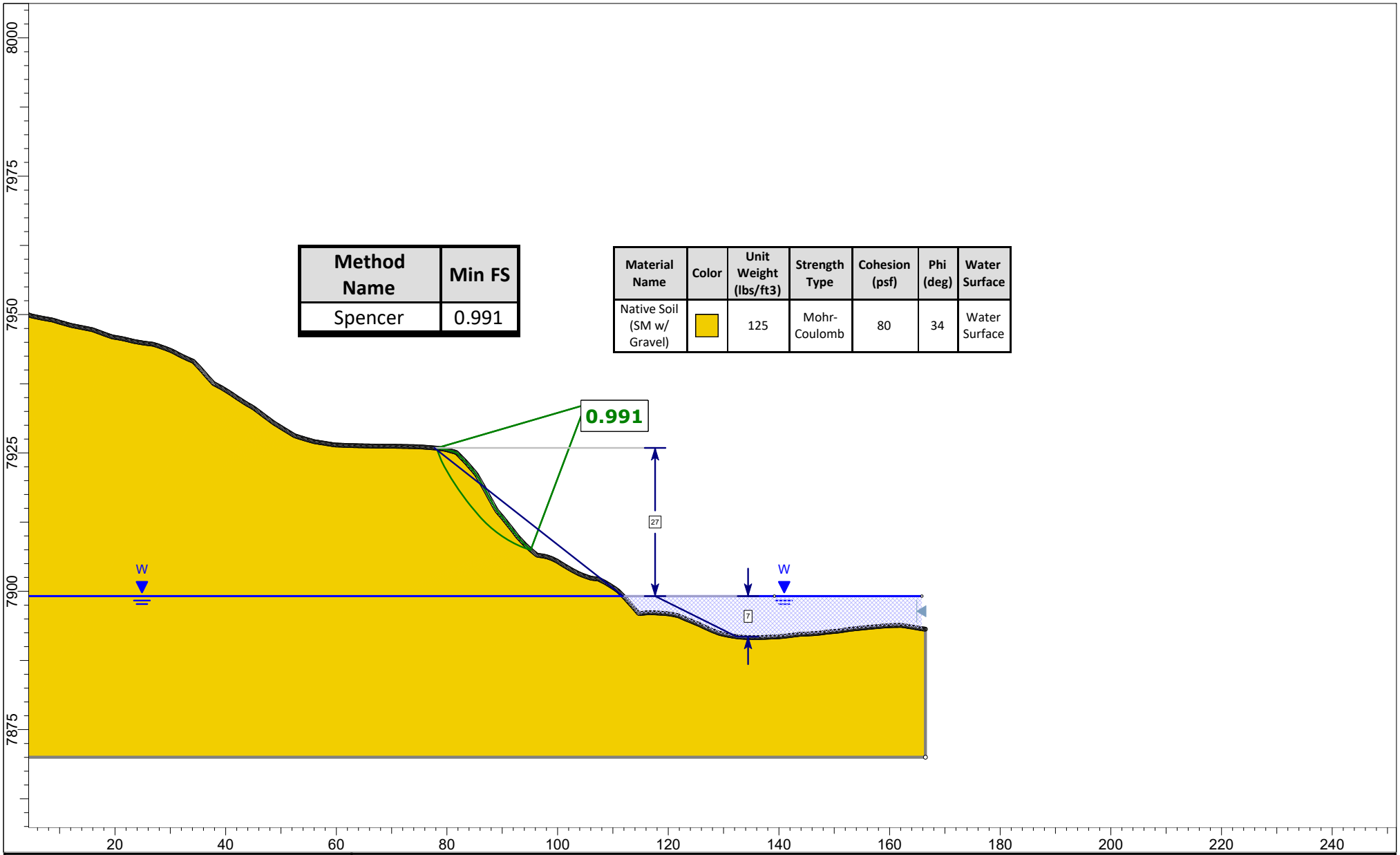
HATCHER ROAD - SLOPE STABILITY ANALYSIS

Method Name	Min FS
Spencer	0.991

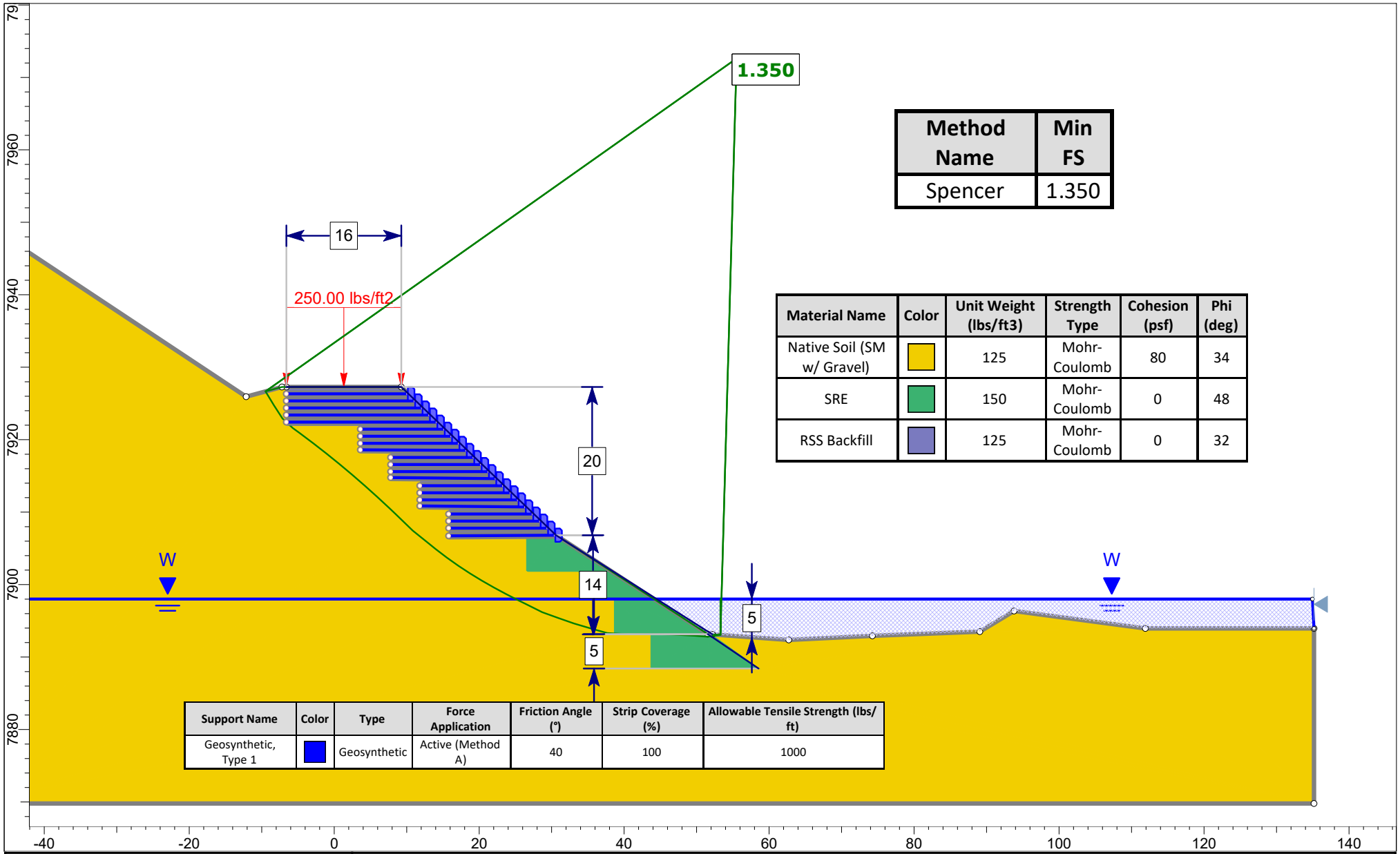
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Native Soil (SM w/ Gravel)		125	Mohr-Coulomb	80	34	Water Surface



U.S. Department of Transportation
Federal Highway Administration

Project	CO ERFO FS SNJUAN2026-1(1) - Hatcher Road		
Group	Back Analysis - STA 6+45	*Scenario*	Back Analysis w/ H20
Drawn By	A.Baukus	*Company*	FHWA
Date	2/19/2026, 1:56:22 PM	*File Name*	Hatcher_Road_v06-No_Key.slmd

SLIDEINTERPRET 9.024



Method Name	Min FS
Spencer	1.350

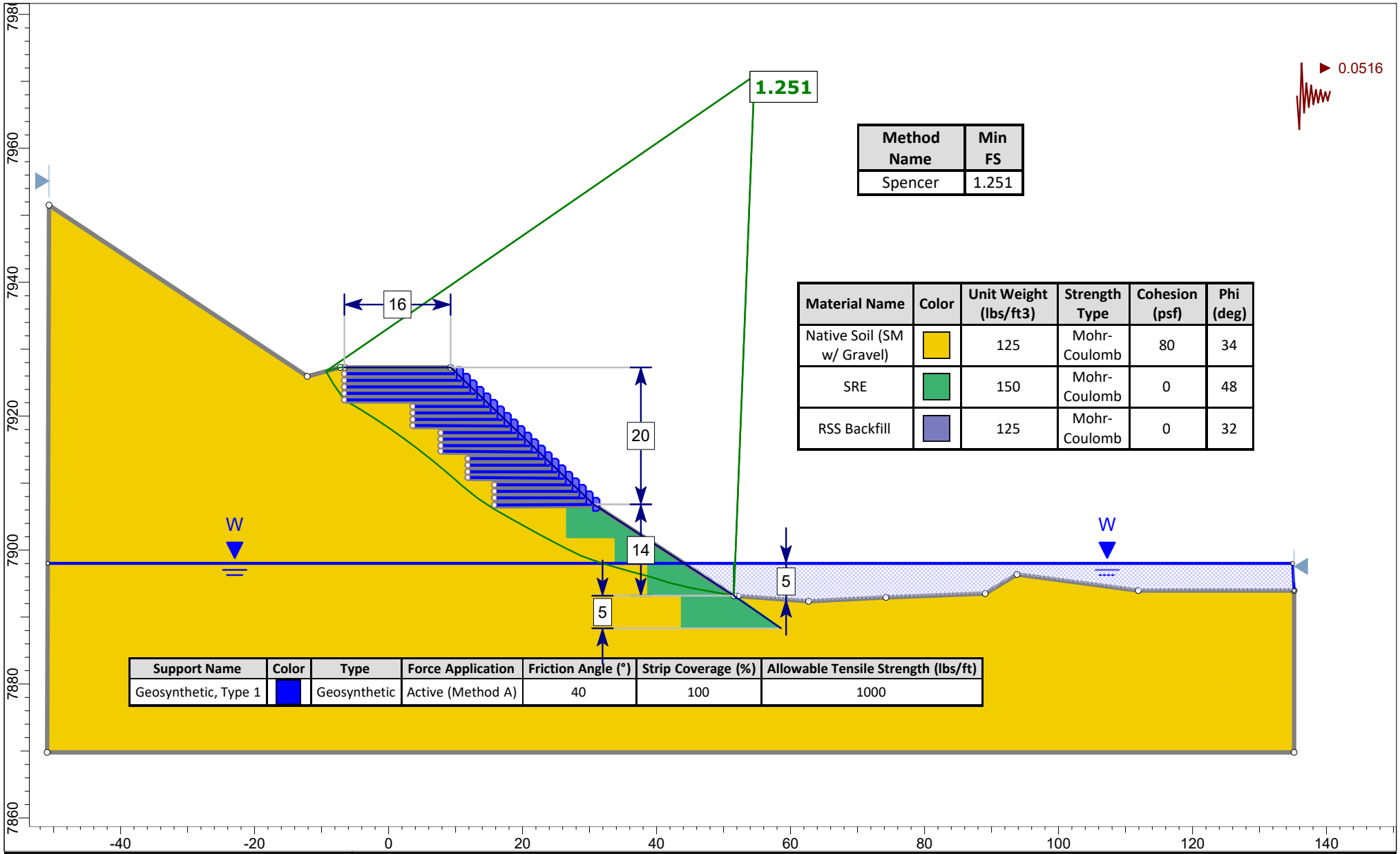
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Native Soil (SM w/ Gravel)		125	Mohr-Coulomb	80	34
SRE		150	Mohr-Coulomb	0	48
RSS Backfill		125	Mohr-Coulomb	0	32

Support Name	Color	Type	Force Application	Friction Angle (°)	Strip Coverage (%)	Allowable Tensile Strength (lbs/ft)
Geosynthetic, Type 1		Geosynthetic	Active (Method A)	40	100	1000

U.S. Department of Transportation
Federal Highway Administration

SLIDEINTERPRET 9.024

Project	CO ERFO FS SNJUAN2026-1(1) - Hatcher Road		
Group	Repair Analysis - STA 6+30	*Scenario*	Hatcher Repair - Global Stability
Drawn By	A.Baukus	*Company*	FHWA
Date	2/19/2026, 1:56:22 PM	*File Name*	Hatcher_Road_v06-No_Key.slmd



Method Name	Min FS
Spencer	1.251

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Native Soil (SM w/ Gravel)		125	Mohr-Coulomb	80	34
SRE		150	Mohr-Coulomb	0	48
RSS Backfill		125	Mohr-Coulomb	0	32

Support Name	Color	Type	Force Application	Friction Angle (°)	Strip Coverage (%)	Allowable Tensile Strength (lbs/ft)
Geosynthetic, Type 1		Geosynthetic	Active (Method A)	40	100	1000

U.S. Department of Transportation
Federal Highway Administration

SLIDEINTERPRET 9.024

Project	CO ERFO FS SNJUAN2026-1(1) - Hatcher Road		
Group	Repair Analysis - STA 6+30	*Scenario*	Hatcher Repair - Seismic Stability
Drawn By	A.Baukus	*Company*	FHWA
Date	2/19/2026, 1:56:22 PM	*File Name*	Hatcher_Road_v06-No_Key.slmd

Hatcher RSS Analysis

ReSSA+: Update #0.180

Report created by ReSSA+: Copyright (c) 2001-2024, ADAMA Engineering, Inc.

PROJECT IDENTIFICATION

Title: Hatcher RSS Analysis
Project Number: CO ERFO FS SNJUAN 2026 1-(1) -
Client: San Juan National Forest
Designer: A.baukus
Station Number: 6+30

Description:

Company's information:

Name: CFLHD
Street: 12300 W Dakota Ave
Lakewood, CO 80228
Telephone #:
Fax #:
E-Mail:

File path and name: C:\Users\A ..... Road\04_Final Design\RESSA\Hatcher_v05-No_Key.MSEp
Original date and time of creating this file: Wed May 13 10:29:18 2026

PROGRAM MODE: Analysis of a General Slope using GEOSYNTHETIC as reinforcing material.

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INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

========== Soil Layer #: ==========	Unit weight, γ [lb/ft ³]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ²]
1	125.0	34.0	80.0
2	125.0	32.0	0.0
3	125.0	32.0	0.0
4	150.0	48.0	0.0
5	125.0	34.0	80.0

REINFORCEMENT

Reinforcement Type #	Geosynthetic Designated Name	Ultimate Strength, Tult [lb/ft]	Reduction Factor for Installation Damage, RFid	Reduction Factor for Durability, RFd	Reduction Factor for Creep, RFc	Additional Reduction Factor, RFa	Coverage Ratio, Rc
1	Geosynthetic type #1	1000.00	1.20	1.10	1.67	1.00	1.00

Interaction Parameters Type #	Geosynthetic Designated Name	== Direct Sliding == Cds-phi	Cds-c	==== Pullout ==== Ci	Alpha
1	Geosynthetic type #1	0.80	0.00	0.80	0.80

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Unit weight of water = 62.45 [lb/ft ³]
Water pressure is defined by phreatic surface in Effective Stress Analysis.

SEISMICITY

Not Applicable

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DRAWING OF SPECIFIED GEOMETRY - GENERAL

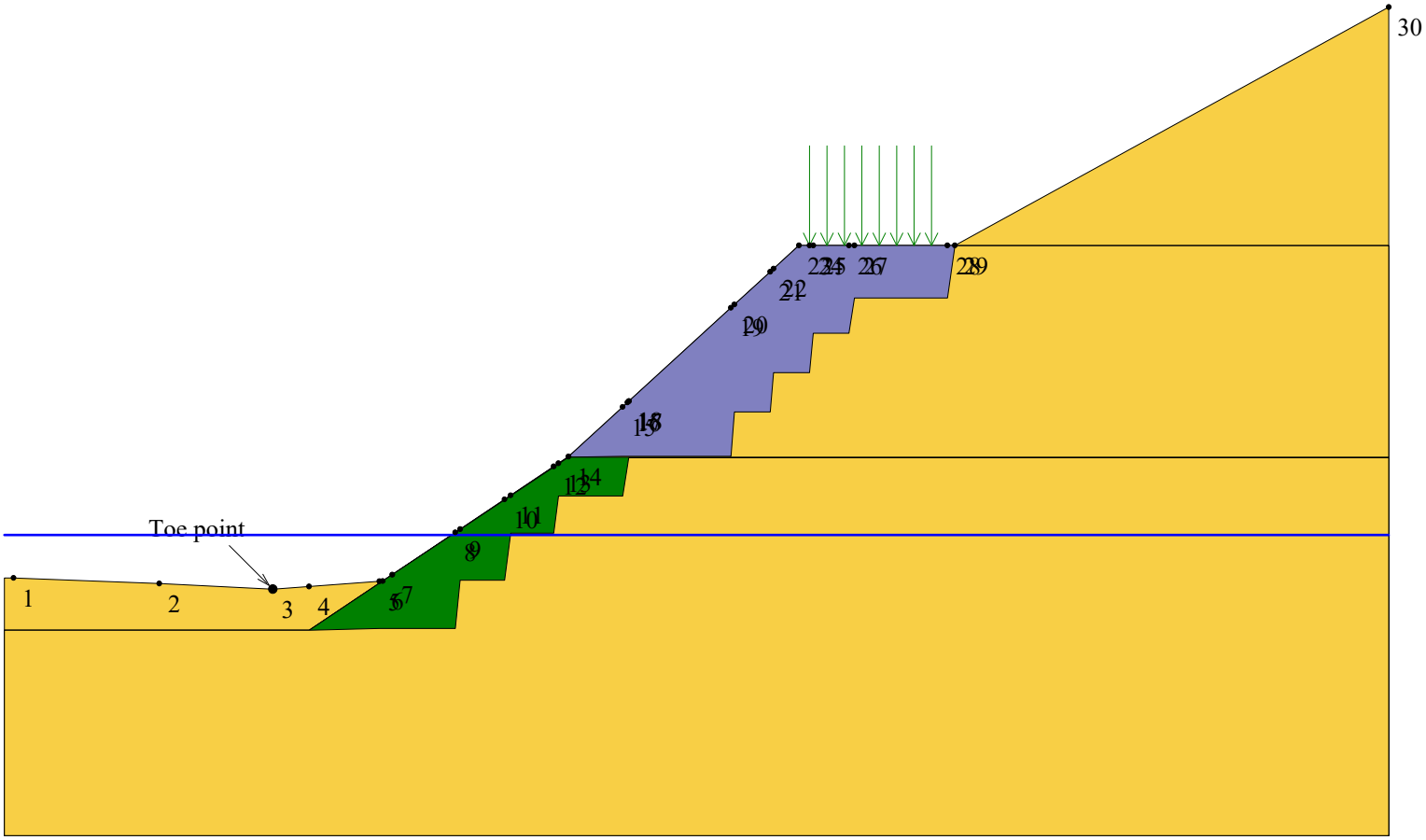
-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.
-- Xw,Yw represents the coordinates of phreatic surface.

GEOMETRY
Soil profile contains 5 layers

WATER GEOMETRY
Phreatic line was specified.

UNIFORM SURCHARGE
Load Q1 = 250.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X1s = 127.00 and ends at X1e = 141.00 [ft].
Surcharge load, Q2.....None
Surcharge load, Q3.....None

STRIP LOAD
.....None.....



SCALE:

0 2 4 6 [ft]

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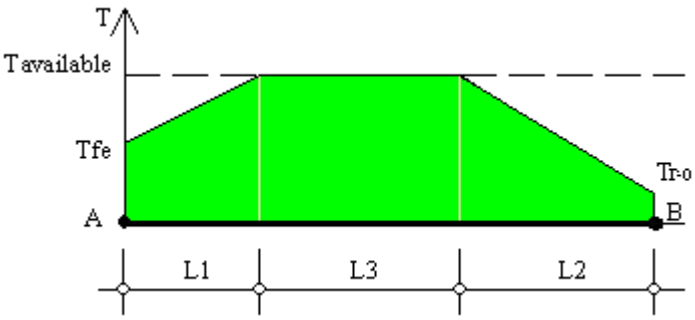
DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER



A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)
Tr-o = Pullout resistance at rear-end

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tr-o [lb/ft]	Tavailable [lb/ft]
1	Geosynthetic type #1	14.00	14.80	0.00	0.52	14.28	453.64	0.00	453.64
2	Geosynthetic type #1	15.00	13.70	0.00	0.56	13.14	453.64	0.00	453.64
3	Geosynthetic type #1	16.00	12.70	0.00	0.62	12.08	453.64	0.00	453.64
4	Geosynthetic type #1	17.00	11.70	0.00	0.66	11.04	453.64	0.00	453.64
5	Geosynthetic type #1	18.00	14.70	0.00	0.52	14.18	453.64	0.00	453.64
6	Geosynthetic type #1	19.00	13.70	0.00	0.56	13.14	453.64	0.00	453.64
7	Geosynthetic type #1	20.00	12.70	0.00	0.62	12.08	453.64	0.00	453.64
8	Geosynthetic type #1	21.00	11.70	0.00	0.66	11.04	453.64	0.00	453.64
9	Geosynthetic type #1	22.00	14.70	0.00	0.56	14.14	453.64	0.00	453.64
10	Geosynthetic type #1	23.00	13.70	0.00	0.59	13.11	453.64	0.00	453.64
11	Geosynthetic type #1	24.00	12.70	0.00	0.62	12.08	453.64	0.00	453.64
12	Geosynthetic type #1	25.00	11.70	0.00	0.69	11.01	453.64	0.00	453.64
13	Geosynthetic type #1	26.00	14.70	0.00	0.79	13.91	453.64	0.00	453.64
14	Geosynthetic type #1	27.00	13.70	0.00	0.89	12.81	453.64	0.00	453.64
15	Geosynthetic type #1	28.00	12.70	0.00	0.98	11.72	453.64	0.00	453.64
16	Geosynthetic type #1	29.00	11.70	0.00	1.18	10.52	453.64	0.00	453.64
17	Geosynthetic type #1	30.00	20.00	0.00	1.41	18.59	453.64	0.00	453.64
18	Geosynthetic type #1	31.00	19.00	0.00	1.74	17.26	453.64	0.00	453.64
19	Geosynthetic type #1	32.00	18.00	0.00	2.36	15.64	453.64	0.00	453.64
20	Geosynthetic type #1	33.00	17.00	0.00	3.58	13.42	453.64	0.00	453.64
21	Geosynthetic type #1	34.00	16.00	0.00	7.41	8.59	453.64	0.00	453.64

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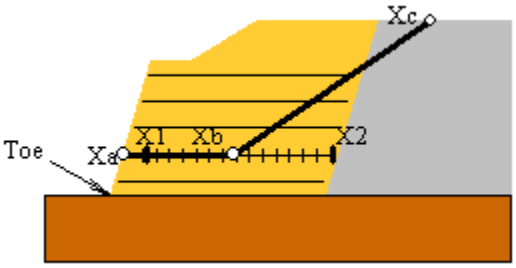
RESULTS OF TRANSLATIONAL ANALYSIS



Results in the table below represent critical two-part wedges identified between specified starting (X1) and ending (X2) search points. Wedges along all reinforcement layers and at elevation zero are reported. The critical two-part wedge, one for each predetermined elevation, is defined by Xa, Xb and Xc where Xa is the front end of the passive wedge (slope face), Xb is where the passive wedge ends and the active one starts, and Xc is the X-ordinate at which the active wedge starts.

Critical two-part wedge along each interface:

Interface	Height Relative to Toe [ft]	(Xa, Ya) [ft]		(Xb, Yb) [ft]		(Xc, Yc) [ft]		Fs	STATUS
At toe elevation	0.00	72.46	0.00	92.30	0.00	194.73	59.14	1.39	Minimum on Edge
Reinf. Layer #1	14.00	103.08	14.00	118.10	14.00	142.86	35.52	1.29	Minimum on Edge
Reinf. Layer #2	15.00	104.17	15.00	118.10	15.00	144.68	36.52	1.31	Minimum on Edge
Reinf. Layer #3	16.00	105.26	16.00	118.20	16.00	143.97	36.13	1.36	Minimum on Edge
Reinf. Layer #4	17.00	106.35	17.00	118.30	17.00	146.70	37.63	1.42	Minimum on Edge
Reinf. Layer #5	18.00	107.45	18.00	119.54	18.00	150.78	39.87	1.51	OK
Reinf. Layer #6	19.00	108.54	19.00	122.50	19.00	143.00	35.60	1.54	Minimum on Edge
Reinf. Layer #7	20.00	109.63	20.00	122.60	20.00	144.12	36.21	1.61	Minimum on Edge
Reinf. Layer #8	21.00	110.72	21.00	122.70	21.00	142.16	35.14	1.70	Minimum on Edge
Reinf. Layer #9	22.00	111.81	22.00	123.94	22.00	141.72	34.92	1.81	OK
Reinf. Layer #10	23.00	112.91	23.00	118.92	23.00	135.94	34.92	1.90	OK
Reinf. Layer #11	24.00	114.00	24.00	119.62	24.00	135.22	34.92	1.96	OK
Reinf. Layer #12	25.00	115.09	25.00	120.22	25.00	130.49	34.92	2.00	OK
Reinf. Layer #13	26.00	116.18	26.00	122.52	26.00	134.36	34.92	2.10	OK
Reinf. Layer #14	27.00	117.27	27.00	123.22	27.00	135.42	34.92	2.17	OK
Reinf. Layer #15	28.00	118.36	28.00	123.92	28.00	131.34	34.92	2.10	OK
Reinf. Layer #16	29.00	119.46	29.00	124.62	29.00	132.20	34.92	2.37	OK
Reinf. Layer #17	30.00	120.55	30.00	125.12	30.00	129.25	34.92	2.38	OK
Reinf. Layer #18	31.00	121.64	31.00	126.02	31.00	129.08	34.92	2.44	OK
Reinf. Layer #19	32.00	122.73	32.00	126.92	32.00	129.64	34.92	2.53	OK
Reinf. Layer #20	33.00	123.82	33.00	127.82	33.00	130.19	34.92	3.48	OK
Reinf. Layer #21	34.00	124.92	34.00	128.72	34.00	130.03	34.92	5.00	OK

Note: In the 'Status' column, OK means the critical two part-wedge was identified within the specified search domain. 'Minimum on Edge' means the critical result corresponds to a minimum on the edge of the search domain; i.e., either on X1 or X2 or the internally preset limits on Xc.

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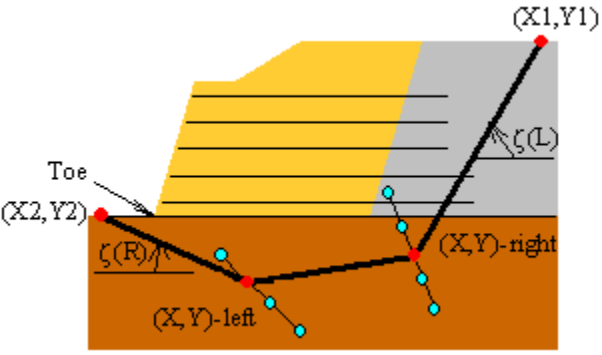
RESULTS OF 3-PART WEDGE ANALYSIS



Results in the table below represent the critical slip surface composed of a three-part wedge and identified by the specified points (X-left, Y-left) and (X-right, Y-right) and angles Zeta(L) and Zeta(R). ReSSA finds the (X,Y) coordinates, as well as the angles Zeta, based on user-specified search domain. The trace of the critical three-part wedge is fully defined by four points: (X1, Y1), (X-left, Y-left), (X-right, Y-right), (X2, Y2).

Critical 3-part wedge (Automatic search):

(X2, Y2) [ft]	Zeta(L) [degrees]	(X-left, Y-left) [ft]	(X-right, Y-right) [ft]	Zeta(R) [degrees]	(X1, Y1) [ft]	Fs
(34.83, 1.14)	20.00	(56.10, -6.60)	(97.10, -9.85)	35.00	(195.63, 59.14)	1.579

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CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.24
Critical Circle: Xc = 76.09[ft], Yc = 69.35[ft], R = 69.45[ft]. (Number of slices used = 77)

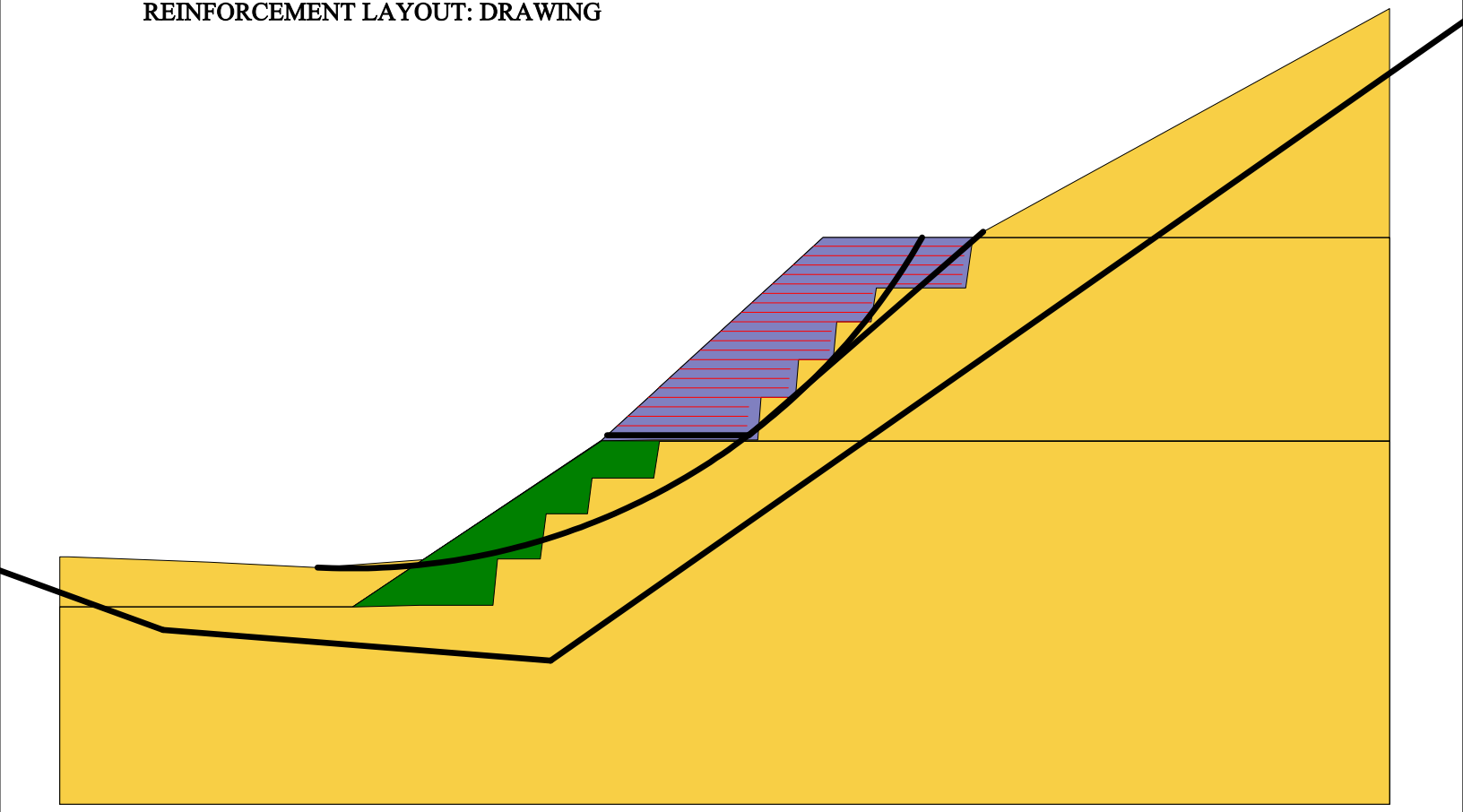
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Minimum Factor of Safety = 1.29
Critical Two-Part Wedge: (Xa = 103.08, Ya = 14.00) [ft]
(Xb = 118.10, Yb = 14.00) [ft]
(Xc = 142.86, Yc = 35.52) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 28.38 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.58
Critical Three-Part Wedge: (X2 = 34.83, Y2 = 1.14) [ft]
(X-left = 56.10, Y-left = -6.60) [ft]
(X-right = 97.10, Y-right = -9.85) [ft]
(X1 = 195.63, Y1 = 59.14) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 17.45 [degrees]

REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 2 4 6[ft]

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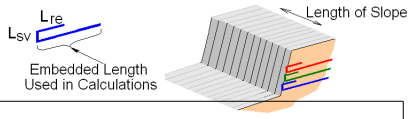

REINFORCEMENT LAYOUT: TABULATED DATA & QUANTITIES

Layer #	Reinf. Type #	Geosynthetic Designated Name	Height Relative to Toe [ft]	Embedded Length [ft]	Covergae Ratio, Rc	(X, Y) front [ft]		(X, Y) rear [ft]		Lsv * [ft]	Lre [ft]
1	1	Geosynthetic type #1	14.00	14.80	1.00	103.08	14.00	117.88	14.00	0.00	0.00
2	1	Geosynthetic type #1	15.00	13.70	1.00	104.17	15.00	117.87	15.00	0.00	0.00
3	1	Geosynthetic type #1	16.00	12.70	1.00	105.26	16.00	117.96	16.00	0.00	0.00
4	1	Geosynthetic type #1	17.00	11.70	1.00	106.35	17.00	118.05	17.00	0.00	0.00
5	1	Geosynthetic type #1	18.00	14.70	1.00	107.45	18.00	122.15	18.00	0.00	0.00
6	1	Geosynthetic type #1	19.00	13.70	1.00	108.54	19.00	122.24	19.00	0.00	0.00
7	1	Geosynthetic type #1	20.00	12.70	1.00	109.63	20.00	122.33	20.00	0.00	0.00
8	1	Geosynthetic type #1	21.00	11.70	1.00	110.72	21.00	122.42	21.00	0.00	0.00
9	1	Geosynthetic type #1	22.00	14.70	1.00	111.81	22.00	126.51	22.00	0.00	0.00
10	1	Geosynthetic type #1	23.00	13.70	1.00	112.91	23.00	126.61	23.00	0.00	0.00
11	1	Geosynthetic type #1	24.00	12.70	1.00	114.00	24.00	126.70	24.00	0.00	0.00
12	1	Geosynthetic type #1	25.00	11.70	1.00	115.09	25.00	126.79	25.00	0.00	0.00
13	1	Geosynthetic type #1	26.00	14.70	1.00	116.18	26.00	130.88	26.00	0.00	0.00
14	1	Geosynthetic type #1	27.00	13.70	1.00	117.27	27.00	130.97	27.00	0.00	0.00
15	1	Geosynthetic type #1	28.00	12.70	1.00	118.36	28.00	131.06	28.00	0.00	0.00
16	1	Geosynthetic type #1	29.00	11.70	1.00	119.46	29.00	131.16	29.00	0.00	0.00
17	1	Geosynthetic type #1	30.00	20.00	1.00	120.55	30.00	140.55	30.00	0.00	0.00
18	1	Geosynthetic type #1	31.00	19.00	1.00	121.64	31.00	140.64	31.00	0.00	0.00
19	1	Geosynthetic type #1	32.00	18.00	1.00	122.73	32.00	140.73	32.00	0.00	0.00
20	1	Geosynthetic type #1	33.00	17.00	1.00	123.82	33.00	140.82	33.00	0.00	0.00
21	1	Geosynthetic type #1	34.00	16.00	1.00	124.92	34.00	140.92	34.00	0.00	0.00

* Vertical distance between layers.

QUANTITIES

Reinf. Type #	Designated Name	Coverage Ratio	Area of reinforcemnt [ft²] / length of slope [ft]
1	Geosynthetic type #1	1.00	301.30

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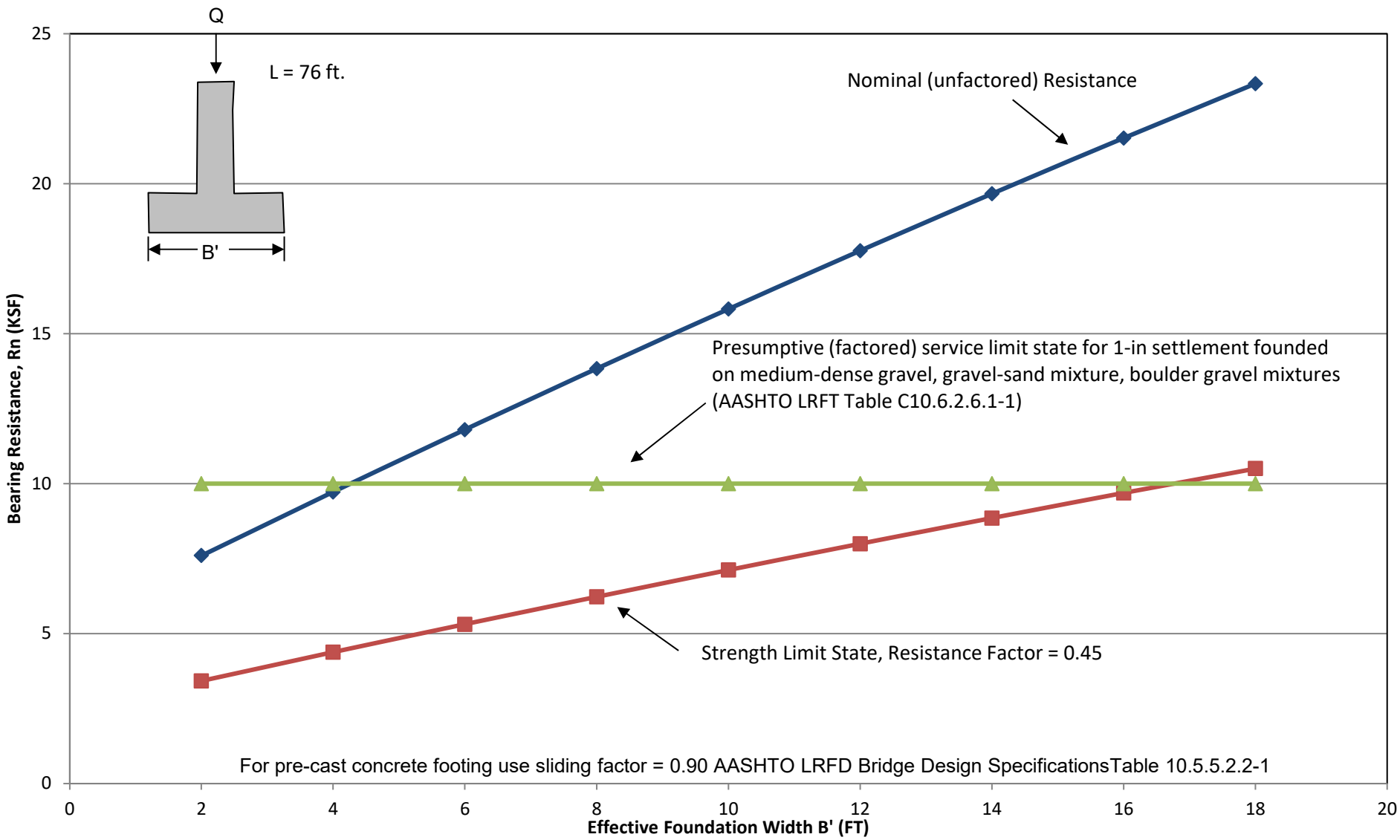
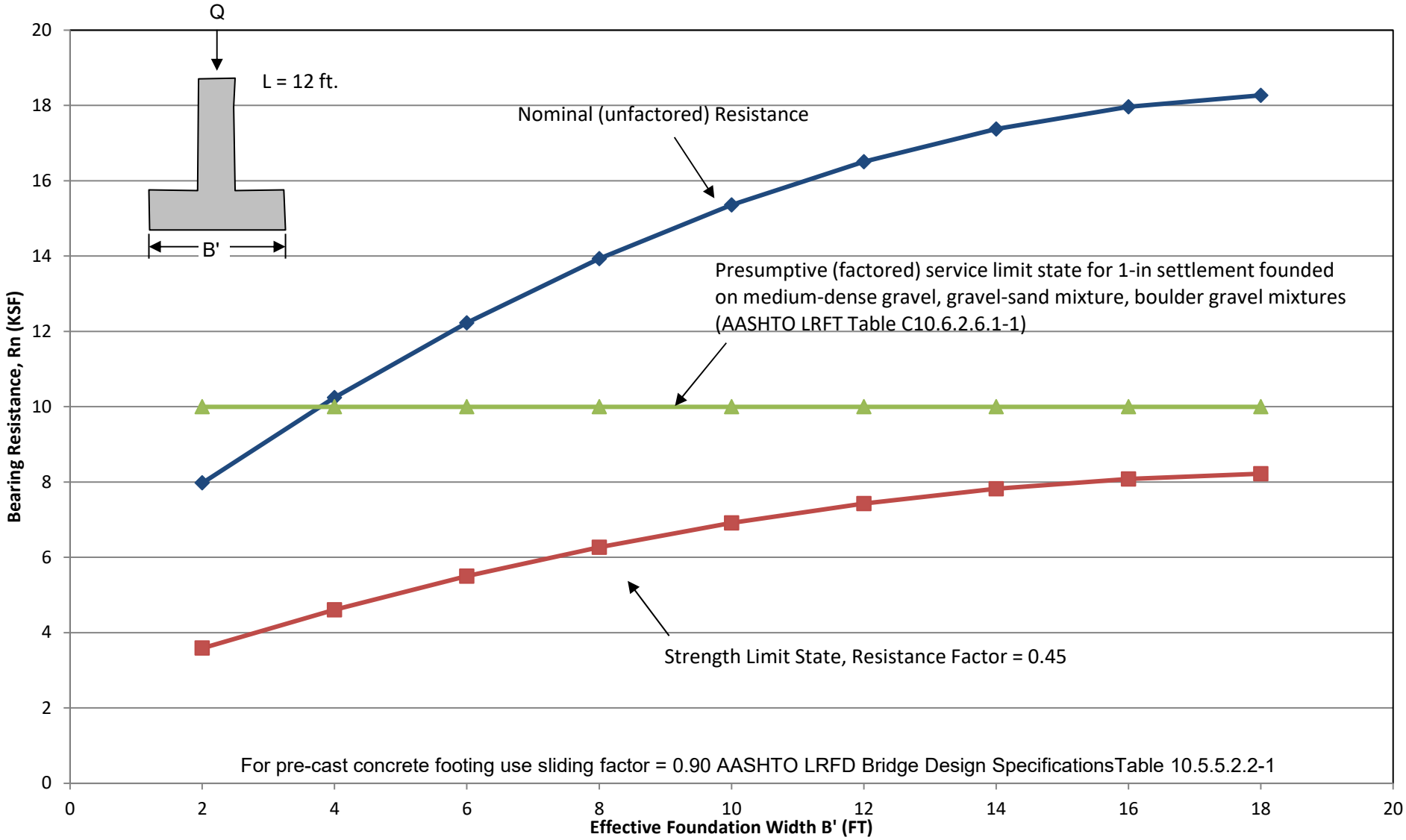
FALL CREEK ROAD – BEARING CAPACITY ANALYSIS

EFFECTIVE WIDTH vs. BEARING RESISTANCE (LRFD)
Box Culvert (Fall Creek Road)



EFFECTIVE WIDTH vs. BEARING RESISTANCE (LRFD)
Wing Walls (Fall Creek Road)