



COLORADO

Department of Transportation

CDOT Office of Applied Research



Assessing And Characterizing Geotechnical Soil Strength Parameters For A Systematic Process To Analyze Slope Stability in accordance with LRFD methods.

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July 2026
CDOT-2026-10

Technical Report Documentation Page

1. Report No. CDOT-2026-10	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle Assessing and Characterizing Geotechnical Soil Strength Parameters for a systematic process to analyze slope stability in accordance with LRFD methods. Soil Strength Parameters		5. Report Date July 2026	
		6. Performing Organization Code	
7. Author(s) Ben Arndt, PE Richard Johnson, PE		8. Performing Organization Report No.	
9. Performing Organization Name and Address Escalante CM and Engineering, Inc PO Box 40110 Grand Junction, CO 81504 Jeremy Lucero (Prime) RJ Engineering & Consulting, Inc 3082 Evergreen Parkway, Suite E Evergreen, CO 80439 Ben Arndt, PE, Richard Johnson, PE		10. Work Unit No. (TRAIS)	
		11. Contract or Grant No. 23-HAA-XB-00044	
12. Sponsoring Agency Name and Address Colorado Department of Transportation Office of Applied Research 2829 W. Howard Place Denver, CO 80204		13. Type of Report and Period Covered Final	
		14. Sponsoring Agency Code	
15. Supplementary Notes Prepare in cooperation with the US Department of Transportation, Federal Highway Administration.			
16. Abstract This research report reviews fine grained, clay soil slopes that result in landslides and embankment failures in Colorado and Wyoming. The landslides and embankment distress often require constant roadway maintenance, and full mitigation measures are often beyond roadway funding thresholds. Geotechnical site investigations and resulting slope stability analysis can be problematic with the type of laboratory samples collected and the basis for evaluating the existing stability and proposed mitigation. One method that has been used for the mitigation process of these unstable slopes is modeling of the existing failure condition of the landslide to a factor of safety (FS) of 1.0 known as "back-calculation" or "back analysis" to capture and model the site conditions at the time of slope failure. The report compares Atterberg testing from all the sites and suggests various strength parameters for back-analysis (baseline) stability analysis given the similarity in Cretaceous-age properties. The report reviews the use of both soil strength parameters of ϕ (friction angle) and c (cohesion) and how these properties relate to direct shear testing and the fundamental importance of using cohesion in back-calculated stability modeling to provide a useful model for comparing various mitigation options. The report also provides examples of back calculation techniques in these Cretaceous-age materials. The research also evaluates and discusses the use of LRFD methods applied and integrated into both slope and structural considerations for a reasonable level of mitigation. Suggestions for incorporating LRFD with slope stability programs and design protocols are discussed.			
17. Keywords Global Stability Analysis, Cohesion, Landslides, Cretaceous Clays, Embankment Failures, Atterberg Testing, Back Analysis, Back Calculation, LRFD and Slope Stability.		18. Distribution Statement https://www.codot.gov/programs/research	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages	22. Price

The contents of this report reflect the views of the author(s), who is(are) responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views of the Colorado Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

Acknowledgments

The researchers would like to acknowledge the CDOT Research Team and Panel who were invaluable in their support and contributions to the research paper.

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Executive Summary

This research report reviews fine grained, clay soil slopes that result in landslides and embankment failures in Colorado and Wyoming. The landslides and embankment distress often require constant roadway maintenance, and full mitigation measures are often beyond roadway funding thresholds. Geotechnical site investigations and resulting slope stability analysis can be problematic with the type of laboratory samples collected and the basis for evaluating the existing stability and proposed mitigation. One method that has been used for the mitigation process of these unstable slopes is modeling of the existing failure condition of the landslide to a factor of safety (FS) of 1.0 known as “back-calculation” or “back analysis” to capture and model the site conditions at the time of slope failure.

This report reviews and evaluates seventeen landslides and embankment failures that occurred in or near Cretaceous-age clay and claystone materials. Laboratory testing at the sites generally consisted of Atterberg Limits to classify the soil types. Based on the review, all the sites were in a relatively small range of index properties relative to each other suggesting that the Cretaceous geologic depositional environment created a generally uniform material from northern Wyoming to southern Colorado.

The report compares Atterberg testing from all the sites and suggests various strength parameters for back-analysis (baseline) stability analysis given the similarity in Cretaceous-age properties. The report reviews the use of both soil strength parameters of ϕ (friction angle) and c (cohesion) and how these properties relate to direct shear testing and the fundamental importance of using cohesion in back-calculated stability modeling to provide a useful model for comparing various mitigation options. The report also provides examples of back calculation techniques in these Cretaceous-age materials.

The research also evaluates and discusses the use of LRFD methods applied and integrated into both slope and structural considerations for a reasonable level of mitigation. Suggestions for incorporating LRFD with slope stability programs and design protocols are discussed.

Chapter 1. Introduction

1.1. Background and Purpose

Unstable fine grained soil slopes resulting in landslides and embankment failures are a common occurrence in Colorado and the Rocky Mountain West. For the Colorado Department of Transportation (CDOT) embankment distress and resulting landslides can impact public safety and often require ongoing roadway maintenance which can be costly over time. Furthermore, mitigation measures are often high dollar investments and require site investigation and slope stability analysis.

Cost for mitigation measures can vary greatly depending on the evaluation, analysis and mitigation techniques used in the design. Soil testing is typically conducted by obtaining Atterberg limits and/or direct shear samples. Unfortunately, in many instances, material sampling within the actual failure surface of the landslide or embankment failure cannot be obtained directly since these locations are commonly outside of the DOT right of way and/or on very steep slopes that are not accessible with conventional drilling and sampling equipment without specialty drilling methods.

One method that has been used for evaluation of these unstable slopes is modeling of the existing failure condition of the landslide to a factor of safety (FS) of 1.0. A FS of 1.0 is, by definition, the balanced ratio of the resisting forces to driving (loading) forces which occurs at the point of slope failure.

This “back analysis” or “back calculation” method (Mesri, 1993) is essential to describe and model the site conditions at the time of the slope failure based on creating a slope stability model from the physical evidence and slope characteristics at the site at the point of failure for FS of 1.0. The model is used to determine the contributing factors that caused the slide failure. This existing model is used as the baseline when comparing cost and effectiveness with other mitigation modeled options.

The modeling of the failure and appropriate FS method does require that various soil parameters including phi (internal friction angle, ϕ) and cohesion (c) of the soil be used to model the existing and mitigated systems. The use of even very small values of cohesion provides for a representative failure surface in the model to satisfy equilibrium of the model (Day, 1995). This method is invaluable when confronted with a limited amount of laboratory data due to inaccessibility challenges for site evaluation and investigation, funding limitations, rapid project delivery, and/or other constraints.

This report evaluates previously designed landslide mitigation projects and reviews the soil strength parameters with the corresponding geologic units (i.e. shales and claystone materials) to provide

insights when evaluating and designing mitigation for future landslides and embankment failures in similar subsurface materials.

Additionally, this report comments on the Load and Resistance Factor Design (LRFD) for landslide mitigation systems that consist of vertical shaft or pile elements. The structural elements typically have specified load factors that are much greater than the resistance (global stability) factors resulting in a much higher overall factor of safety.

Chapter 2. Project Sites and Geologic Formation

The research project has reviewed seventeen (17) past landslides that were evaluated, analyzed and mitigated with a major construction project between 2006 and 2025. One notable geologic similarity between all the landslide project sites is that they are in association with clay and/or claystone from the Cretaceous Period associated with a western interior seaway from a geologic time within the Cretaceous Period between about 140 to 65 million years ago (Deep Time Maps, 2023). These geologic dates are approximate and are subject to change and variations based on ongoing geologic classifications and evaluations.

2.1. Project Sites – Colorado and Wyoming

The approximate locations of the project sites in Colorado and Wyoming are shown in Figures 1 and 2. The figures are meant to depict the geologic diversity of both states and illustrate that the locations and outcrops of the Cretaceous-age clays are quite varied and geologically complex. Figure 1 depicts the six (6) sites in Colorado that consist of fourteen (14) total landslide mitigation projects the Colorado Department of Transportation (CDOT) undertook between 2006 and 2024. Figure 2 depicts the three (3) sites in Wyoming that consisted of three (3) total landslide mitigation projects that the Wyoming Department of Transportation (WYDOT) undertook between 2012 and 2025.

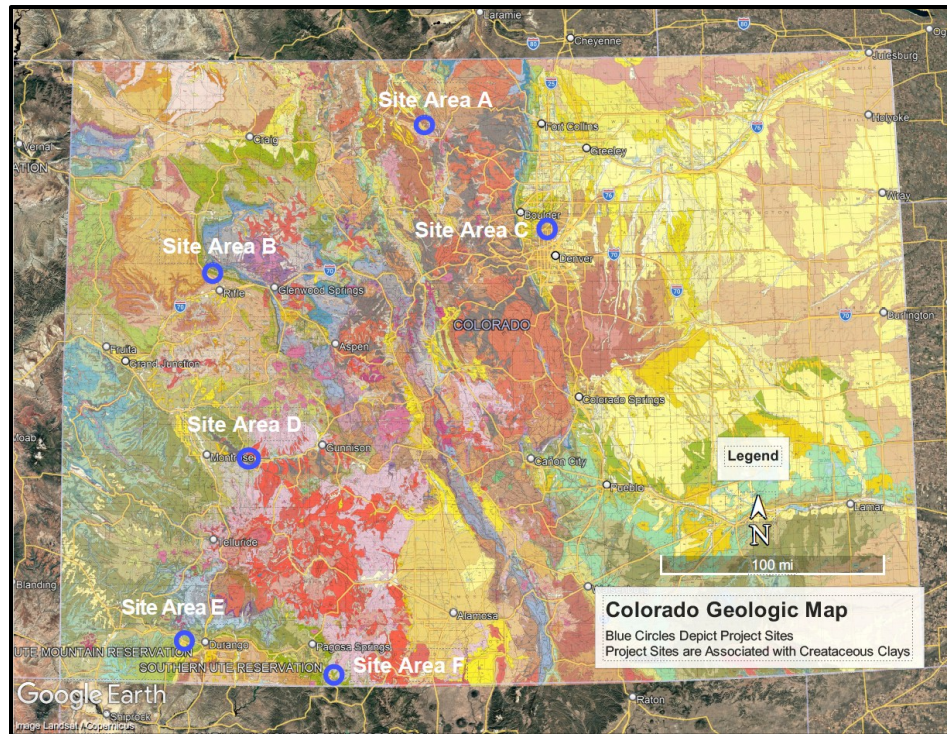


Figure 1. Colorado statewide map showing the general location of the landslide/embankment failure mitigation projects located in Cretaceous Clays. The geologic background conveys the complexity of the geologic conditions in the state.

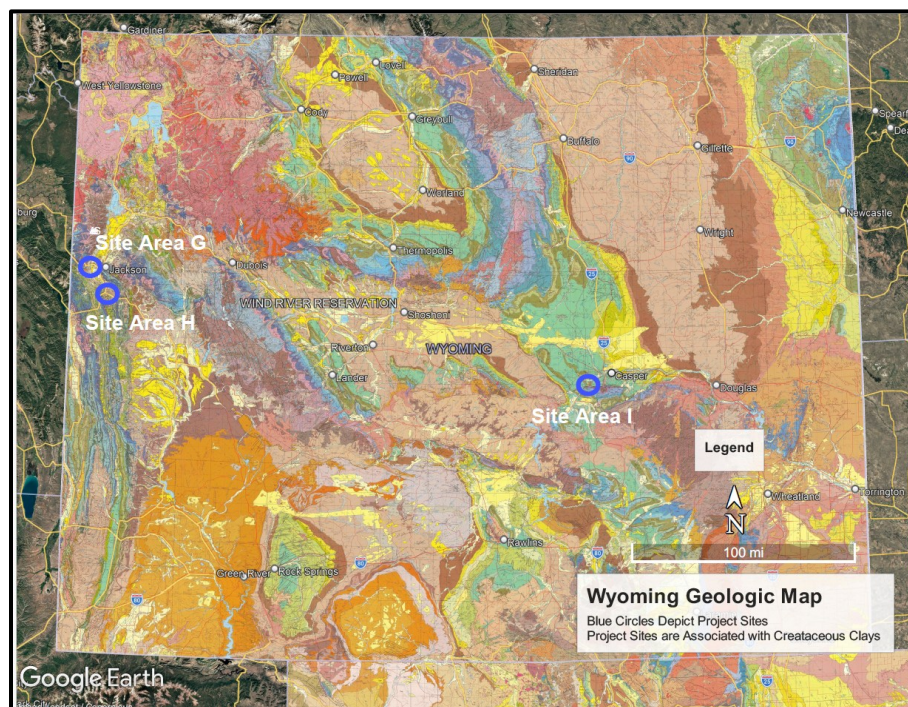


Figure 2. Wyoming statewide map showing the general location of the landslide/embankment failure mitigation projects located in Cretaceous Clays. The geologic background conveys the complexity of the geologic conditions in the state.

Table 1 depicts the project site areas and individual sites depicted on the previous figures. All sites are within the Cretaceous Period, and the table is an attempt to further categorize the sites by sub dividing the Cretaceous into time slices, however many of the areas and time slices are open to interpretation as the geology is so varied and complex. The attempt was to determine if there were any other geologic factors that might provide comparisons or differences with the project sites. All sites were marine or fluvial depositional environments or a combination of both in the geological historic record and all sites are associated with the western shore of the Western Interior Seaway depicted in Figure 3.

Table 1. Project Site Areas (Figures 1 and 2) subdivided into specific sites by Highway Milepost and Approximate Geologic Time Subdivision within the Cretaceous (*the ages and subgroups are interpreted from geologic maps).

Figures 1 and 2 - Site Areas	Highway	MP	Geology	Geologic Age (Subgroup)*	Geologic Age (Ma)*	Geologic Setting
A	US 40	156.7	Colorado Group	Cenomanian	88-100	Marine
B	CO 13	15.9	Mesa Verde / Williams	Campanian-Maastrichtian	75-80	Fluvial-Delta
C	US 36	50.05	Arapahoe	Maastrichtian	67-70	Fluvial
D	US 50	103.7	Mancos	Cenomanian - Campanian	72-100	Marine
	US 50	104.3	Mancos	Cenomanian - Campanian	72-100	Marine
	US 50	104.8	Mancos	Cenomanian - Campanian	72-100	Marine
	US 50	108	Mancos	Cenomanian - Campanian	72-100	Marine
	US 50	121.4	Mancos	Cenomanian - Campanian	72-100	Marine
	US 50	121.8	Mancos	Cenomanian - Campanian	72-100	Marine
	US 50	121.8	Mancos	Cenomanian - Campanian	72-100	Marine
E	US 160	69.8	Mesa Verde / Lewis	Campanian - Maastrichtian	67-83	Marine
	US 160	74.1	Mesa Verde / Lewis	Campanian - Maastrichtian	67-83	Marine
	US 160	75.1	Mesa Verde / Lewis	Campanian - Maastrichtian	67-83	Marine
	US 160	75.3	Mesa Verde / Lewis	Campanian - Maastrichtian	67-83	Marine
F	US 84	7.6	Mesa Verde / Lewis	Campanian - Maastrichtian	67-83	Marine
G	WY 22	12.8	Upper Cretaceous	Maastrichtian-Cenomanian	66-100	Marine
H	WY 191	141.6	Aspen/Bear River	Cenomanian	100-94	Brackish
I	WY 220	101.6	Cloverly / Mowry	Cenomanian	97-140	Marine-Fluvial

2.2. Discussion of the Cretaceous

The Cretaceous period that the landslides in this report generally occurred between approximately 140 to 65 million years ago and in the case of the landslides and embankment failures we have studied, generally consists of clay sediments in a shallow to deep sea-basin. This seaway is sometimes referred to as the Western Interior Seaway (Cretaceous Atlas, 2023) extended from southwestern Canada to Mexico. The Cretaceous period has been subdivided into more than 30 separate time slices within 140-to-65-million-year time period that split the time eras from Upper Cretaceous to Lower Cretaceous. (Cretaceous Atlas, 2023).

2.2.1. Geologic Depositional History

As described, the project sites in this research are located within Cretaceous deposits was based on review of USGS State Geology Maps and local geology. The essence of this research paper is to describe that the geologic age of the landslide sites reviewed were within a relatively narrow geologic time slice.

Figure 3 depicts a generalized view of the Western Interior Seaway, and the yellow dots depict the approximate locations of the landslides and embankment failures reviewed in this report. Figure 4 depicts a generalized cross-sectional setting of the geologic deposits and generalized landslide material sites studied in this report (Cretaceous Atlas, 2023).

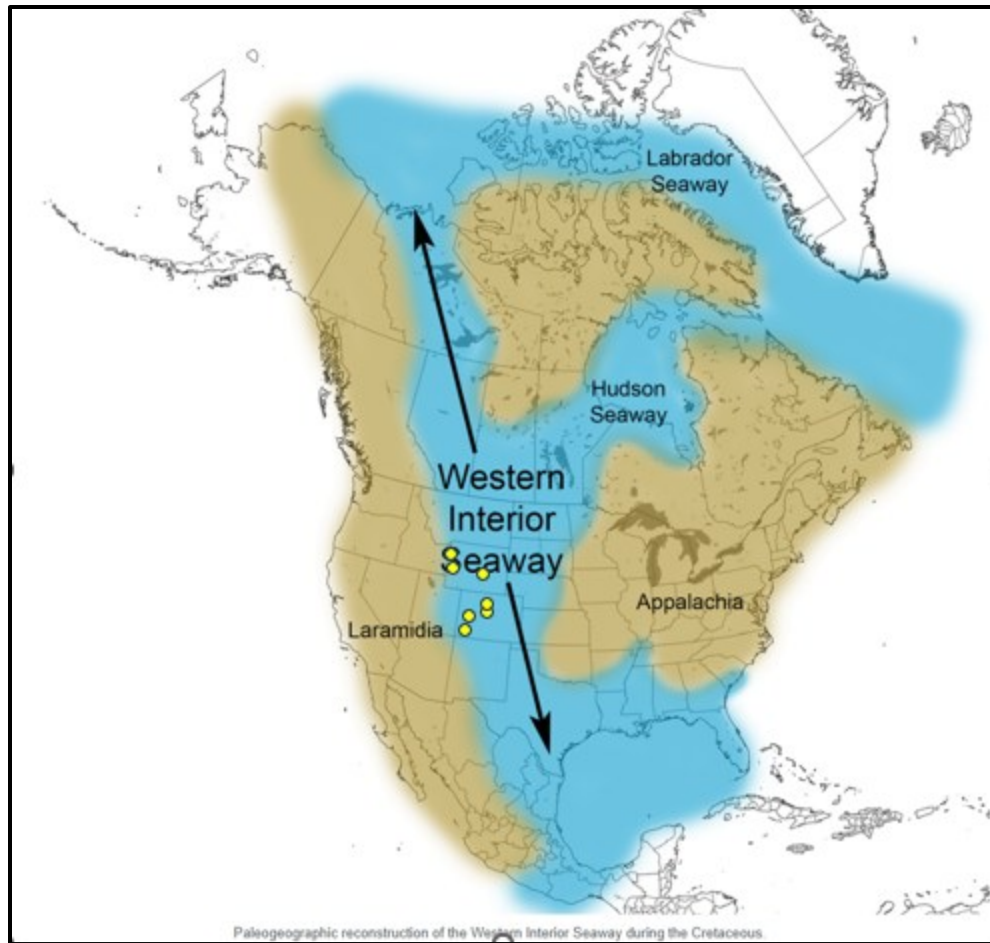


Figure 3 depicts a generalized view of the Western Interior Seaway and the yellow dots depict the approximate locations of the landslides and embankment failures reviewed in this report. (Work under Creative Commons NonCommercial International Licenses, <https://www.cretaceousatlas.org/geology/>).

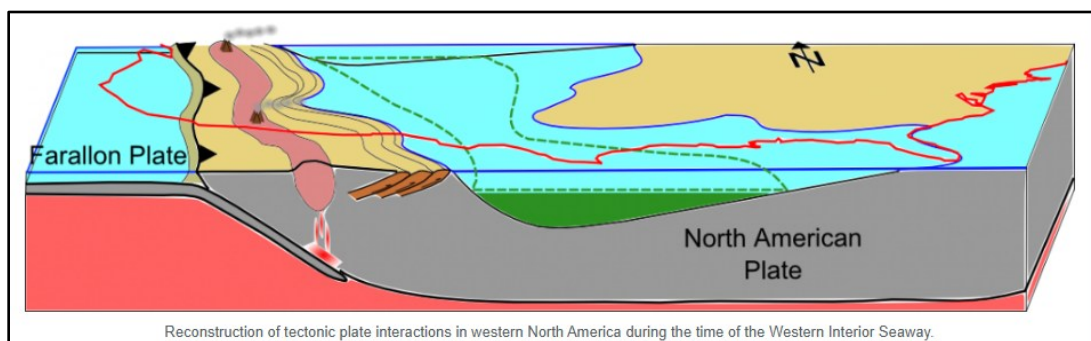


Figure 4 depicts a generalized cross sectional view of the Western Interior Seaway (west to east). (Work under Creative Commons NonCommercial International Licenses, <https://www.cretaceousatlas.org/geology/>).

Other literature with respect to Figure 3 (Fagel, 2007) describes how clays can be characterized by the depositional environment. Clay minerals are typically formed from precipitation in closed continental or

marine sedimentary basins known as authigenic clay or from weathering off the continent known as detrital clays. Typically, detrital clays are the most common. The intent of this study is to illustrate that geology and geologic depositional environments can have effects on the strength properties and that all the failures in this report are geologically within a similar geologic unit and history.

A few engineering index comparison takeaway items that seem reasonable to consider are the following:

- Consolidation magnitude. Cretaceous clays may have had hundreds or thousands of feet of overburden loading in the geologic past.
- Deep sea sediments can be deposited by two main processes (Fagel, 2007):
 - Authigenesis or in-situ precipitation in a closed sedimentary system
 - Detrital or weathering of the clay on the continent
- Both of these geological weathering processes are present in clay sediments that result in differing types of clays.
 - Detrital clays can be illite and kaolinite (less swelling nature of clays)
 - Authigenic clays can be more of a smectite variety (more swelling nature of the clays).

In general, the literature regarding strength testing of clays is categorized based on selected test results with no real discussion of geology or geological environmental other than a brief name or description with resulting strength properties in various publications. Also, the natural moisture content of a sample may not be provided in the literature making comparisons with various geologic properties more difficult.

2.3. Overview of Project Sites

The following is an overview of the project sites in this research study. Various selected field photos are provided to illustrate the failures prior to mitigation and photos during to illustrate the mitigation features.

2.3.1. Site Area A

US 40, MP 156.7, Site Area A was a regular maintenance problem for CDOT with pavement cracking resulting from slope instability over decades. Various attempts to avoid the landslide by moving the roadway alignment uphill of the failure were unsuccessful. Groundwater was also relatively high at this location adjacent to a lake. To mitigate the site a soldier pile and tieback wall was designed and constructed to support the two-lane roadway. Figure 5 depicts the partially completed project illustrating the insertion of ground anchor tiebacks into the soldier pile wall.



Figure 5. US 40 MP 156.7 depiction of installation of a tieback into the soldier pile lagging wall.

2.3.2. Site Area B

CO 13, MP 15.9, Site Area B consisted of a site that was a constant embankment fill problem for CDOT maintenance with cracking pavement and slope instability. The roadway stability issue began in 2005 with roadway cracking and vertical movement which accelerated and eventually the entire roadway in this section dropped vertically approximately 4 feet in just a few days. Groundwater was also relatively high. To mitigate the site a geofoam lightweight fill solution was designed and constructed to support the two-lane roadway. Figure 6 depicts the toe of slope which failed as a bearing/slide over-rode surface material below. Figure 7 depicts the vertical movement that occurred in just a few days. Figure 8 depicts the placement of geofoam at the site. These selected geofoam projects have been published in (Arndt, 2016).



Figure 6. CO 13 MP 15.9 depicts toe of slope which failed as a bearing/slide and over-rode materials.



Figure 7. CO 13 MP 15.9 depicts the vertical movement that occurred in just a few days

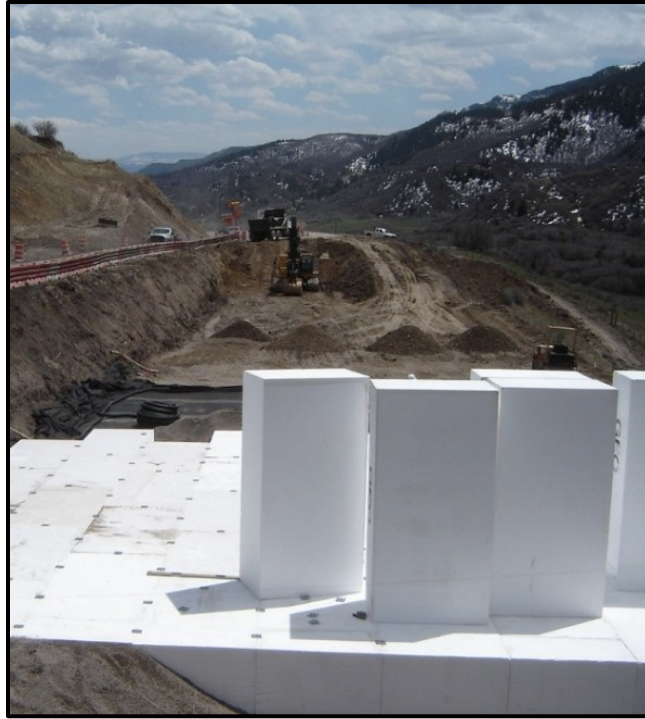


Figure 8. CO 13 MP 15.9 depicts the placement of geofoam at the site.

2.3.3. Site Area C

US 36 MP 50.05, Site Area C consisted of a site that was a recently constructed MSE wall on a clay foundation. The stability issue in 2019 began essentially overnight with pavement cracking of a major highway between Denver and Boulder (US 36) and eventually the entire roadway in this section dropped vertically approximately 10 feet in about a week. Figures 9 and 10 depict the rigid pavement cracking that occurred in the summer of 2019. Figure 11 depicts the failure toe materials overriding lower sections of the slope. This project was written up (Arndt, 2021).



Figure 9 depicts the rigid pavement cracking that occurred in the summer of 2019.



Figure 10. US 36 MP 50.05 depicts the rigid pavement cracking that occurred in the summer of 2019.



Figure 11 US 36 MP 50.05 depicts the toe of the slope that was failing in bearing/stability, and the toe was overriding the lower surface.

2.3.4. Site Area D

Site Area D consists of six landslide/embankment/wall failure sites along US 50 between MP 103.7 and 121.8. Much of the area is mapped as landslide deposits on USGS maps and is underlain by the Mancos Shale which is a predominate shale in western Colorado. The mitigation options along the corridor consist of geofoam embankment stabilization, micropile foundations for failing block walls and near the eastern end of the corridor a couple shear shaft system that will be mentioned for other locations in this research report. Figures 12 through 15 depict various sections of the roadway corridor that was mitigated. It should be noted that the roadway along US 160 (Site Area E) is very similar to geofoam mitigation repairs to the US 50 (Site Area D) corridor roadway and are essentially very similar. These selected Geofoam projects have been published in (Arndt, 2016).



Figure 12. US 50 MP 104.3 depiction of failed block faced wall.



Figure 13. US 50 MP 104.3 depiction of micropiles to support failed block faced wall.



Figure 14. US 50 MP 108 depiction of desiccated clay materials underneath the roadway.



Figure 15. US 50 MP 108 depiction of construction of geofabric mitigation stabilization.

2.3.5. Site Area F

Site Area F consisted of a landslide/embankment failure along US 84 between MP 7.6. Site Area F consists of a structural mitigation option using paired drilled shafts to mitigate the landslide area. Figures 16 to 17 depict the failing area and construction of the paired drilled shafts.



Figure 16. US 84 MP 7.6 depiction of failing roadway shoulder at landslide site.



Figure 17. US 84 MP 7.6 depiction drilled shafts to stabilize roadway using shafts that are coupled together in pairs.

2.3.6. Site Area G

Site Area G depicts the Teton Pass embankment failure that occurred in 2024 due to higher-than-normal groundwater conditions resulting from rapid snowmelt. Figures 18 through 20 depict the failure and the final mitigation photos. The project was written up in an ASCE publication (Arndt, 2026).



Figure 18. WY 22 MP 12.8 depiction beginning of embankment landslide failure in 2024.



Figure 19. WY 22 MP 12.8 aerial view of embankment landslide failure in 2024.



Figure 20. WY 22 MP 12.8 aerial view of embankment stabilization in 2025.

Chapter 3. Review of Soil Strength Characteristics

Clay materials are considered fine-grained and can present various characteristics in the field. The degree of water saturation combined with the fineness of the clay particles have great influence on the nature and character of the clay materials. From an engineering perspective there are various laboratory methods and classification systems to categorize the clays. This report uses the Unified Soils Classification System (USCS) system. A brief overview is provided in the following sections.

3.1. Atterberg, Moisture Content, and Consolidation of Clays

Atterberg Limits are the prevailing engineering soil classification method to characterize the type of clay at various water contents. The three main types of classification limits are liquid limit (LL), plastic limit (PL), and plasticity index (PI). The required parameters to calculate these limits are the natural moisture percentage of the material and how the material is classified after the specific test methods to determine LL, PL and PI which are well described in the literature. The LL consists of the water content in which a clay soil sample transitions from a liquid type of behavior to a plastic type of behavior. The PL is the water content in which a clay soil sample transitions from a plastic type of behavior to a semi-solid type of behavior. The PI of a clay soil sample depicts the range of moisture contents in which the soil behaves like a plastic material. The PI represents how sensitive the soil is to water content with a typical range from 0 that signifies non plastic to greater than 35 which would be highly plastic and prone to shrink and swell depending on the water conditions.

Another type of soil behavior can be presented as the liquidity index (LI).

$$LI = \frac{W - PL}{LL - PL}$$

- Where PL is the Plastic Limit
- LL is the liquid limit
- w is the natural moisture content

LI can be used in clays to relate the natural water content of a soil to the plastic and liquid limits. This type of behavior can then provide an indication of the soil's consistency and sensitivity potential changes in moisture content. Table 2 illustrates the definitions of LI.

Table 2. Liquidity Index (LI) definitions.

LI = 0	Soil is at its plastic limit
LI = 1	Soil is at its liquid limit
LI > 1	Soil is in a liquid state
LI < 1	Soil is in a solid state

An LI of zero or near zero indicates a soil that is at its plastic limit. Additionally, the LI of a material can lend insight into the consolidation history of the materials with a LI less than or equal to 1.0 indicating an over-consolidated material and a LI greater or equal to 1.0 indicating a normally consolidated material. More detailed descriptions of the above items can be found in FHWA NHI-16-072 (Loehr, 2016).

3.2. Strength Testing of Clays

The strength testing of clays is a much-discussed topic that has volumes in the literature search. For the purposes of this report the following strength testing of clay topics are considered relevant to the review and research of the landslides and landslide mitigation:

- The type of clay shear testing method.
- The consolidation state of the clay.
- The orientation of the clay bedding planes and geologic depositional features.
- The sample locations in reference to the actual landslide failure plane.
- The saturation of the clay sample undergoing testing.

3.2.1. Drained vs Undrained Testing of the Sample

Prior to describing the various clay shear strength methods, it is necessary to describe drained vs undrained testing. As implied, the difference between drained and undrained testing is how pore water pressure dissipates in the test sample. Drained conditions imply the pore water drains and dissipates not affecting the general outcome of the soil particle interactions (effective stress condition) whereas undrained conditions indicate that pore water pressure may build since the water cannot dissipate and may affect the soil particle interactions (total stress analysis). Direct shear and ring shear test are generally considered drained tests. The triaxial test can be performed as a drained or undrained test.

3.2.2. Consolidation/confinement Testing of the Sample

Also necessary is a brief description of consolidation and confinement during testing. Consolidation and/or confinement consists of either one- dimensional or three-dimensional confinement of the sample. Direct shear and ring shear test are generally considered a one-dimensional

consolidation/confined test. The triaxial test can be performed as a three-dimensional consolidation/confinement test or as an unconfined test.

3.2.3. Shear Strength Testing of Clay Materials

There are three (3) general clay shear strength testing methods:

- Direct Shear Testing (One-dimensional consolidation that is drained)
- Ring Shear Testing (One-dimensional consolidation that is drained)
- Triaxial Testing (Unconfined or three-dimensional consolidation that is drained or undrained)

Detailed descriptions of the above testing are described in FHWA NHI-16-072 (Loehr, 2016) and summarized here. There are pros and cons to the above testing and limitations for each test that are discussed in the publication. With respect to this research effort the primary shear testing results that were obtained based on review of the data set were only from Direct Shear Testing. No ring shear or triaxial testing was reported or completed for any of the projects involved

3.2.3.1. Direct Shear Testing

Direct shear testing of clay soil is a common practice in Colorado and Wyoming and can be performed by many soil testing firms. One field sampling method that streamlines direct shear testing, is to obtain samples with a California Sampler. The sample liner is a 2-inch circular outside diameter brass sleeve that is 4 inches in length. The sleeves are stacked in the sampling operations and obtain a relatively undisturbed sample with the associated blow counts. After sampling, the clay samples can be extruded from the sleeve and placed in a direct shear apparatus for testing at the approximate density and moisture in which it was obtained. Direct shear tests are relatively inexpensive and relatively quick turnarounds.

3.2.3.2. Ring Shear Testing

The use of ring shear testing is not common in Colorado or Wyoming, and samples typically are sent to specialty soils labs that are out of state. The ring shear also requires a relatively large sample size that may need to be recompacted, and moisture conditioned to approximate the site conditions. In practice it typically is not feasible to obtain a representative sample to test in the field when dealing with steep or inaccessible slopes consistent with landslides or embankment failures. Certain instances may lend itself to obtaining a sample in the field, but it needs to be an easily accessible site.

3.2.3.3. Triaxial Testing

Triaxial testing is performed in Colorado and to a less degree in Wyoming, although the cost and time turn around makes these tests only used for projects that have special geotechnical concerns and long

durations in which to turn around the sample results. There are many benefits from triaxial testing however, it is problematic to obtain representative residual strength values due to limitations with the relatively small displacements associated with the triaxial apparatus required to fully evaluate a residual shear strength.

3.3. Discussion of Direct Shear Peak and Residual Conditions

Direct shear testing for over consolidated clay samples for landslides is generally performed to determine a peak strength and a residual strength of the clay materials at three (3) confining pressures determined based on the depth of the slide plane. However, it is very rare to perform the direct shear test along the actual slide plane within a landslide for many reasons which include:

- The slide plane can be at a high angle to the vertical boring sampler, so the sample is not normal to the slide plane unless the boring is centered within the landslide failure mass.
- Failure slide planes are typically only an inch or less in thickness, so it is incredibly difficult to determine the slide plane from samples, especially if logging samples are only taken every 5 or 10 feet. Also, the limited slide plane thickness limits the amount of material that can be tested accurately and repeatedly.
- Continuous samplers can be employed although these may require specialized drilling skills and evaluation of the sample material may be difficult to assess where the actual failure plane may be located or multiple failure planes may be indicated in the sample.
- Many geotechnical laboratories do not adequately shear the sample multiple times to obtain a representative residual strength, and the test results may indicate strength parameters that are artificially high. In general practice it is best to perform at least three (3) shear sets on a sample and review the results to verify an actual residual value is being obtained.

Details regarding the direct shear testing can be found in FHWA NHI-16-072, 2016. Suffice to say that for useful evaluation of the shear strength parameters three (3) peak tests and three (3) residual tests are necessary to plot the shear strength properties of the material.

3.3.1. Discussion of Fully Softened Shear Strength

Much of the literature discusses fully softened soil conditions. Typically, the fully softened condition is associated with first time landslides and/or where clay particles are thought to be randomly oriented. The residual condition is the lowest strength the solid is thought to achieve after larger displacements and use used for ongoing or reactivated landslides. Details regarding the fully softened condition can be found in FHWA NHI-16-072 (Loehr, 2016).

3.3.2. Discussion of Peak and Residual

For discussions pertaining to the peak and residual shear strength testing, the peak strength is often defined as the condition where the maximum shear strength is achieved. Residual strength is a term that represents the soil strength at significant displacement/movement. The residual shear strength often cannot be reasonably measured in a triaxial test, and a direct shear or torsional ring shear device is used to generate the needed displacements since triaxial displacements magnitudes are limited (FHWA NHI 16-072, 2016). Figure 21 below depicts the results of a generalized direct shear test in which the peak and residual values are plotted on a graph and can be used to determine phi and cohesion.

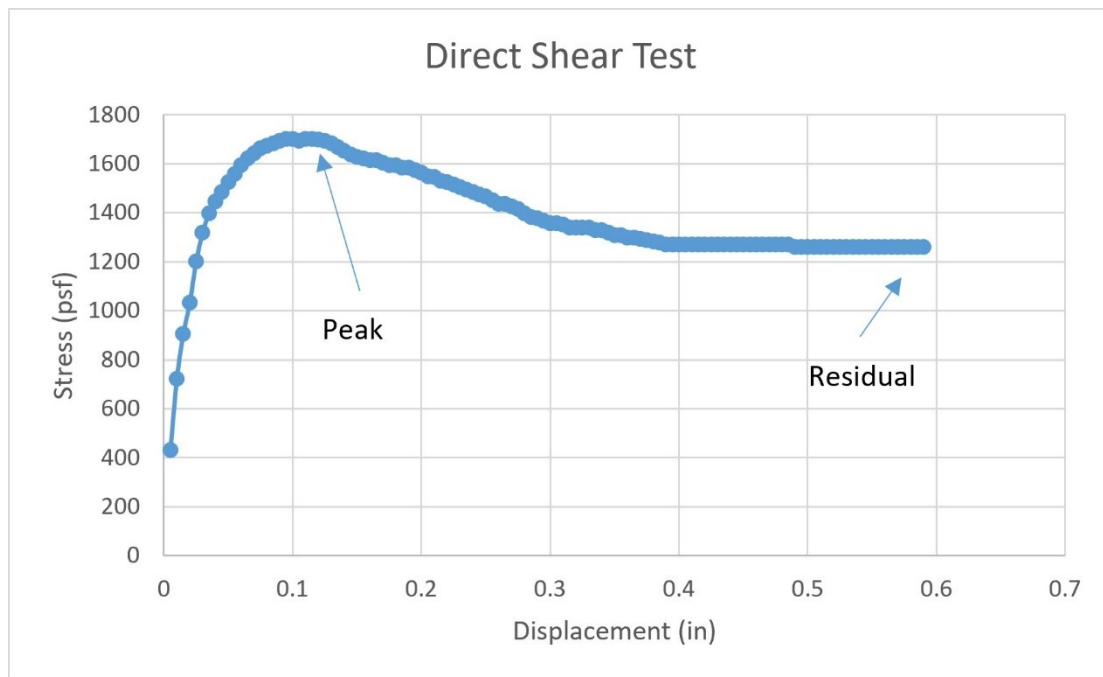


Figure 21. Typical direct shear test for one consolidation pressure of an over-consolidated clay sample. Note that there is a peak value at the top of the curve and then it levels out depicting more of a residual type of linear behavior.

Based on the stress and displacement results the peak and residual strength parameters can be plotted on shown in Figure 22.

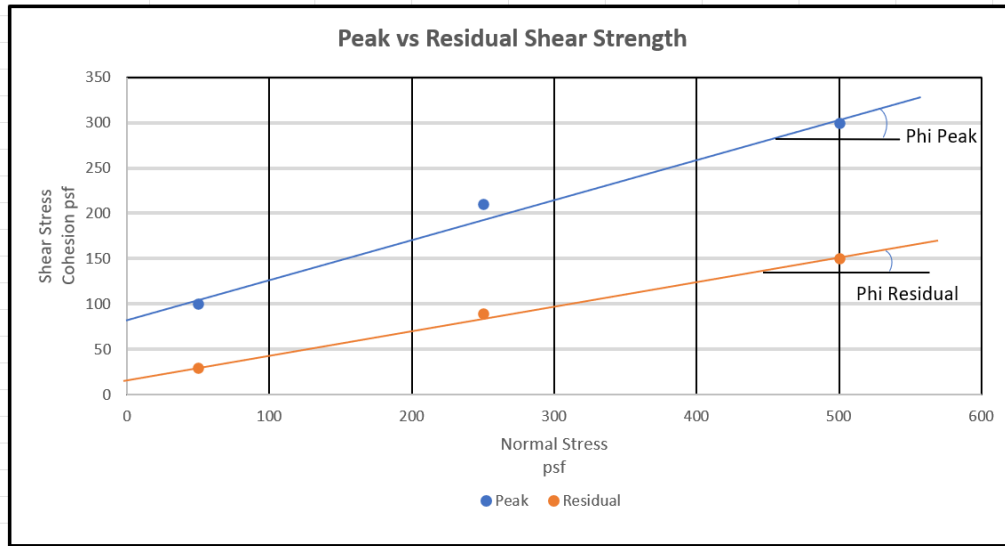


Figure 22. General depiction of Peak vs Residual Shear Strength Graph.

Chapter 4. Literature Review Testing of Clay Materials

A literature review was conducted to provide a background in evaluating subsurface materials associated with the project sites. The literature review covers testing, testing methods, testing assumptions, saturation, and publications regarding testing of clay materials. There have been countless studies on soil and soil testing, and the intent of this research is to highlight specific testing and testing studies that are relevant and pertain to the project sites for this study.

4.1. Shear Strength Testing of Clays

The following publications described below are notable publications that reference shear strength testing of clays.

4.1.1. Skempton 1964 and 1972

Skempton authored multiple publications between 1964 and 1972 regarding clay strengths and laboratory testing. In the 1964 publication Long Term Stability of Clay Slopes (Skempton, 1964), Skempton discusses normally consolidated and over consolidated clay materials strain softening and residual strengths. In a later 1972 publication describing a landslide at Walton's Wood, (Early & Skempton, 1972), the paper discusses shear testing of landslide materials. Both papers discuss the results of the Walton's Wood landslide and the back calculation that was performed. The 1964 publication also compares over consolidated and normally consolidated tests and suggests that the cohesion is zero for normal consolidated clays for both the peak and residual tests. However, in the 1964 publication Skempton discusses that direct shear test results show that the cohesion is very small and "probably not significantly different from zero".

The 1972 publication provides further detail into Walton's Wood Landslide and cross sections with geology and slope stability were provided (Skempton, 1972). Direct shear testing was reportedly conducted on the field failure surfaces, and a summary of the shear test results were provided.

The peak failure was reported to have phi angle of 21 degrees and cohesion of 2.2 psi; however, the residual strength was reported to have a phi angle of 13 to 15 degrees with zero cohesion. The slide mass was modeled with the residual strength values with cohesion at zero, for a FS of 1.0.

4.1.1.1. Commentary

The 1964 publication appears to be one of the first in evaluating normal and over consolidated clays.

The 1972 publication appears to be one of the first back calculation models where shear strength testing

was performed on what was believed to be the failure surface and then compared with slope stability programs of the day to provide a back-calculation. Both publications show the effective pressure vs the shear strength for both the peak and residual strengths. However, given that the papers were published in an era prior to the use of computers and modeling software it is difficult to reproduce the regression lines and the stability analysis contained in the articles precisely.

For example, the graphs provided effective earth pressure vs the shear strength regression and shear strengths results, however, when approximated into an Excel file to calculate the regression analysis the peak values are close but not duplicated.

- Peak Strength in the 1964 and 1972 publication is shown to be $\phi' = 21$ deg and $c' = 320$ psf. A spreadsheet approximation based on the factual data provided calculates a Peak Strength $\phi' = 18.6$ deg and $c' = 416$ psf.
- Residual Strength in the 1972 publication is shown to be $\phi' = 13$ deg and $c' = 0$ psf. A spreadsheet approximation based on the factual data provides a Residual Strength $\phi' = 11.4$ deg and $c' = 43$ psf. If the trendline for the Residual Shear is forced to zero, then result is $\phi' = 12.2$ deg and $c' = 0$ psf.
- Stability modeling a cross section also suggests that the $\phi' = 13.0$ to 15.0 deg with $c' = 0$ psf for a Factor of Safety (FS) of 1.0.
- A LE modeling effort with current software provides that if $\phi' = 13.0$ to 15.0 deg is used with $c' = 0$ psf, it is not possible to get a FS of 1.0 but rather a FS < 0.9 with a forced initiation and termination point, and a FS < 0.76 if the initiation and termination points are not forced.
- To truly create a similar failure surface as the 1972 publication it is necessary to use $\phi' = 15.0$ deg with a $c' = 84$ psf in current software models.

4.1.2. Skempton 1970

Skempton discusses in First Time Slides in Consolidated Clays (Skempton, 1970) shear testing of over consolidated clay materials and discussed strain softening and residual strengths. Skempton depicts generalized graphs where peak over consolidated clays have cohesion, but then fully softened or residual clays have zero cohesion.

4.1.3. Skempton 1985

In the 1985 publication Residual strength of clays in landslides, folded strata and the laboratory (Skempton, 1985) discussed residual strength conditions. The paper describes various clays and landslides with residual Normal Effect Stress vs Shear Strength graphs.

4.1.3.1. Commentary

As with the previous 1962 and 1972 publications, the Walton's Wood Landslide also has a different Residual Strength graph.

- Residual Strength in the 1985 publication is $\phi' = 13.2$ deg and $c' = 0$ psf.
- A modern spreadsheet approximation based on the factual data provided a Residual Strength $\phi' = 13.2$ deg and $c' = 108$ psf. If the trendline for the Residual Shear is forced to zero, then result is $\phi' = 12.8$ deg and $c' = 0$ psf.

4.1.4. Duncan and Stark 1992

Stark has authored or coauthored multiple publications between 1992 and 2010 regarding clay strengths and laboratory testing. In the 1992 publication the paper addresses stability and performance of slopes and embankments (Duncan and Stark, 1992) in which back analysis to evaluate soil strengths is partially discussed. Conclusions in the report suggest back analysis is a useful procedure for determining soil strengths from slope failures. It also suggests that the best procedure appears to be to assume the value of ϕ , using good judgement and whatever experience can be brought to bear, and calculate the value of cohesion that corresponds to a FS of 1.0.

4.1.5. Stark 2005

Stark in the paper Drained Shear Strength Parameters for Analysis of Landslides (Stark, 2005) suggests that for the residual strength value of cohesion be set equal to zero in stability analysis. This is supported by various means in the article and refers to previous Skempton publications. The apparent main reason for cohesion equal to zero is the assumption that in pre-existing shear surfaces that are slickensided the strength is controlled by frictional resistance of face-to-face contacts on the oriented surfaces and should only be represented by a friction angle not cohesion. However, there was one comment suggesting that the value of cohesion should be zero unless back calculation of local case histories suggest a value greater than zero. (Mesri and Abdel-Ghaffar, 1993).

4.1.6. FHWA Publication 2016

FHWA NHI-16-072 (Loehr, 2016) Geotechnical Site Characterization provides overview of laboratory testing. The publication suggests if "direct-residual shear tests are performed at different effective normal stresses; a residual shear strength envelope can be established as illustrated in Figure 7-47 of that publication. Residual shear strength envelopes generally have negligible cohesion intercept, and friction angles that are often substantially less than values established for peak strengths". The publication then provides for a comparison of the peak and residual strength envelopes from direct residual test with peak and residual regressions.

4.1.6.1. Commentary

As with the previous commentary on the Skemptions' graphs, it appears the FHWA publication also has forced the residual regression to show a $c=0$ psf.

- Figure 7-47 in that publication shows a Peak Strength of $\phi'=19.5$ deg and $c=800$ psf. A spreadsheet approximation based on the factual data provided a Peak Strength $\phi'=19.5$ deg and $c = 743$ psf.
- Residual Strength in the publication shows a $\phi'=13.5$ deg and $c'=0$ psf. A spreadsheet approximation based on the factual data provided a Residual Strength $\phi'= 10.3$ deg and $c = 252$ psf. If the trendline is forced to zero, then the Residual Strength is $\phi'= 13$ deg and $c'=0$ psf.

4.1.7. Day 1995

Cohesion Intercept in Effective Stress Analysis (Day, 1995) is a publication in which the authors studied the effect of cohesion on the stability analysis of clay slopes and embankments. The report does review the Skempton publications prior and does provide this suggestion that any analyses of stable slopes as well as back analyses of landslides, including those involving relatively high plasticity and high clay content soils, require "some magnitude of c' to satisfy equilibrium" as discussed previously.

4.1.7.1. Commentary

The concept that some magnitude of c' (cohesion) is required to satisfy equilibrium supports this research effort. Our experience suggests some cohesion, however minor, is necessary to accurately model landslide and embankment failure field conditions.

4.2. Saturation of Clays

The actual strength of a soil in relation to the saturation of the soil did not appear to be a large topic item in the literature review. However, there was one publication that did provide insight into this topic.

4.2.1. Gong 2022

The publication of the *Statistical analysis on the relationship between shear strength and water saturation of cohesive soils* (Gong, 2022) provides discussion on strength of partially and fully saturated materials. The paper reported that more than 2000 sets of data were collected for friction and cohesion and compared to the degree of saturation. Takeaways from the study as it applies to this research paper include:

- The paper characterized cohesion for various levels of saturation.

- The change in the degree of saturation has a significant effect on the shear strength with respect to cohesion.
- The degree of saturation has more influence on the decrease of cohesion compared with that of the friction angle.

The article describes that soil strength does become small or approach zero in a fully saturated state especially for cohesive soils but that most sliding surfaces for water induced landslides are above the water table in which the failing soils are unsaturated.

The conclusions suggest:

- In the study, 2040 and 2119 sets of experimental data of cohesive soils are collected for cohesion and friction angle, respectively, which are adopted to establish a database of shear strength versus degree of saturation with cohesion in a range of 1.0 kPa ~ 255.8 kPa (20.8 psf to 5000 psf) at a mean of 41.8 kPa (872 psf) and friction angle ranging from 3° to 57° at a mean of 23.0°.
- The change of degree of saturation plays a significant role in shear strength of cohesive soils; both mean cohesion and mean friction angle decrease with the increasing degree of saturation.

4.2.1.1. Commentary

The report provides table for the cohesion and friction angle of clay at various saturation percentages with shear strength values.

- For the c (cohesion) between a saturation of 90-100% the c values are 3.8 to 61 kPa (80 to 1273 psf) with a mean of 18.5 kPa (376 psf).
- For a saturation between 90-100% the ϕ values reported varies from 4 to 30.6 degrees with a mean of 13.3 degrees.
- The smallest cohesion reported is over 80 psf and none of the data sets report zero cohesion.

Chapter 5. Laboratory Testing at the Project Sites

The main intent of this research was to review and evaluate the laboratory data from the project sites based on a review of project plans and reported data. All the project sites had geotechnical drilling conducted and the results of the laboratory testing are as follows:

- Atterberg testing was conducted for all the sites and general Atterberg indices were provided.
- Some sites did not record natural moisture contents from the samples collected and therefore it is not possible to calculate the void ratio of the materials for use in the overall evaluation, but it can be approximated.
- Direct shear testing was only completed on a few of the sites and in some cases only the peak values were reported. Residual values were typically not reported.
- Triaxially testing was not completed on any of the sites.
- Specific void ratio testing was completed for the US 36 site and calculation using void ratio for partially saturated conditions resulted in similar void ratio values from 0.35 to 0.55

5.1. Atterberg and Saturation Discussion of the Project Sites

Each project site with available data and information was evaluated where the natural moisture of the soil was compared to the plastic limit of the soil. In most instances there were geotechnical borings within the presumed landslide or embankment failure that showed the natural moisture content was greater than the plastic limit. If there were borings outside the presumed unstable slope area, the samples typically showed that the natural moisture content of the soil was less than the plastic limit.

For example, in Figure 23, the blue highlighted borings indicated higher natural soil moistures in the range from 21% to 24% whereas the other orange highlighted borings indicated natural soil moistures in the range of 10% to 20%. The plastic limits for both sets of borings ranged from 18% to 25% and did not indicate a significant distinction. Also, the calculated average saturation for the blue highlighted borings was 97% whereas the orange highlighted borings was 79%.

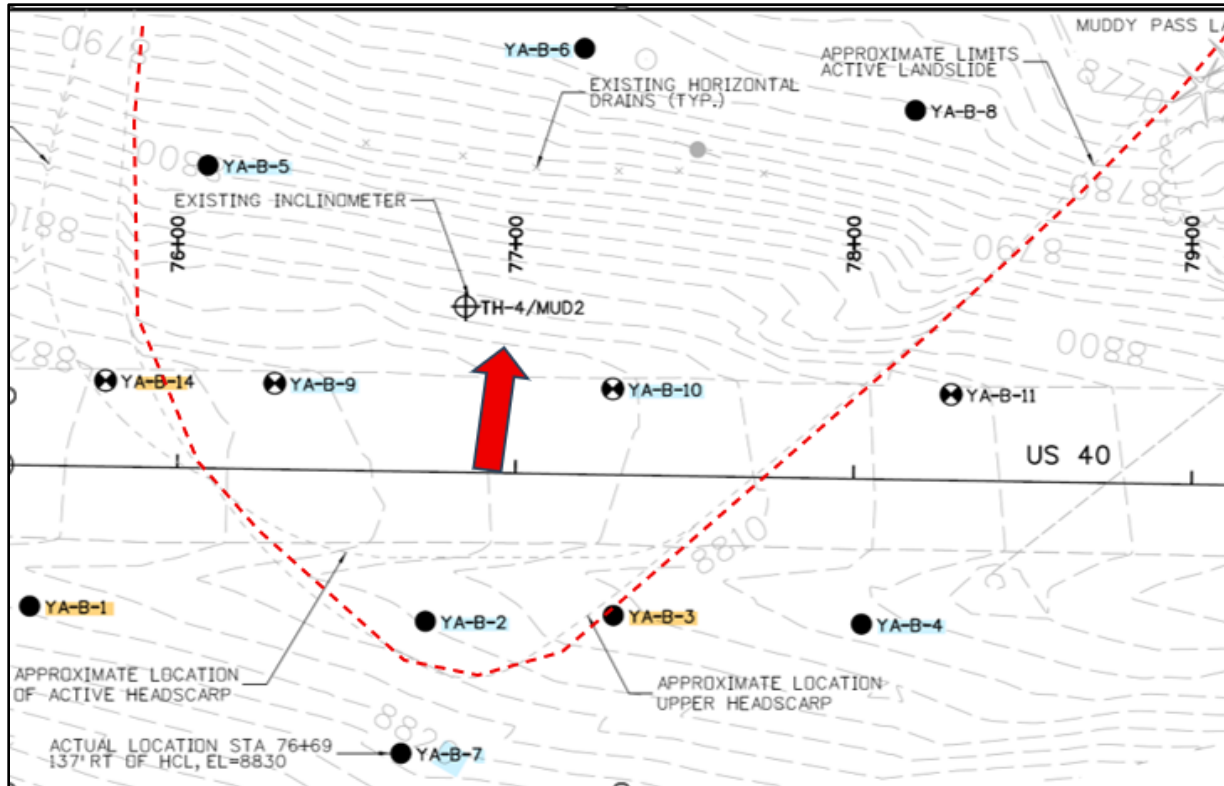


Figure 23. US 40 MP 156.7 Landslide. Blue highlighted borings have natural moisture contents greater than the plastic limit whereas orange borings have natural moisture contents less than the plastic limit. Other borings either have no laboratory data or were very deep below the failure surface. Slide outline is marked in red with red arrow depicting movement direction.

5.2. Summary Of Testing Conducted at the Project Sites

5.3. Background

The project site data was partly from historic records and partly from data obtained from CDOT. The project data generally consisted of a pdf Plan Set that described the overall project with a sheet with geotechnical laboratory testing information and a location. The data was not always complete as some of the projects were more than 20 years ago and global stability modeling was rarely included in the project documents. However, in some cases, the strength parameters were shown and/or the current research author (who had been the designer of record for these projects) pieced together the strength parameters used for mitigation of the project. Stability analysis from the older sites was performed with discontinued global software packages that are no longer available.

5.4. Summary of Laboratory Testing

Appendix A is a summary of the borings for the seventeen (17) sites described in Figures 1 and 2.

Appendix A depicts the location and number of samples that were within the failure zone and within the likely sliding mass. Most of the sites did not have inclinometer data so it was assumed that most slides were failing in the overburden clays above the bedrock and that the failure was within this zone or at the bedrock surface.

5.4.1. Specific Items for Reviewing Appendix A Table

A few items are of importance in reviewing the table. The void ratio e was calculated assuming that the clays were not 100% saturated using the following equation:

$$e = \frac{G_s \cdot \gamma_w}{\gamma_d} - 1$$

- Where G_s is the specific gravity
- γ_w is the unit weight of water
- γ_d is the dry density

The saturation, S_r , of the material was calculated using the following equation:

$$S_r = \frac{w \cdot G_s}{e}$$

- Where w is the natural moisture content
- G_s is the specific gravity of the soil

The Liquidity Index LI (discussed in Chapter 2)

$$LI = \frac{W - PL}{LL - PL}$$

- Where PL is the Plastic Limit
- LL is the liquid limit
- w is the natural moisture content

A new concept designated as B Ratio (Br) was developed by the paper researchers as a ratio between the natural water content and the plastic limit that is which is shown in the following relationship:

$$\text{Br} = \frac{w}{\text{PL}}$$

- Where w is the natural moisture content
- PL is the plastic limit

The purpose of the B Ratio (Br) is to illustrate that the material samples where the natural moisture of the soil sample is greater than the PL in the form of a percentage.

Table 3 depicts the summary of the seventeen sites that are compiled into an average, standard deviation, high and low ranges for the natural moisture, densities, Atterberg testing, saturation, liquidity index, and ratio of moisture to plastic limit from table shown in Appendix A at the end of this report.

Table 3 is the basis to illustrate that the project sites all seem to have properties that are similar and do not deviate greatly when compared to other clay and claystone materials that are not located in Cretaceous materials. For example, other sites across the country that are not associated with Cretaceous materials may have very different natural moisture, liquid limits, etc. that are double the properties shown here.

Table 3. Statistical averages for the sites shown in Appendix A

	Natural Moisture w (%)	Dry Density (pcf)	Fines #200	LL (%)	PL (%)	PI (%)	Sr	LI	B Ratio
Ave	22	102.9	80	43	17	25	92%	0.2	127%
StdD	2	3	13	6	3	5	7%	0.1	17%
High	26.0	107.6	98.5	53.0	23.5	35.0	98%	0.4	164%
Low	18.4	97.5	49.6	34.6	13.6	15.5	78%	0.1	107%

5.5. Unit Weight, Cohesion and Phi Angles Used for the Sites

Using Table 3 as a basis of comparison to demonstrate that Cretaceous period materials are very similar based on moisture and Atterberg testing, Table 4 is a depiction of the seventeen sites that is summarized for the shear strength properties used in the back-analysis relative to the Br ratio. Table 4 illustrates the average standard deviation and high and low values for all the sites. Appendix B at the end of the report provides for a listing of all the sites.

Table 4 is the basis to describe reasonable strength parameters using phi and cohesion in the back analysis for setting up the existing stability model in landslides and embankment failures in Cretaceous clay and claystone materials. It was generally found that when the Br ratio is greater than 100%, those sample materials are within a failing slope and at a higher degree of saturation.

Table 4. Table of slope stability inputs used for all the sites in the project area.

	B Ratio	Unit Weight (pcf)	c' (psf)	Φ' (deg)
Ave	127%	115	206	15
StdD	17%	8	151	4
High	164%	130.0	550.0	21.0
Low	107%	105.0	55.0	7.0

Chapter 6. Global Stability Modeling

This section presents two (2) typical sites that are modeled with slope stability software. As discussed previously, most of the global stability evaluations, especially more than a decade ago, were either not found or analyzed in defunct software that is no longer available. The intent of this section is to present the process as follows for two typical global stability analyses for:

- one stability model with complete geotechnical information.
- one stability model with incomplete geotechnical information.

In our experience, we typically do not have complete geotechnical information, so the reality is the stability analysis falls somewhere in between the two. The following are concepts to consider in the global stability evaluation:

- Existing or Pre Failure Condition: Evaluation of the existing and/or pre-failure condition at the site. This generally may use higher shear strength parameters and/or a less saturated site condition. This is an optional process that may not be available, but it can provide interesting insights into the landslide mechanisms.
- Failed/Existing Condition: Evaluation of the failed condition and/or failing condition is essential. For most the slides at the project sites they are generally re-activated slide surfaces than in many cases have been failing for decades, years, months, or days. This is back analyzed to a FS of 1.0.
- Proposed Mitigation: Evaluation of the proposed mitigation using the conditions and strength parameters of the failed/existing condition for a direct comparison of the benefits of the mitigation and increase on FS relative to a baseline FS of 1.0.

6.1. Description of Items from Field Physical Evidence

For all the landslides described in this research effort there is much data and general information that can be obtained or considered if performing a geotechnical investigation. The following is a brief list of items that may be determined in the field or can be determined with a geotechnical investigation:

- Location of the Head Scarp. This is generally present for almost all landslides as it is very noticeable especially when driving or maintaining roadway corridors.
- Groundwater Elevation. This may be more difficult to determine without a standpipe or piezometer, but in many instances ground water seeps can be found at the base of the slide near the toe to at least locate one point. However, many of the clays and claystones have fractured flow patterns. It may be that ground water is following fractures and cracking in the soils and claystones that will not be captured in a piezometer or standpipe.

- **Direct Shear Data.** Direct shear data can be very problematic as has been discussed. Sampling of the failure surface is generally not possible, in many instances, it is only a few inches in width. Also, direct shear data that is from a location that is not near or within the failure surface may provide misleading information and may not be of use in a stability analysis.
- **Atterberg Testing.** Assuming geotechnical borings can be drilled within the landslide footprint, if the natural moisture is at or greater than the plastic limit (B ratio greater than 100%) it may indicate the sample is near the slide failure surface and can be a useful data point in the modeling.
- **Movement of the Slide at Depth.** Inclinometers or standpipes (for rapid slides) are incredibly useful to determine the depth of the failure surface and magnitude of the slide.
- **Cross Sections and Toe Slope Data.** A detailed cross-sectional survey is a great tool at determining subtle changes in the slope topography which can lead to insights of where the slide toe may be pushed out on the surface.
- **Bedrock Inclination.** For two lane roadway corridors in Colorado at least two (2) borings are needed in cross section to define a bedrock inclination for useful modeling a cross section of the slide.

Table 7 depicts the general concept of the preceding bullet items that are useful for modeling.

Table 7. Items that are useful for global stability modeling.

Designation	Field Physical Evidence	Comments
A	Location of Head scarp	Common Feature
B	Ground Water Elevation	Problematic Depending on Borings/Piezometer Etc.
C	Direct Shear Data	Uncommon to Sample Actual Failure Surface
D	Atterberg Testing (Natural Moisture vs Plastic Limit – B Ratio)	Obtained from Geotechnical Borings
E	Movement of Slide at Depth	Inclinometers, etc.
F	Cross Section Data and Toe Slope	Survey and/or other 3 Dimensional Softwares, etc.
G	Bedrock Inclination	Need at Least Two (2) Borings to Define

6.2. US 36 MP 50.05 Back Calculation Example

The following example is from the US 36 Wall / Embankment / Global Stability failure. The US 36 example is used since this was a unique project in that most of the items in Table 7 were available. This project has also been published in an ASCE publication (Arndt, 2021). Of the items listed on Table 7 it included the following:

- A. The head scarp was identified very early in the project which resulted in closing the roadway corridor.
- B. The groundwater levels were determined during excavation of the failing wall.
- C. There was a fortunate opportunity to sample the correct orientation and actual location of the failure plane that exhibited slickensides. Figures 24 and 25 depict the slickensides for one section of the failure plane surface. Figure 26 depicts the resulting peak and residual direct shear testing from the project.
- D. There were geotechnical borings and samples pre and post failure with Atterberg testing.
- E. CDOT Geotech was able to install a critical boring and standpipe at toe of the failure during the failure process to determine depth of failure surface and soil conditions.
- F. Pre failure and post failure geometry of the landslide and toe slopes was available.
- G. Two borings within the modeling cross section indicated bedrock inclination and position.



Figure 24. US 36 MP 50.05 Location of Slickensides Donating Failure Plane in Excavation.



Figure 25. US 36 MP 50.05 Location of Slickensides denoting Failure Plane in Excavation with California Sampler Hand Driven to collect sample for Direct Shear.

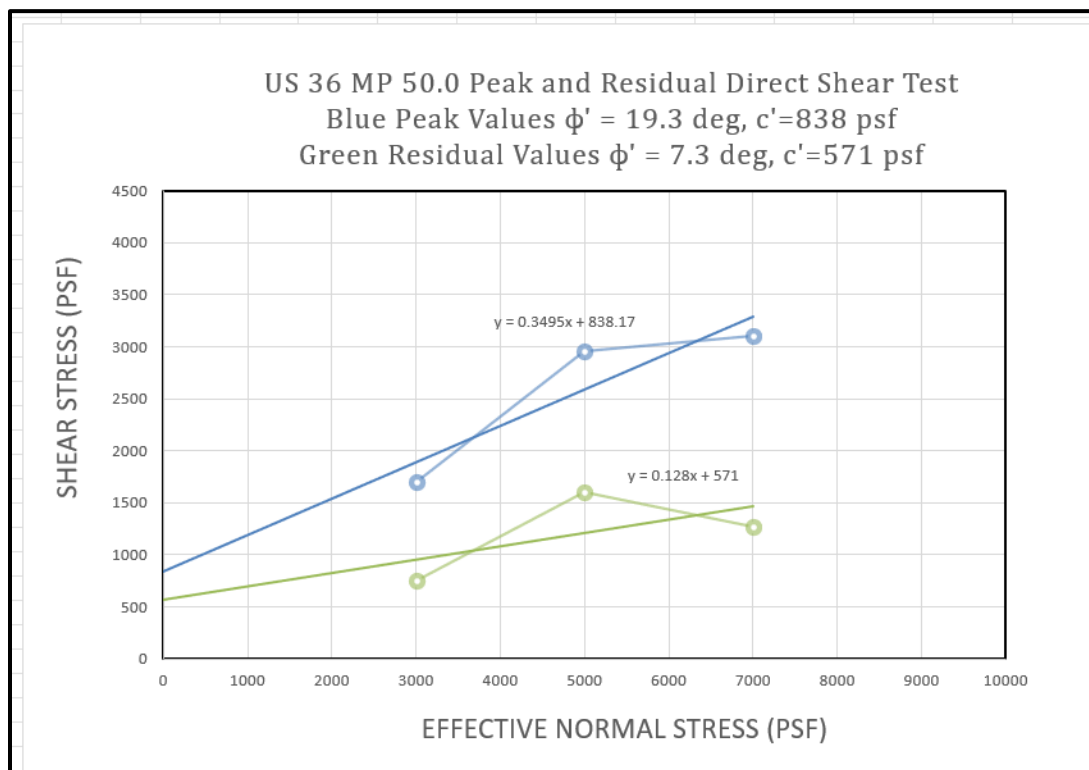


Figure 26. US 36 MP 50.05 Peak and Residual Direct Shear Testing Based on California Sample in Figure 25 for fully saturated conditions. Direct shear testing in which three separate shearing processes occurred at three confinement pressures provide well defined peak and residual values for use in the modeling. The residual cohesion was not forced to zero and indicated a value of 571 psf.

6.2.1. Pre Failure Condition (Peak Strength)

Overall, it is typical to begin with a circular model and for most the stabilities in the report the Bishop Simplified was used with Spencer methods to check. This model is the pre-failure geometry using the peak shear strengths that were obtained from Fig 26. Notice that the letters A through G correspond to Table 7 above for consideration of the field elements. It should be noted that the ϕ angle used from the peak strength was not modified, but the residual cohesion was lowered from 838 psf to 400 psf where the FS was at 1.0 or slightly less than one.

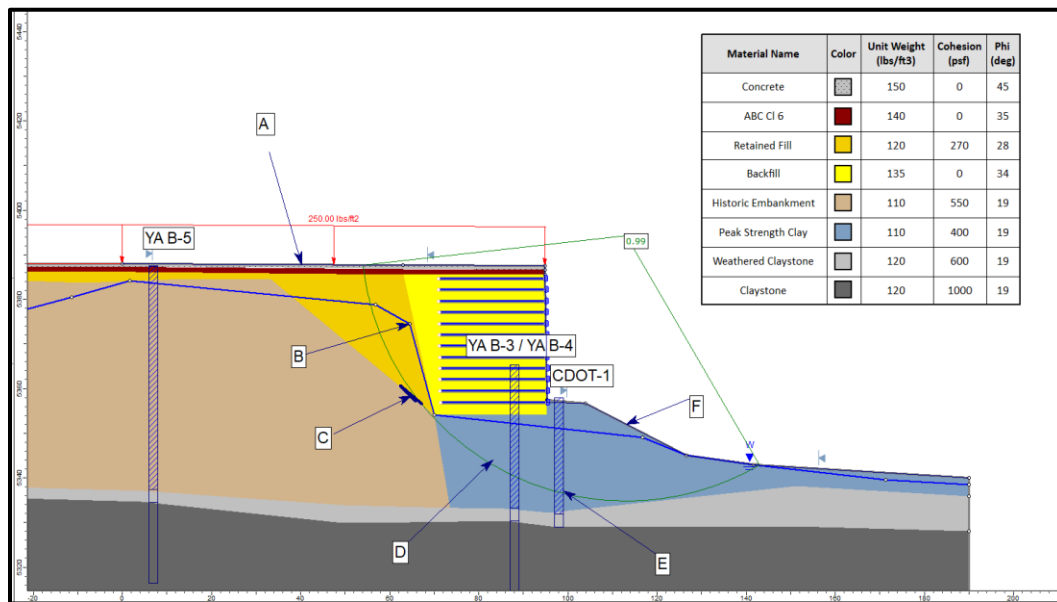


Figure 27. US 36 MP 50.05 Peak Shear Strength Values used from Direct Shear Testing ($\phi' = 19$ deg, cohesion (c) from 838 to 400 psf).

6.2.2. Failed/Existing Condition (Residual Strength)

This model effort was to evaluate the failed/existing condition using the residual strength shear testing values with the vertical drop in the actual wall relative to the previous location. Also notice that letters A through G correspond to Table 7 above for consideration of the field elements in which all the bullet items are addressed. The ϕ angle was used from the residual strength, but the residual cohesion was lowered from 571 psf to 350 psf where the FS was at 1.0 or slightly less than one.

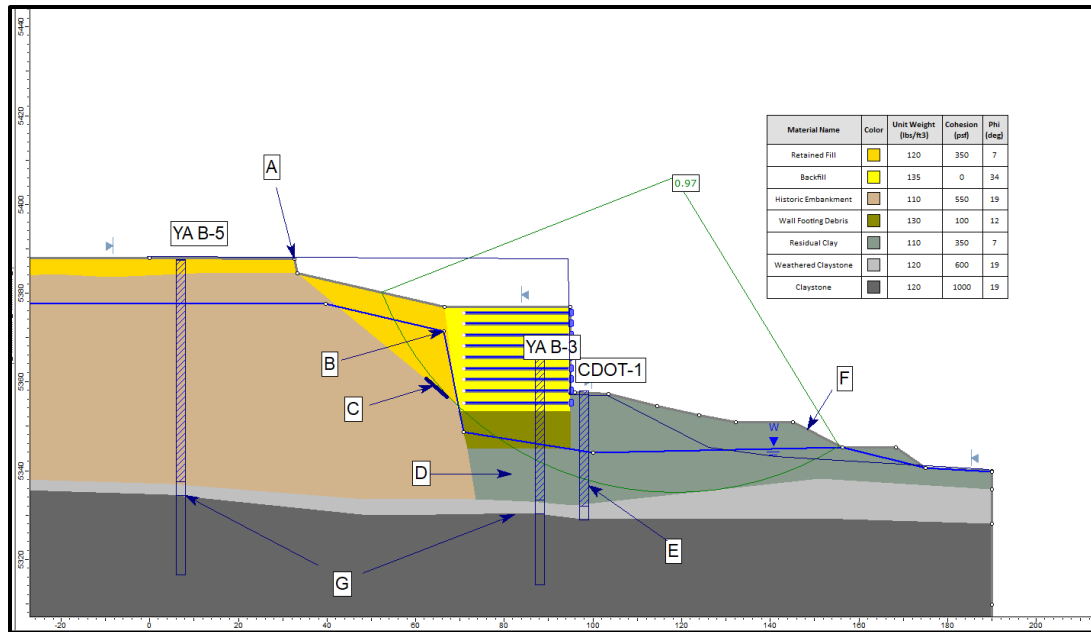


Figure 28. US 36 MP 50.05 Residual Shear Strength Values used from Direct Shear Testing ($\phi' = 7$ deg, cohesion (c) lowered from 571 to 350 psf).

6.2.3. Mitigation Option (Check Mitigation with Residual Strength Condition)

This option takes whatever the proposed mitigation is and compares it directly to the existing failed condition to evaluate the benefit of the mitigation option as compared to a baseline of the failure. For this project a FS of greater than 1.50 was part of the early criteria.

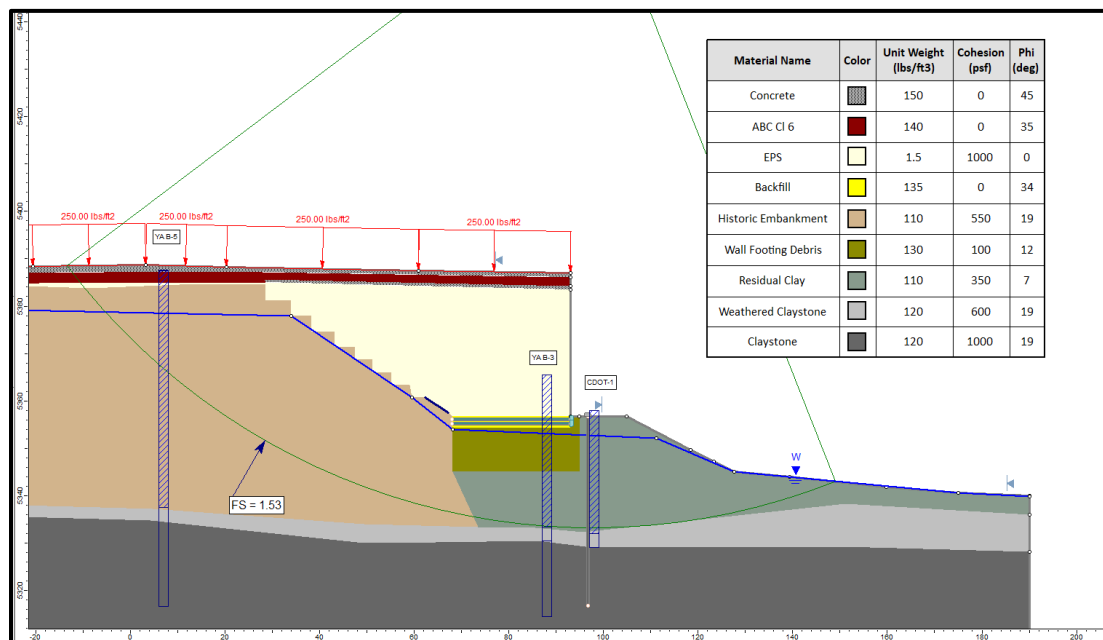


Figure 29. US 36 MP 50.05 Residual Shear Strength Values used from Direct Shear Testing for final build out to achieve a FS > 1.50 ($\phi' = 7$ deg, cohesion (c) lowered from 571 to 350 psf).

6.2.4. Discussion of Using Zero Cohesion Resulting in Nonrepresentative and Misleading Stability Model.

This section provides an example of a non-representative and misleading back calculated stability model that is based on some papers in the literature that describe zero cohesion should be used for the residual strength properties. The following example is intended to show that using a cohesion of zero in a back calculated model is not more conservative and the resulting model is misleading which is not useful for comparing various mitigation options.

The question that arises is what process would occur to use zero cohesion to the above evaluation/analysis and how does that relate to using cohesion in a slope mitigation evaluation? If we take the residual shear testing and force the regression/trendline through zero it results in the ϕ angle would increase from 7 degrees to 13 degrees. Figure 30 below depicts forcing a residual regression failure through zero. Figure 31 below depicts what the stability would look like with just using a $\phi = 13$ degrees with a cohesion of zero. Note that the residual phi of 13 degrees is nearly double the original phi value of 7 degrees which is non-representative and results from eliminating cohesion from the model.

Figures 30 and 31 depicts forcing a stability model using an increased residual phi angle that results from forcing the cohesion to zero. The result is a stability model where the resulting FS is 0.65 and is non-meaningful and misleading since no cohesion was used to maintain equilibrium. There is no reasonable way to provide for a back calculated FS of 1.0 without extraordinary manipulation of the failure/existing model. We believe eliminating cohesion does not provide for any meaningful analysis. What can occur is as follows:

- As provided by the publication on cohesion (Day, 1995) we believe that all models require some magnitude of c' to satisfy equilibrium. This is necessary to achieve a back calculated FS of 1.0.
- Using zero cohesion the only way to increase the model to a FS = 1.0 is to increase the ϕ angle up artificially, which in this case, would nearly double the actual residual ϕ phi angle.
- Using an inflated ϕ angle, depending on the slope geometry and conditions, may result in an over excessively conservative or under conservative model.

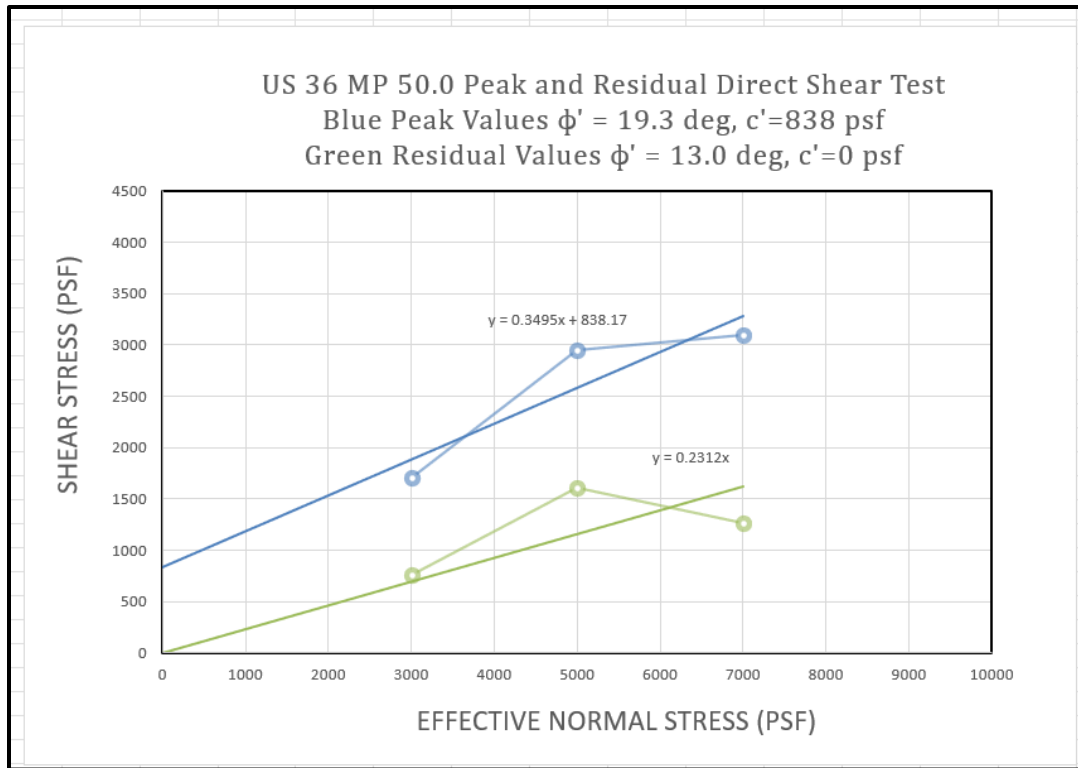


Figure 30. Example of Misleading Model for US 36 MP 50.05 Peak and Residual Direct Shear Testing Based on California Sample in Figure 26 by artificially forcing the cohesion (c) to zero which increases the ϕ from 7 to 13.0 deg. This is a comparison with the actual values determined from direct shear testing in Figure 26.

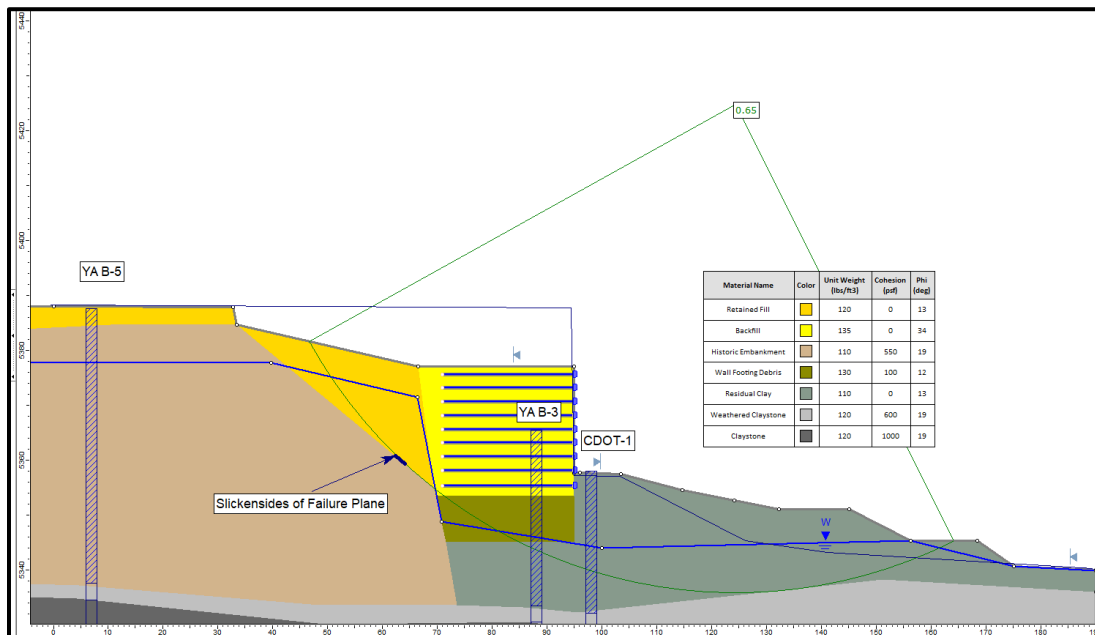


Figure 31. Example of Misleading Model for US 36 MP 50.05 Residual Shear Strength Values used from Direct Shear Testing ($\phi' = 13$ deg but forcing cohesion (c) to zero psf). FS is 0.65 and is a forced non-meaningful and misleading back calculated FS.

6.3. US 50 MP 104.3 Back Calculation Example

US 50 MP 104.3 is provided as an alternative site that does not have all the information from Table 7.

For this site the following were available A, D, and F from Table 7:

- The location of the head scarp.
- Atterberg testing from one (1) boring.
- A detailed slope topography of the cross section using web-based software.

Figure 32 below depicts the typical section of the roadway failure. To evaluate the site the following was evaluated:

- A. Location of Failure Head Scarp. Very evident.
- B. Groundwater Elevation. Estimated.
- C. Direct Shear Data. None.
- D. Atterberg Testing. One sample was obtained; the natural moisture was at or greater than the Plastic Limit (B ratio greater than 100%) so is likely near the slide failure surface.
- E. Movement of the Slide at Depth Data. None.
- F. Cross Sections and Toe Slope Data. A detailed survey was conducted.
- G. Bedrock Inclination. Estimated.

Overall, with the assumptions made the only reasonable method is to perform a parametric analysis which essentially is varying the unknown parameters relative to the known parameters to a FS of 1.0. Using a parametric process varying one parameter at a time, it becomes evident what factors are controlling the slide stability and then the model can focus on changing which parameters seem to have the most effect. Also, since the overall slope surface angle is 27 degrees, a higher ϕ range was used. Conversely, if the slope surface is flatter, such as 10 to 15 degrees, then a lower ϕ may be warranted.

Figure 32 depicts the results of a parametric evaluation to obtain what would seem to be a reasonable model. Figure 33 is then an evaluation of the mitigation option to see what the benefit of the mitigation is and if it is over the established final FS.

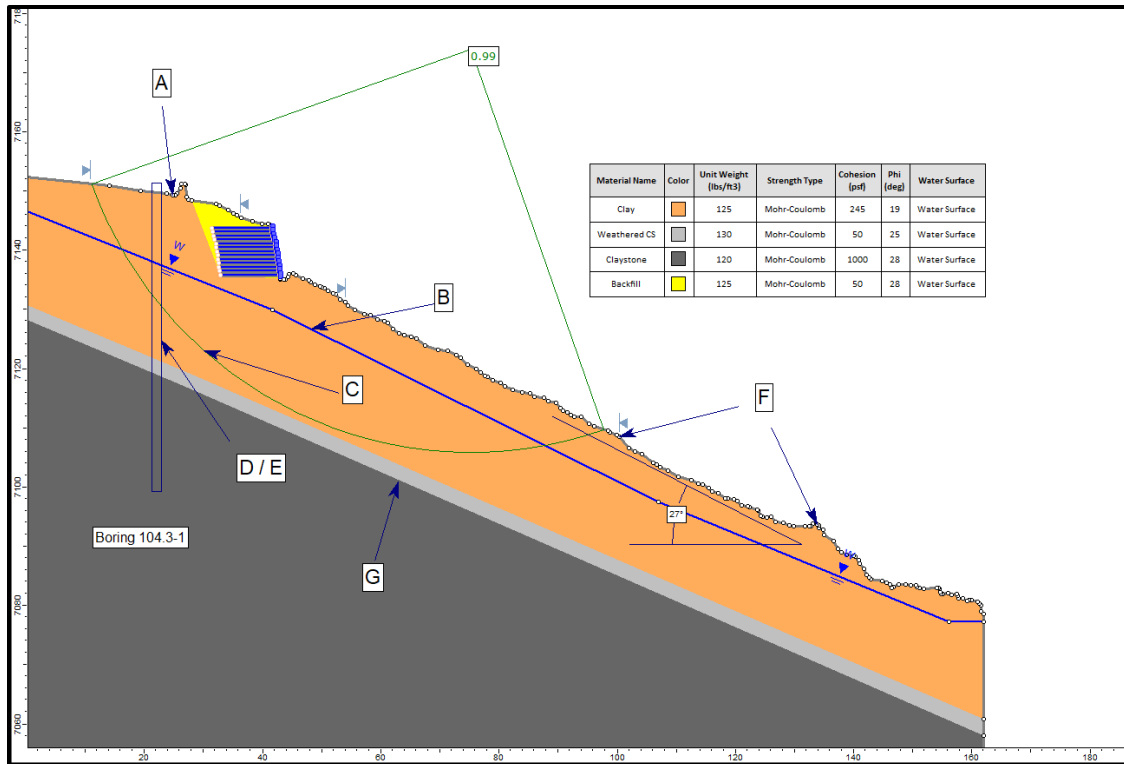


Figure 32. US 50 MP143.5 Residual Shear Strength Values estimated based on one (1) boring. FS is at 0.99 or just at failure for a back-calculated condition.

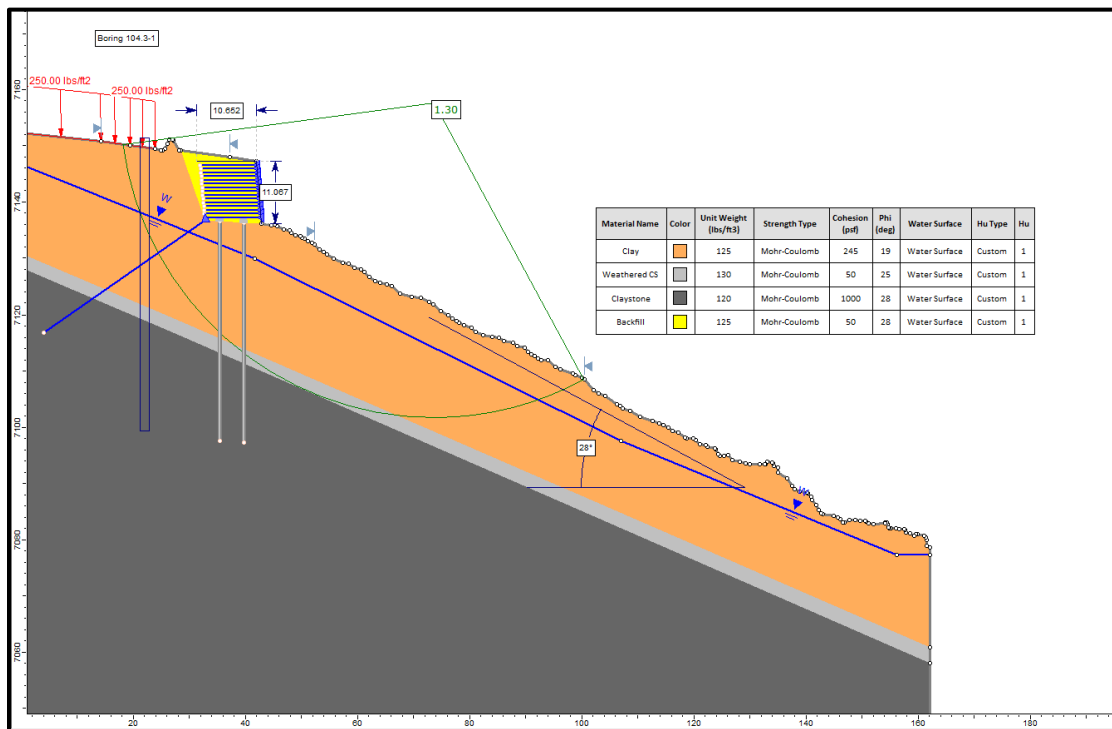


Figure 33. US 50 MP143.5 Residual Shear Strength Values estimated based on one (1) boring. FS is at 1.3 with a micropile and ground anchor tieback system that is compared to the existing condition Fig 32.

Chapter 7. LRFD Methods and Global Stability

Landslide and/or embankment stabilization systems in this study consisted of the following three main categories of:

- Lightweight fill replacement with geofoam or foamed glass for embankment stabilization.
- Ground anchor panels with tiebacks.
- Foundation support elements of either drilled shafts or micropiles.

To correspond to AASHTO, it is generally accepted to refer to Section 11.6.2.3 – Overall Stability. In this section the manual suggests that the evaluation of overall stability of earth slopes. Essentially the section states “In lieu of better information, the resistance factor ϕ (not to be confused with ϕ), may be taken as:

- Where the geotechnical parameters are well defined, and the slope does not support or contain a structural element.....0.75.
- Where the geotechnical parameters are based on limited geotechnical information, or the slope contains or supports a structure element.....0.65.

Generally, what is interpreted for FS is to use the reciprocal resistance values to represent a global FS:

- Case 1. $FS = 1/0.75 = 1.33$.
- Case 2. $FS = 1/0.65 = 1.54$.

Furthermore, it should be noted that with respect to AASHTO, one convenient method to evaluate the function of a structure is to look at a ratio of the total factored resistance to the total factored loading. This ratio is also referred to the Capacity/Demand Ratio (CDR).

- $CDR = \text{Total Factored Resistance} / \text{Total Factored Load} \geq 1.0$

What generally occurs is that either an FS of 1.33 or 1.54 is selected for an embankment or landslide stabilization system. If the system is a lightweight fill and/or ground anchor, typically the system is designed for the ground anchors, panel strength, and anchor bond pullout, but no other loading factors are applied.

However, when a drilled shaft system is designed a FS is selected, the drilled shaft or micropile design engineer may refer to AASHTO Table 3.4.1-2 – Load Factors for Permanent Loads γ_p . This table provides, for example, various Load Factors for Horizontal Earth Pressure which for the Active Case is 1.50 (Maximum). From a CDR perspective when evaluating mitigation options the following can occur.

- Scenario A. Lightweight fill and/or Ground Anchor Tieback Systems. For a lightweight fill replacement and/or ground anchors with tieback panels the following occurs for a Global FS of 1.33:
 - Resistance Factor AASHTO Section 11.6.2.3 = (0.75)
 - Load Factor per AASHTO Table 3.4.1-2 = (1.0) (No defined horizontal earth pressure)
 - $CDR = Resistance / Load = 0.75/1.0 = 0.75$ (which the reciprocal for Global FS is 1.33)
- Scenario B. Drilled structural shaft, the following may occur for a Global FS of 1.33:
 - Resistance Factor per AASHTO Section 11.6.2.3 (0.75).
 - Load Factor per AASHTO Table 3.4.1-2 = (1.5 per table for structural element horizontal earth pressure).
 - $CDR = Resistance / Load = 0.5$ (which the reciprocal for Global FS is 2.00).

From this example, the drilled shaft mitigation option has a higher CDR since both the global stability resistance factor and the earth pressure load factor are both used for structural drilled shaft systems. This results in significantly higher costs that may make a certain mitigation option cost prohibitive relative to others.

The intent of this research paper is to highlight that using a global stability resistance factor and an earth pressure load factor results in a very conservative overall design that affects the mitigation options as costs for drilled shaft systems become prohibitive compared to simple systems that do not have additional earth pressure loading factors.

Chapter 8. Conclusions

8.1. Study Results

The study reviewed seventeen (17) previously constructed CDOT and WYDOT landslides which included various mitigation measures. The research also evaluated and compared the soil strength parameters by geologic type with the unit weight, phi, and cohesion to provide appropriate design values for a basis for landslide and embankment mitigation. The study then reviewed how the models can be used and provided two (2) examples of back-calculation stability methods as part of a landslide mitigation process.

We believe the most significant items that were generated from this report are:

- The landslides and embankment failures were all in the Cretaceous geologic unit.
- The Atterberg index tests are all similar with relatively small variations in values.
- The B ratio as described indicates when the natural moisture is greater than the plastic limit the samples taken are generally within the landslide mass indicating the material is saturated and weaker which forms an unstable slope.

The strength parameters were also notable and provide a range for back-calculation for landslide evaluation and analysis. We also reviewed the literature and believe the following is significant:

- Publications that do not allow for cohesion in the stability analysis should be reviewed with caution as the stability analysis will not be representative of the actual field conditions and may be misleading and unconservative.
- We attempted to demonstrate in this research publication that some degree of cohesion is a necessary component for the back calculation modeling and analysis that is used to compare the existing failing condition with the proposed design mitigation.

Based on the study we believe it is acceptable to preliminarily use the strength parameters described in Table 4 as a range of strength parameters that can be used with a back-calculation stability method when site specific data cannot be obtained for direct site analysis for landslides and embankment failures in Cretaceous materials.

Also, considerations were provided when designing structural drilled shaft systems using LRFD methods and that resistance and load factors should be reviewed by all the design parties to provide for a reasonable level of mitigation by understanding and using the load and resistance factors so that the mitigation design is not overly conservative.

Chapter 9. Further Study

Further study and research could be conducted for several topics generated in this research which include:

- The effects of wetting and drying (desiccation) on Cretaceous and other clay materials. Based on the literature review, cracking voids from desiccation remain open even when dry, which results in even deeper wetting when saturation occurs. This tends to result in a shear strength reduction in the embankment materials.
- The effects of saturation on strength parameters of clays. One article that is referenced in this report (Gong 2022) reflected the effects, but it seems more research is warranted to establish lower bounds of cohesion for fully saturated clays which suggest that cohesion is greater than zero.
- Further review of the actual reasoning and justification behind why residual values are forced to zero in some notable publications. Further research could acquire the actual data numbers that were used in generating the peak and residual shear strength graphs to calculate the actual cohesion value if the graphs were not forced to zero.
- Further research into other landslides and Atterberg index properties to compare and contrast with other geologic units and settings.
- It was suggested by a reviewer to consider soil activity as a basis for comparison of the samples, however most general grain size analysis testing does not define the specific clay size materials without hydrometer testing which was not available from the test data presented in this study but could be interesting to review for future study.

References

- Arndt, Ben, 2016, "*Mitigation of Landslides in Colorado and Wyoming Using Geofoam Fill*", Rocky Mountain Geo-Conference 2016 GPP 10, ASCE.
- Arndt, Ben, Johnson, Richard, Kionka, Kurt, 2021, "*U.S. 36 Emergency Project—Geofoam Fill to Support Urban Roadway Traffic*", Rocky Mountain Geo-Conference 2021 GPP 13, ASCE.
- Arndt, Ben, Johnson, Richard, Dahill, James, Booher, Jeff, Wittke, Seth, 2026, "*The Big Fill – Teton Pass Embankment Failure and Mitigation*", Rocky Mountain Geo-Conference 2026, ASCE.
- Cretaceous Atlas of Ancient Life – Western Interior Seaway, 2023, National Science Foundation, Accessed June 1, 2026, <https://www.cretaceousatlas.org/geology/>
- Day, Robert, 1995, "*Cohesion Intercept in Effective Stress-Stability Analysis*", Journal of Geotechnical Engineering, ASCE.
- Duncan, J., Stark, T, 1992, "*Soil Strengths from Back Analysis of Slope Failures*", Stability and Performance of Slopes and Embankments-II Proceedings – Berkeley, CA June 1992.
- Early, K.R., Skempton, A.W., 1972, "Investigations of the landslide at Walton's Wood, Staffordshire", Q. Jl Engng Geol. Vol. 5, 1972, pp. 19-41. Printed in Great Britain.
- Fagel, Nathalie, December 2007, Developments in Marine Geology, Part 1 Deep Sea Sediments, Chapter 4 – Clay Minerals, Deep Circulation and Climate.
- Gong, W., et al., July 2022, "*Statistical analysis on the relationship between shear strength and water saturation of cohesive soils*", Bulletin of Engineering Geology and the Environment (2022) 81: 337.
- Loehr, Eric, et. al., November 2016. FHWA-NHI-16-072 *GEOTECHNICAL SITE CHARACTERIZATION GEOTECHNICAL ENGINEERING CIRCULAR NO.5*. Federal Highway Administration.
- Mersi, G, et. al., 1993. J.Geotech. Engrg, 1229-1249, *Cohesion Intercept in Effective Stress-Stability Analysis*.
- Skempton, A.W., 1964, "*Long Term Stability of Clay Slopes*". Geotechnique, 14, 77-107. <http://dx.doi.org/10.1680/geot.1964>.
- Skempton, A.W., 1972, "*First-Time Slides In Over-Consolidated Clays*". Fourth Rankine Lecture – Technical Notes.
- Skempton, A.W., 1985. (198.5), "*Residual strength of clays in landslides, folded strata and the laboratory**". Geotechnique 35, No. 1. 3-18
- Stark, Timothy, Choi, Hangseok, McCone, Sean, May 2005, "*Drained Shear Strength Parameters for Analysis of Landslides*", Journal of Geotechnical and Geoenvironmental Engineering, ASCE.

Western Interior Seaway – Deep Time Maps, 2023, Accessed June 1, 2026,
<https://deeptimemaps.com/map-lists-thumbnails/western-interior-seaway/>

Appendix A. Summary of Laboratory Testing

Fig 1 and 2 Site Areas	Specific Site	Highway	MP	Geology	DOT Project Number	Year	Mitigation Type	No. Samples	Natural Moisture w (%)	Dry Density (pcf)	Fines #200	LL (%)	PL (%)	PI (%)	Sr (%)	LI	B Ratio
A	1	US 40	156.7	Colorado Group	19248	2013	Soldier Pile/Tieback	8	23.3	102.5	88	46	21	26	97%	0.08	109%
B	2	CO 13	15.9	Mesa Verde / Williams	15918	2006	Geofoam	4	21.1	97.5	55	35	19	16	78%	0.12	110%
C	3	US 36	50.05	Arapahoe	23254	2019	Geofoam	5	20.6	106.3	76	44	17	27	98%	0.12	119%
D	4	US 50	103.7	Mancos	18145	2010	Geofoam	7	20.6	102.3	74	44	17	27	86%	0.13	121%
	5	US 50	104.3	Mancos	17738	2012	Micropile Support	1	22.6	100.5	96	53	18	35	90%	0.13	126%
	6	US 50	104.8	Mancos	17738	2012	Geofoam	1	22.9	100.5	91	38	14	24	91%	0.37	164%
	7	US 50	108	Mancos	17738	2012	Geofoam	1	20.5	107.6	99	43	14	29	98%	0.22	146%
	8	US 50	121.4	Mancos	17735	2012	Geofoam	5	22.7	-	87	49	17	32	-	0.18	134%
	9	US 50	121.8	Mancos	19495	2016	Coupled Shear Shafts	3	23.2	101.3	80	47	18	29	95%	0.18	129%
E	10	US 160	69.8	Mesa Verde/Lewis	15068	2005	Geofoam	5	22.1	-	87	42	19	16	-	0.12	114%
	11	US 160	74.1	Mesa Verde/Lewis	14978	2004	Geofoam	1	22.8	-	80	41	16	25	-	0.27	143%
	12	US 160	75.1	Mesa Verde/Lewis	14978	2004	Geofoam	3	19.8	-	50	35	16	19	-	0.18	121%
	13	US 160	75.3	Mesa Verde/Lewis	14978	2004	Ground Anchor Tieback	1	24.5	-	84	46	17	29	-	0.26	144%
F	14	US 84	7.6	Mesa Verde/Lewis	23758	2024	Coupled Shear Shafts	5	18.4	-	88	42	17	22	-	0.05	107%
G	15	WY 22	12.8	Upper Cretaceous	ND32401	2025	Lightweight Foamed Glass	6	19.8	-	70	35	14	21	-	0.30	146%
H	16	WY 191	141.6	Aspen/Bear River / Colorado Group	N104087	2014	Coupled Shear Shafts	1	26.0	-	72	52	24	28	-	0.09	111%
I	17	WY 220	101.6	Cloverly/Mowry	N212107	2012	Coupled Shear Shafts	9	19.3	107.2	92	36	17	20	91%	0.14	117%

Appendix B. Summary of Shear Strength Properties Used at Project Sites.

Fig 1 and 2 Site Areas	Specific Site	Highway	MP	DOT Project Number	Year	Mitigation Type	No. Samples	B Ratio	Unit Weight (pcf)	c' (psf)	Φ' (deg)
A	1	US 40	156.7	19248	2013	Soldier Pile/Tieback	8	109%	130	200	10
B	2	CO 13	15.9	15918	2006	Geofoam	4	110%	120	500	12
C	3	US 36	50.05	23254	2019	Geofoam	5	119%	110	345	7
D	4	US 50	103.7	18145	2010	Geofoam	7	121%	105	275	20
	5	US 50	104.3	17738	2012	Micropile Support	1	126%	125	160	16
	6	US 50	104.8	17738	2012	Geofoam	1	164%	105	150	14
	7	US 50	108	17738	2012	Geofoam	1	146%	105	150	14
	8	US 50	121.4	17735	2012	Geofoam	5	134%	110	80	16
	9	US 50	121.8	19495	2016	Coupled Shear Shafts	3	129%	120	180	12
E	10	US 160	69.8	15068	2005	Geofoam	5	114%	100	100	18
	11	US 160	74.1	14978	2004	Geofoam	1	143%	110	100	18
	12	US 160	75.1	14978	2004	Geofoam	3	121%	110	75	19
	13	US 160	75.3	14978	2004	Ground Anchor Tieback	1	144%	110	100	18
F	14	US 84	7.6	23758	2024	Coupled Shear Shafts	5	107%	120	80	10
G	15	WY 22	12.8	ND32401	2025	Lightweight Foamed Glass	6	146%	135	100	18
H	16	WY 191	141.6	N104087	2014	Coupled Shear Shafts	1	111%	120	150	11
I	17	WY 220	101.6	N212107	2012	Coupled Shear Shafts	9	117%	120	380	12