CHARACTERIZATION, ANALYSIS, AND REMEDIATION OF THE CEDAR PASS LANDSLIDE COMPLEX, BADLANDS NATIONAL PARK, SOUTH DAKOTA

by

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ABSTRACT

The Cedar Pass Landslide Complex is located in the North Unit of Badlands National Park, South Dakota. The National Park Service has had to regularly maintain the approximately 1.25 km section of Badlands Loop Road (South Dakota State Hwy-240) that travels through the landslide complex. Road surface distress caused by slope movement and other natural processes in the Cedar Pass area have created a financial burden for the park, as the Park Service is responsible for maintenance of the highway. While there has been successful mitigation work to stabilize portions of the road, stability and erosion problems have persisted. Maintenance and mitigation work completed since the 1990s include the construction of two large earth buttresses, roadway resurfacing, regular crack sealing and asphalt patching, grinding to smooth surface offsets, and the installation of a new stormwater collection and conveyance system.

This study used a combination of field reconnaissance, sample collection, laboratory testing, and slope stability modeling to estimate and delineate boundaries of several landslides in the Cedar Pass Landslide Complex, assess the current stability, and investigate the sensitivity of these landslides to factors that may increase or decrease stability. These factors include fluctuations in groundwater, reduction in shear strength of landslide materials, and erosion within the landslide mass. Additionally, the effectiveness of mitigation measures was investigated using a probabilistic analysis to identify those methods that result in the greatest increase in stability. Direct shear testing was carried out to measure the drained residual shear strength of soils in the complex, and Atterberg limits and grain-size distributions were measured to characterize soils and to estimate residual strength using an empirical correlation developed by Stark and Eid (1994). Slope stability modeling was conducted using two-dimensional limit equilibrium methods.

Results show that highway surface damage in the complex is related to a combination of both movement in smaller, unique areas and movement of much larger landslides. For instance, damage to the Cliff Shelf parking lot is related to destabilized areas above the head scarp of the Prairie Island Landslide located to the southeast. This is compared to highway distress in the Upper and Lower Wedge areas that may relate to settlement and erosion of an embankment fill and continued deformation of the Cliff Shelf Landslide which was thought to be dormant until the late 1990s. The overall slow movement of the landslides observed over the past 30 years may be attributed to dilatant strengthening, which suggests that the landslides present a risk to park
infrastructure (mainly the highway), but pose less of a danger due to sudden movement. However, it is prudent to assume that more rapid failure may be possible if climatic and geologic conditions change, specifically, if average groundwater levels across the complex increase or if soils along the landslide slip surfaces reach the critical state density.

Mitigation of these landslides may be possible on a localized scale with the construction of earthen buttresses, gravity retaining walls, tieback walls, and/or improved slope drainage. Mitigation of the larger landslides such as the Cliff Shelf Landslide is less feasible due to their size. Therefore, highway distress and deformation may continue and it is recommended that highway improvements include the addition of a flexible pavement or road base that can help distribute deformation and may decrease the frequency of required maintenance in certain areas.
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CHAPTER 1
INTRODUCTION

Landslides are some of the most common geologic hazards around the world, as they can occur in a wide variety of settings and circumstances. The economic losses due to landslides can be large with millions of dollars of damage caused to property and infrastructure. Thus, proper investigations to fully characterize a landslide and the factors contributing to the instability, including aspects of the geology, climate, hydrogeology, land use, and soil mechanical properties, can help make informed decisions that lessen or possibly eliminate the impacts of this hazard.

The Cedar Pass Landslide Complex located in Badlands National Park, South Dakota, has been the focus of many geotechnical investigations over the previous three decades due to the persistent infrastructure damage caused by slope instability in this region. A majority of the damage involves South Dakota State Route 240 (SR-240). SR-240, also known as Badlands Loop Road, is the main artery for local, commercial and tourist traffic through the northern unit of the park. Slope movements in multiple locations along the highway within the complex have created a financial burden for the National Park Service (NPS), as the Park Service is responsible for maintaining this portion of the highway through the park.

This thesis will address the following tasks in order to improve and expand upon the knowledge about the Cedar Pass Landslide Complex:

1. Identify landslide boundaries with an emphasis on those areas where suspected landslide movement is negatively impacting the highway or other park structures. This will be completed by interpreting the orientation and location of landslide-induced geomorphic features, and by confirming or revising the boundaries mapped by the National Park Service.
2. Propose an interpretation of the interaction between different blocks within certain landslides and their direction of movement. If landslide movement is not thought to be responsible for the observed damage, propose the cause of damage to park infrastructure.
3. Provide the Park Service with an analysis of the current stability of the most important landslides identified within the complex, as well as an idea of what factors
are playing a primary role in causing instability and driving movement. Important landslides are those that are actively causing damage to park infrastructure and have not been stabilized.

4. Describe the likely effectiveness of a variety of remedial measures the Park Service may be able to implement to improve stability along the highway and reduce the amount of required maintenance by evaluating changes in stability of the landslides as a result of remediation measures.

5. Synthesize all of the information above to develop a clearer picture of the characteristics and failure mode within the complex, as well as discuss the hazard implications for this landslide complex and the park in general.

These tasks were completed using detailed field mapping across the landslide complex, soil sample collection and material properties testing, and slope stability modeling including landslide sensitivity to various geologic and environmental factors and mitigation options. It is the author’s hope that the information contained within this thesis can help Park Service staff make informed decisions regarding the management of the highway through the Cedar Pass Landslide Complex, and throughout other areas within the park affected by landslides.
CHAPTER 2
OBJECTIVE AND SCOPE

In the past 30 years, there have been several geotechnical studies conducted in the Cedar Pass Landslide Complex in order to address slope stability issues that have continually caused damage to the highway. Past studies have provided general comments about failure mechanisms, landslide sensitivity and approximate landslide boundaries, but have not provided a thorough analysis of any of these issues. The main objective of this study is to provide a comprehensive assessment and characterization of several landslides in the complex in order to better understand the type(s) of landslide present and how these landslides behave. Additionally, this study aims to address how these landslides may be mitigated by analyzing the effectiveness of different remediation techniques. In order to complete these objectives, this work focuses on answering the following questions:

1. What are the boundaries of identified landslides, located within the Cedar Pass Landslide Complex? Is slope movement responsible for damage to Badlands Loop Road and other park infrastructure?
2. What is an effective methodology for locating the toe of a landslide that is characterized by more defined features at the head but with an ambiguous lower boundary?
3. What geologic or climatic factors control instability in this area? It is hypothesized that landslide movement is predominantly driven by the low strength of landslide materials when exposed to water, by periodic fluctuations of groundwater, and by topographic changes in the landslide mass including the erosion of material at/near the toe.
4. What is the effectiveness of different mitigation techniques such as improved drainage or the construction of earthen berms or retaining walls in stabilizing untreated landslides within the complex?

The first and fourth questions are specific to Badlands National Park and are meant to provide a clearer picture of where instability exists in the landslide complex, whether or not areas of instability are related to each other, and how well various mitigation techniques work at improving stability. The National Park Service has attempted to identify individual landslides in the complex with a relatively limited amount of fieldwork (Figure 2-1). A more robust mapping campaign is intended to refine mapped landslide boundaries, as well as provide a baseline from
which the Park Service can track changes in the location and distribution of landslide-induced geomorphic features and highway damage. The different mitigation techniques analyzed are those that are typically used to improve stability and include some methods already employed in Badlands National Park. It should be noted that this study does not attempt to explain highway damage throughout the entire complex but addresses damage in areas that have not been remediated and require the most consistent maintenance. These areas are considered a higher priority.

Figure 2-1. The boundary of the Cedar Pass Landslide Complex and the extent of other identified landslides within the complex as well as estimated directions of movement produced by the National Park Service (2016) based on observations made in the field.

The second question is aimed at developing a simple methodology that may be utilized by other researchers to help constrain the lower boundary or toe of a landslide while building a slope model when the location could not be identified confidently in the field. This is an issue in
Badlands National Park, and likely in other locations around the world with high erosion rates, where toe features may be destroyed or modified to a point where they cannot be accurately mapped. Poorly constrained boundaries, especially in two-dimensional (2D) slope models, and in instances where sufficient geotechnical information is not available, leads to a high variability in slope stability analyses.

The third question is specific to the landslides in Badlands National Park but may be applicable to other areas around the world with similar geology and climate. The hypothesis is investigated through sensitivity analysis of the slope stability models.
3.1 Badlands National Park and Badlands Loop Road (SR-240)

Badlands National Park, also known as the Big Badlands or White River Badlands, is renowned for its colorful cliffs of horizontal rock strata, dramatic spires and rugged topography, and the largest assemblage of known late Eocene and Oligocene mammal fossils (NPS 2020). This National Park, located in southwestern South Dakota on the Great Plains, is the largest area of badlands topography in the world covering approximately 10,400 square kilometers (4000 sq. mi.) (Darton 1921; Smith 1958). In 1939, the area was officially designated as a National Monument by President Franklin D. Roosevelt. The monument was enlarged and given National Park status in 1978. The park is divided in three units including the North, South, and Palmer Units. The North Unit is the largest of the three and is the location of Park Headquarters and the Ben Reifel Visitor Center (Figure 3-1).

Figure 3-1. Badlands National Park with an inset of the Cedar Pass area (adapted from National Park Service 2019a; National Park Service 2019b). The blue circle in the inset is the approximate location of the Cedar Pass Landslide Complex.
South Dakota State Highway 240, also known as Badlands Loop Road, is the main road through the North Unit and traverses approximately 44 kilometers between the Northeast Entrance Station which is 6 kilometers south of Exit 131 on Interstate 90 at Cactus Flat, and the Pinnacles Entrance Station, 11 kilometers south of Wall, SD at Exit 110 on Interstate 90. From the Northeast Entrance, Badlands Loop Road travels along both the upper prairie and lower prairie making the transition between the two by way of four named passes that intersect the most dominant geomorphic feature in the North Unit known as the Badlands Wall or simply “The Wall”. This east-west trending escarpment extends for over 100 kilometers and is the divide between the lower prairie along the White River to the south and undissected upland (upper prairie) to the north. The first pass encountered when driving west along Badlands Loop Road from the Northeast Entrance is Cedar Pass. The road utilizes the gentler angled slopes created by two large paleolandslides that define the boundaries of the Cedar Pass Landslide Complex (Figure 3-2) as the highway descends from the pass down to the Ben Reifel Visitor Center located approximately 2 kilometers west along the road from the pass. The section of highway that travels across the landslide complex is approximately 1.25 kilometers long. This section of road is vital to the park as it is the most heavily used segment with an estimated 75 percent of all visitors traveling through the park entering through the Northeast Entrance (Anderson et al. 2004; FHWA 2013). In addition, this portion of highway also provides a crucial route for local and commercial traffic traveling from the north side of the park and Interstate 90 to the town of Interior and the Pine Ridge Reservation located to the south.

3.1.1 Physiography and climate

Badlands National Park sits within the geographic province known as the Great Plains. The province is dominated by low-relief topography and extends from the eastern slope of the Rocky Mountains out to the eastern sides of North Dakota on down into northwest Texas (Kiver and Harris 1999). Located in southwest South Dakota, Badlands National Park is completely surrounded by mixed grass prairies and is approximately 120 km east of Rapid City and the Black Hills.

Temperatures in Badlands National Park can range from 40 degrees Celsius in the summer to -40 degrees Celsius in the winter (NPS 2020). Data assembled from the National Oceanic and Atmospheric Administration, National Centers for Environmental Information show
that the park receives an average of 450 mm (18 in) of rain based on data from the last 50 years, recorded at the weather station located at the Ben Reifel Visitor Center. The wettest time of year is typically late spring through the summer months. On average, around 70% of the annual precipitation can fall between April and August. Rainfall events can be long or short duration, and convective type events are common throughout the late spring and summer months. These storms are capable of producing tens of millimeters of rain in the matter of a few hours. See Appendix C for tabulated rainfall data at the visitor center for 1970-2019.

Figure 3-2. The approximate boundary of the Cedar Pass Landslide Complex, the Cedar Pass paleolandslide and the much larger Cliff Shelf paleolandslide. Base map imagery is georeferenced Google Earth imagery from 2016. Inset shows the approximate location of the complex within the park boundaries.
3.1.2 Geologic setting

Badlands National Park is named for the badland topography that dominates the landscape in this portion of southwestern South Dakota. Badlands refers to a heavily dissected and channelized landscape created as the result of erosion in poorly consolidated sediments (Stoffer 2003). The erosion can occur due to rain, surface water flow or groundwater flow and create large areas of intricate channels and ravines. The badlands, in this location, are bounded by the Cheyenne River to the north and west, the Bad River to the east and the White River to the south. The Cheyenne and White Rivers, specifically, have played a role in the formation of the Big Badlands.

The oldest rocks in the park are made up of the fine-grained sediments of the Cretaceous age Pierre Shale which was deposited approximately 75 million years ago when the Western Interior Seaway covered a large portion of what is now the Great Plains. In the North Unit of the Park, the Pierre Shale is exposed in the vicinity of Sage Creek Campground (Benton et al. 2015) and generally only at the bottom of the deepest gullies on the south side of the Badlands Wall (Kiver and Harris 1999). Sage Creek is located approximately 18 km along Sage Creek Rim Road west of Highway 240 and the Pinnacles Entrance. Figure 3-3 shows a very generalized sequence of the rocks exposed in the park.

Overlying the Pierre Shale are the thin silty shales and fine-grained sandstones of the Fox Hills Formation. These sediments were deposited within the delta of a river system flowing into the Western Interior Seaway, and based on the fossil record, concluded a period of deposition that ended approximately 67 million year ago (Benton et al. 2015). During the beginning of the Tertiary, the Western Interior Seaway drained away and uplift during the Laramide Orogeny resulted in a prolonged period of soil formation and erosion. Benton et al. (2015) describe the climate during this time as having humid and tropical conditions. This time period, lasting through the early Tertiary until the late Eocene, provided a nearly 30 million-year gap in deposition which did not resume in the Badlands region until around 37 Ma.

The majority of the exposed rocks in Badlands National Park are part of the White River Group, which unconformably overlies the Fox Hills Formation, and were mainly deposited 23-35 million years ago (Stoffer 2003). The White River Group consists of the Eocene Chamberlain Pass Formation, the Eocene Chadron Formation, the Oligocene Brule Formation, and the Oligocene Sharps Formation. Additionally, a thick layer of ash known as the Rockyford Ash lies
between the Brule and the Sharps Formation. Due to its lateral continuity, the Rockyford Ash is considered a prominent geologic marker at the bottom of the Sharps Formation. White River Group rocks are made up of sediments eroded from the ancient core of the Black Hills (Stoffer 2003) and volcanic ash and dust (Evanoff et al. 2010). Benton et al. (2015) state that the volcanic sediments were mainly transported to the Badlands by wind and water from the west where massive volcanic eruptions were occurring in what is now the Great Basin of Utah and Nevada. In addition to the fluvial and aeolian deposits, the White River Group also has deposits of fluvial origin including stringers of freshwater limestone. Due to the volcanic origin of many of the sediments in the White River Group, the rocks and soils can contain abundant amounts of bentonite clays. According to Van Houten (1953), those specific clays are montmorillonite and illite.

Figure 3-3: Simplified stratigraphic sequence of the sedimentary rocks in Badlands National Park (NPS 2020). Some of the smaller formations are omitted and many of the formations are subdivided into members.

Deposition in the Badlands is thought to have ceased around 660,000 years ago when the Cheyenne River pirated the headwaters of the ancestral Bad and White Rivers (Stamm et al. 2013). Downcutting and widening of flood plains, specifically the Bad, Cheyenne and White Rivers, is partly responsible for the cliffs and escarpments located along the river valleys in the area (Stoffer 2003; Benton et al. 2015).
3.1.2.1 Landslides within Badlands National Park

Slope instability has been a persistent problem in many locations around Badlands National Park. Movement within the Cedar Pass Landslide Complex was noted as early as 1920 by H. R. Wanless from Princeton University (Wanless 1920). He observed at least two small lakes located in the Cedar Pass area and noted that they were likely formed by landslides which had disrupted drainage networks. Another area of noted instability is along Norbeck Ridge located approximately 10 km west of Cedar Pass along the highway. A study conducted from 2010 to 2011 identified one of the landslides along Norbeck Ridge as a rotational slump with only periodic movement (Baldauf et al. 2011). In general, the typical mode of failure within the badlands of western South and North Dakota is slump or earthflow (Trimble 1979; Gonzalez 2010), however a translational failure mechanism has also been proposed for the Cedar Pass Landslide Complex (Kumar & Associates 1998; Kumar & Associates 1999; Anderson et al. 2004).

3.1.3 Geology of the Cedar Pass area

3.1.3.1 Stratigraphy

The landslide material within the Cedar Pass Landslide Complex is stratigraphically (from bottom to top) derived from the top 20 meters of the Upper Scenic Member of the Brule Formation and the bottom 52 meters of the Lower Poleslide Member of the Brule Formation (Ellen Starck, personal communication, December 2017). The Brule Formation was originally named by Darton (1899) and divided into the higher Scenic and lower Poleslide Members by Bump (1956) with type sections located to the southwest of the North Unit at the town of Scenic and Poleslide Canyon on Sheep Table Mountain. Both the Scenic and Poleslide members can be further subdivided into upper, middle and lower as described by Bump (1956) in his descriptions of both type sections. Benton et al. (2015) describe the upper Scenic as dominated by grey to brown mudstone beds. They also describe the lower Poleslide as dominated by massive, thick siltstone beds with blanket sandstones and an interval of mudstone. The description goes on to state that a distinguishing characteristic of the lower Poleslide in the eastern section of the park is that this portion of the Poleslide is composed of twelve stratigraphic units that can always be found in the same order with the same diagnostic features. Evanoff (2003a) produced a detailed
stratigraphic section for these units in which he identified most of the units as siltstone with occasional interbedding of mudstone and occasional more prominent bench and ledge forming sandstone layers with thin, interbedded red to red-brown mudstones. Capping the lower Poleslide in the Cedar Pass area is a distinctive white, silty unit informally known as the Cedar Pass white layer (Benton, et al. 2015). Displaced portions of this unit can be easily seen in the scarp of the larger slumps in the Cedar Pass area.

3.1.3.2 Faults/Structures

The stratigraphy within the Park is generally horizontal with a regional dip of about 1-2 degrees (Smith 1958). Stoffer (2003) states that there are a few faults present but they generally only show offsets on the order of a few meters. The most prominent structure in the North Unit is the Sage Arch, also called the Sage Creek anticline/fault system (SCAFS) by Stoffer (2003), which is a northwest to southeast trending fold and normal fault system that runs along Sage Rim Road to the west all the way through the Cedar Pass area to the east (Benton et al. 2015). Smith (1958) describes four normal faults near Cedar Pass with maximum offsets of approximately 5-8 meters. However, it is unclear where exactly these faults are located. Geologic mapping conducted within the Cottonwood and Interior quadrangles by Raymond and King (1974) show the nearest fault to Cedar Pass located a little under ½-miles to the southwest from the summit of the pass. A geotechnical report produced by Central Federal Lands Highway Division (CFLHD) of the Federal Highway Administration (FHWA) in 2013 regarding a landslide in the Cedar Pass area states that, “there are no Quaternary faults mapped within the general vicinity of the project site.”

3.2 Badlands Loop Road maintenance history

The road through Cedar Pass was first constructed in 1935. Construction was completed by the South Dakota Highway Department of Transportation at the request of Ben Millard and Senator Peter Norbeck, who played instrumental roles in the establishment of Badlands National Monument (Shuler 1989). The difficulty in building and maintaining a road through the Badlands seemed to be recognized by highway engineers at that time as one engineer is quoted as saying, “Because this project was constructed through an area recognized as one of the
world’s most fantastic examples of erosion, it is expected that maintenance will be neither simple nor easy” (Shuler 1989).

Maintaining the Cedar Pass portion of Badlands Loop Road has created a financial burden for the Park over the last three decades. From 1990-1997, material costs for maintenance were on the order of $12,000/year (Kumar & Associates 1999). From 1997 through the early 2000s, costs increased to approximately $20,000/year (FHWA 2002). From 2003-2013, the Park spent roughly $1,000,000 (~$100,000/year) on projects related to the highway incline to Cedar Pass (FHWA 2013). These maintenance costs apparently do not include the additional expenses related to construction of mitigative structures and other improvements including a $14 million United States Department of Transportation project in 2000 to stabilize a portion of the highway at Cedar Pass detailed in Kumar & Associates (2000).

It was not until the 1950s that the highway was graded and paved. In 1958, a scenic overlook was constructed immediately south of Cedar Pass on an embankment fill on the downslope side of the road. The construction of the overlook also corresponds to the first observed signs of slope stability issues at Cedar Pass (Kumar & Associates 1998). A geotechnical report prepared by Parsons Brinckerhoff Quade & Douglas, Inc. in 2004 states that the highway had to be resurfaced in the summer of 1967 after the road surface across the slump block of the Cedar Pass Landslide dropped 15 centimeters (6 in). In 1990, slope movements were causing damage to the scenic overlook and by 1993 the overlook was abandoned (Kumar & Associates 1999). By the late 1990s, the road surface across the landslide at the Cedar Pass summit was being kept as gravel. The decision to leave the highway with a soft surface was made after damage from landslide movement to at least two previous road surface overlays (Kumar & Associates 1999).

In 2000, an 80,000 m$^3$ buttress was constructed on the slope below the old overlook and along a short portion of the highway to arrest slope movement in that area. The buttress appeared to have sufficiently slowed or stopped movement as an Interferometric Synthetic Aperture Radar (InSAR) survey conducted in the late 1990s to the early 2000s showed no appreciable movement after the construction of the buttress (Anderson et al. 2004).

Maintenance and construction work shifted to the Cliff Shelf area during the next 15 years, specifically to the portion of the highway directly west of the Cliff Shelf parking lot that had been experiencing slope movement and settlement. Sometime before 2012, the road was
resurfaced and a deep patch was installed in the project area as indicated in a technical memorandum produced by the Federal Highway Administration (2012). However, the exact date and location of this project could not be verified by the author. Multiple patching and resurfacing projects culminated in the construction of another earthen buttress and new stormwater collection and conveyance system in 2015. The stormwater system was designed to collect all surface water on slopes draining towards the road above the Cliff Shelf parking lot as well as all highway and parking lot runoff and transport it to an outlet at the base of the slope below the buttress. The earthen buttress was built to stabilize the failing slope below the highway and protect the road from future movement. In addition to the buttress, the highway was reconstructed with a deep patch across the top of the buttress to provide a stable base for the highway and assist in the drainage of groundwater beneath the road. Ongoing road surface distress in the vicinity of the buttress has become the focus of investigations in the past few years. Most recently, crack sealing and asphalt grinding to smooth bumps in the asphalt have been the primary maintenance operations.

3.3 Overview of previous geotechnical studies

The Cedar Pass Landslide Complex is located roughly 1-kilometer to the northeast of Park Headquarters and 6 road kilometers from the Northeast Entrance along Badlands Loop Road. The complex consists of two paleolandslides identified as the Cedar Pass Landslide – also called the Bowl Landslide in post-2016 maps – and the larger Cliff Shelf Landslide (Kumar & Associates 1998) (See Figure 3-2). Kumar & Associates (1998) states that these landslides had been previously observed and described in 1993 and 1996 by engineering geologist Dr. Perry Rahn and South Dakota Department of Transportation (SDDOT) engineer Vernon Bump. The exact date of these larger paleolandslides is unknown, however one report states that they occurred, “several hundred years ago” (Parsons Brinckerhoff Quade & Douglas, Inc. 2004), while interpretative signs along the Cliff Shelf Trail located in the landslide complex suggest that the slumps occurred on the order of thousands of years ago. Rahn and Bump describe the Cedar Pass Landslide as consisting mainly of bedrock blocks and debris. This is supported by the presence of bedrock blocks still containing defined bedding planes observed within the landslide mass as well as a large (5-6 meter-wide) block with bedding rotated out of the horizontal sitting directly below the head scarp just south of the Cedar Pass summit. Rahn and Bump also suggest
that other landslides in the area are likely made up of bedrock material and debris from rockfall initiating along Millard Ridge, which is a tall cliff thought to be the head scarp of both paleolandslides, and a named section of the Badlands Wall. The uppermost failure point of the landslides could be N 65 W to N 80 W striking, near-vertical joints along Millard Ridge that provide a discontinuity on which bedrock blocks are able to separate from intact portions of the Millard Ridge (Kumar & Associates 1998).

Focusing on the reactivation within the Cedar Pass Landslide near the summit, and based on exploration of the subsurface of the slide, Kumar & Associates (1999) states that the shear plane is likely 15-20 meters below the surface with a dip of approximately 2 degrees. The shallow dip angle is attributed to the shear plane possibly following the low angle dip of bedding within the underlying Brule Formation. As a result, it is thought that movement of the Cedar Pass Landslide is mainly translational, which has many of the typical characteristics of translational rock landslides outlined by Glastonbury and Fell (2008). They suggest that translational rock slides generally occur in horizontally-bedded, sedimentary rocks with a near-horizontal rupture surface. They go on to say that translational rock slides contain more intact rock near the head of the slide with more disaggregation occurring near the toe.

Observations of groundwater within the Cedar Pass slide range from approximately 3 meters below the surface to greater than 25 meters, with the shallower depths located in uphill portions of the slide near the highway and Millard Ridge (Kumar & Associates 1998). Variations in groundwater levels have been attributed to seasonal fluctuations in precipitation including rainfall and snow melt and local drainage patterns. Perched zones may exist due to the aforementioned factors and the subsurface drainage characteristics (Yeh and Associates 2016).

The much larger Cliff Shelf paleolandslide was thought to be dormant through the late 1990s; however, field observations by engineers and subsurface exploration and monument surveys in 1998 and 1999 note reactivation of the slide mass and localized areas of settlement in the highway, possibly related large-scale landslide movement. The sliding mechanism of this landslide is thought to be complex due to its large size, and as of the year 2000, geotechnical investigations were unable to establish the depth of the slide plane or the sliding mechanism (Kumar & Associates 2000). Subsequent investigations of the Cedar Pass and Cliff Shelf landslides in the late 1990s and early 2000s confirmed that translational movement was likely the predominant movement type across the entire complex (Anderson et al 2004).
In the late 2000s and early 2010s, attention shifted to the Cliff Shelf paleolandslide with the onset of highway surface distress in the area and other observations of movement based on ground and air surveys. Several investigations between 2010 and 2016 attempted to identify slide planes of several suspected landslides in the Cliff Shelf. Investigation included borings (FHWA 2013; Yeh & Associates 2016) and geophysical methods (Zonge International 2013). At a location directly downhill and west of the Cliff Shelf parking lot, two slide planes were identified at approximately 10 and 15 meters below the ground surface. These depths are slightly shallower, but similar to the estimated slide plane of the Cedar Pass Landslide. A significant slope failure in 2013 at the same location was mitigated with another earthen buttress.

The consensus among investigators has been that periods of above normal precipitation increased groundwater levels and the saturation of highly plastic clay and claystone layers in the subsurface units, is the major cause of landslide movement in the Cedar Pass Landslide Complex (Kumar & Associates 1999; Kumar & Associates 2000; Anderson et al. 2004; FHWA 2012; FHWA 2013). Field observations, as well as monitoring data, show increased movement during the months after a period of wetter than normal conditions. Specifically, this trend is noted in 1998 and 2011, when more significant movement of landslides in the area were directly preceded by several years of either normal or above normal precipitation (Figure 3-4).

See Appendix A for detailed summaries of each geotechnical survey.

### 3.4 Slope stability

Slope stability, in general, is the measure of the instability or stability of a natural or engineered slope based on a variety of factors including geologic conditions, site topography, climate and material properties. When a slope is unstable, ground movement can occur. Slope stability analyses often use a single value known as the factor of safety (FS) to provide a numerical approximation of stability. The factor of safety is the ratio between forces resisting ground movement and forces driving ground movement (Equation 3-1). Therefore, when the FS value is greater than 1.0, the resisting forces are greater than the driving forces and the slope may be considered stable. When the FS value is equal to 1.0, driving and resisting forces are equal and the slope is at equilibrium. Any value less than 1.0 means that failure has occurred (or is expected to occur), assuming there is no change in conditions.

\[
\text{Factor of Safety} = \frac{\text{resisting forces}}{\text{driving forces}} \tag{3-1}
\]
3.4.1 Numerical modeling of slope stability

Many slope stability analyses use the concept of limit equilibrium to calculate stability. Limit equilibrium exists when the shear stress along a failure surface is equal to the shear strength of the surrounding material. This ratio can also be represented by a factor of safety, with a value of 1.0, equaling unity (Equation 3-2). The shear stresses along a failure surface as well as the shear strength of soils within a landslide can be impacted by a variety of factors. An estimate of shear strength can be made by field observations or more accurately, by laboratory testing.

Most methods for slope stability analysis utilize some version of the method of slices to divide a slope into either a predetermined or user-chosen number of individual slices in order to calculate a factor of safety. Each of those slices is acted on by a system of forces and is treated as a unique block. The bottom and top of the area divided into slices are the slope surface and shear plane, respectively. Slope stability software will iterate through many different slide planes until
the surface with the lowest factor of safety (called the global minimum) is found. The global minimum is the surface that defines the bottom of a mass of soils that exhibits the lowest ratio between resisting and driving forces. In order for calculations to be carried out, slope stability software requires three pieces of typical input data including the following: geologic conditions such as groundwater and stratigraphy, site topography, and material properties such as shear strength.

\[ FS = \frac{S}{\tau} \]  

(3-2)

Where \( \tau \) = the shearing stress along failure surface  
\( S \) = the shear strength of soil  
\( FS \) = the factor of safety

Stability modeling of slopes in the landslide complex has been carried out as part of two investigations (Kumar & Associates 1999; FHWA 2013) to evaluate the effectiveness of earthen buttresses as a mitigation technique at two specific locations along the Badlands Loop Road. However, other potential landslides possibly impacting the highway have not been modeled. The shear strength of landslide materials used in these stability analyses has been estimated through back analysis of stability models and empirical correlations with material properties such as clay content and Atterberg limits. Shear strength has not previously been tested for in the laboratory due to the lack of success in identifying and sampling slide plane materials during boring operations.

### 3.5 Behavior of fine-grained, cohesive soils

A large portion of the geologic units found in Badlands National Park consists of fine-grained, cohesive materials. The behavior of these materials has implications for the stability of hillslopes in the park and the initiation and behavior of landslides. The shear strength of these materials plays an important role in their behavior and is an important input parameter for slope stability analyses. The Mohr-Coulomb failure criterion includes the cohesion and angle of friction of the soil, which are two main components of shear strength of a soil (Equation 3-3 and Equation 3-4). These two parameters can be measured by shearing samples of soil to failure under different normal stresses and plotting the maximum shear stress versus the applied normal stress and approximating the best-fit line to the plotted points. The angle of the line from the horizontal provides the angle of friction and the intersection of the best-fit line with the shear
stress axis provides the value of cohesion. Shear strength can be expressed in terms of total stresses and effective stresses. Effective stresses take into account pore pressures in the soils that decrease the stresses borne by the soil particles themselves.

\[ S = c + \sigma_n \tan \varphi \]  
\[ (3-3) \]

Where:
- \( S \) = the total shear strength of the soil
- \( c \) = the total cohesion of the soil
- \( \sigma_n \) = the total normal stress
- \( \varphi \) = the total internal friction angle of the soil

\[ S' = c' + (\sigma_n' - u) \tan \varphi' \]  
\[ (3-4) \]

Where:
- \( S' \) = the effective shear strength of the soil
- \( c' \) = the effective cohesion of the soil
- \( \sigma_n' \) = the total normal stress
- \( u \) = the pore water pressure
- \( \varphi' \) = the effective internal friction angle of the soil

In cases where soils experience relatively large amounts of shear strain, the residual shear condition may be reached. This is often the case in reactivated landslides. Additionally, a drained condition usually exists along a reactivated shear plane that has reached the residual condition (Terzaghi et al. 1996). This is thought to occur because of the combination of the relative thinness of the shear plane and that pre-sheared clay particles have a tendency to realign and become oriented parallel to the direction of shear, preventing the clay particles from changing volume and developing excess pore pressures (Stark et al. 2005). Several factors can influence the residual strength behavior as noted in Abramson et al. (2002). Those factors are the clay content of cohesive soils, and the proportion of platy particles versus spherical particles (i.e. the proportion of clay minerals versus that of particles with diameters less than 2 microns but that are not clay minerals). The use of residual strength for modeling soils in the landslides in the Cedar Pass Landslide Complex is appropriate because the landslides have been moving semi-continuously for at least the last 5 years and because some of the landslides may be failing along reactivated surfaces within the paleolandslides.

While measuring shear strength directly is the preferred method, this is not always possible. Therefore, various attempts have been made to correlate residual friction angle with other, more easily measured material index properties such as Atterberg limits. Studies including
Stark and Eid (1994) and Wesley (2003) suggest that the use of empirical correlations relying on more than one soil index property provides a more accurate estimation of the residual friction angle. This thesis utilizes an empirical correlation developed and revised by Stark and Eid (1994), Stark et al. (2005), and Stark and Hussain (2013). This correlation was also used by Kumar & Associates (1999) during their investigation of the Cedar Pass Landslide to estimate the residual friction angle of the claystone unit in which the slide plane of that landslide was hypothesized to exist.

Stark and Eid (1994) tested 32 different clays and clayshales and found that the drained residual failure envelope is non-linear and impacted by both the liquid limit, clay fraction and effective normal stress. They concluded that because the liquid limit is a good indicator for clay mineralogy, clay fraction indicates the number of particles in the soil smaller than 0.002 mm, and the effective normal stress influences the interaction between clay particles, these three parameters are important in estimating the residual friction angle. The empirical correlation they created improved upon previous correlations because most previous studies introduced correlations that were stress independent and usually based on only one soil index property. For slope stability analyses, Stark and Eid (1994) recommended the residual strength be modeled using a non-linear failure envelope (Figure 3-5).

Stark and Hussain (2013) expanded upon the empirical correlation developed by Stark and Eid (1994) by conducting more laboratory testing to increase the number of data points and by producing trend line equations; previous versions of the correlation (Stark and Eid, 1994; Stark et al, 2005) were only available in graphical form. The addition of the equations allows for the direct calculation of the drained residual friction angle for the different groups of data based on the effective normal stress, clay size fraction and liquid limit. The only input into the equations is liquid limit, but effective normal stress, clay size fraction and liquid limit need to be considered when choosing the appropriate equation.
Figure 3-5. The relationship between liquid limit and residual friction angle from Stark and Eid (1994). This graph was used by Kumar and Associates (1999) to estimate the residual friction angle of claystone bedrock during the investigation of the Cedar Pass Landslide and an updated version of this correlation was used in this study.
CHAPTER 4
METHODS

4.1 Field reconnaissance

Fieldwork consisted of landslide mapping, topographic profile construction, and soil sampling. Collected samples were used for laboratory tests to estimate shear strength, grain-size distribution, and plastic and liquid limits. Seventeen soil samples were collected from within the landslide complex at the surface using a 63.5 mm (2.5 in) diameter, 152 mm (6 in) long brass sample tube. Fourteen samples were extracted from four different geologic units and 3 samples were collected from an earthen buttress (2) and highway embankment fill (1). Samples were collected from units expected to contain the slide planes based on the stratigraphy observed in intact buttes adjacent to the landslides. These units include the mudstones/claystones of the Upper Scenic Member of the Brule Formation and the siltstone of the Lower Poleslide Member of the Brule Formation. All sampled units are expected to be stratigraphically above the Disappointment limestone interval identified by Evanoff (2003b). See Figure 4-1 for sample locations and an example of stratigraphy of an intact butte. Confidence in matching slide plane units to the geologic units sampled is low because of the disturbed nature of the stratigraphy in the landslide complex and the similarities in physical characteristics of the different geologic units. Table 4-1 presents the individual samples collected and the geologic units or features where they were collected.

Table 4-1. Collected samples and the geologic units from which they were extracted.

| Sample No. | 1  | 2  | 3  | 4  | 5  | 6  | 7  | 8  | 9  | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 |
|------------|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| Unit       | Nodular mudstone | 2000 Buttress | Nodular mudstone | Upper Scenic mudstone/claystone | Lower Poleslide siltstone | Poleslide siltstone | Highway Embankment |

The sampling procedure began by scraping away the desiccated (‘popcorn texture’) material located at the surface. In most cases, the area from which the specimen was extracted was soaked with water to make it slightly easier to drive the sampling tubes. The sample tube was driven into the ground with a sledgehammer to the full length of the tube or until there was enough resistance to prevent the tube from penetrating any farther. Multiple samples from
different units were collected to ensure enough material was available for the different laboratory tests.

Figure 4-1. Soil sample locations. The inset shows stratigraphy on an intact butte called the Ridge of Resistance. Individual units in the Lower Poleslide differentiated by Evanoff (2003a).

Mapping included identifying landslide-induced geomorphic features as well as other features that may contribute to slope movement. Those features included scarps, tension cracks, areas of hummocky topography, areas of standing water or where water could accumulate, and areas of erosion and drainage channels. Additionally, damage to park infrastructure including asphalt cracking, pavement offset, broken curbs, and other signs of deformation was recorded. Mapping began with a review of historical imagery available from Google Earth. Additionally, the Park Service provided a set of 1:2300 high-resolution aerial photographs taken in 2003 that were reviewed in stereo to help focus mapping efforts and to identify prominent landslide features. All accessible areas of the landslide complex were observed and mapped, but mapping
focused on areas in and around concentrations of park infrastructure damage where unmitigated
landslides were suspected to exist. Those landslides were identified by the Park Service as the
Upper and Lower Wedge Landslides, Lateral Shear Landslide, and the Cliff Shelf Parking Lot
Landslide (see Figure 2-1).

An important slope stability model input is the topography along a cross-section of the
landslide oriented parallel to the predicted direction of movement. A total of 14 different
topographic profiles were collected in the field in different orientations so that various directions
of slope movement could be analyzed if necessary. A single profile was surveyed through each
of the smaller landslides including the Upper and Lower Wedge Landslide and the Lateral Shear
Landslide. Figure 4-2 shows the locations of the topographic profiles collected and used for
slope stability analysis. Three profiles were surveyed through what this study is calling the
Prairie Island Landslide located south of the Cliff Shelf Trail because of its relatively larger size
and because more profiles would be needed to fully characterize the landslide mass and to
calibrate sections of the landslide to each other.

Topographic profiles were surveyed using a slope profiler to collect slope angles over 1-
yard intervals. The slope profiler used in this study was a 1 yd-long wood board connected to
two legs of equal length. The slope profiler was moved in 1 yd increments and the average slope
angle over each increment was measured by placing a Brunton compass on top of the profiler
and measuring the angle. Profiles were collected in as straight of a line as possible. If an
obstruction such as a tree was encountered, the slope directly adjacent and parallel to the profile
was measured until the obstruction was bypassed. Some profiles extended into highly eroded
areas (badlands topography) that were too difficult to access, so a 1/3 arc second (approximately
10-meter) digital elevation model (DEM) was employed to create those sections of the profile
using ESRI ArcMap 10.7.1. In areas of badlands topography, an attempt was made to record the
average elevation as badlands are dominated by deep, narrow channels and overall rugged
terrain. The use of the 10-meter DEM eliminated topographic features only present along profiles
and not in adjacent areas because those features were generally smaller than 10 meters across.

4.2 Laboratory testing

A series of tests were conducted on soil specimens collected in the field in order to
characterize the physical and strength properties of the slide materials. Density/unit weight and
direct shear tests were performed to provide input data for numerical modeling. Atterberg limits and grain-size distribution were measured to calculate friction angle using an empirical correlation and to compare values obtained from the correlation to values measured by direct shear testing. The natural moisture content of each sample was not recorded because, in most cases, the sampling procedure included the addition of water.

Figure 4-2. Location of topographic profiles used for slope stability modeling. Filled landslides are those mapped in this study. Hashed red lines are approximate boundaries of landslides mapped by the Park Service that could not be identified in this research. Contour lines are in 3 m intervals. Profile names indicate the landslide through which they were surveyed.

4.2.2 Dry density/unit weight

Dry density was measured following the direct measurement procedure described in ASTM D7263. Samples held within the sampling tube were allowed to dry for an extended
period of time (generally more than 24 hours) in an oven at 112 degrees centigrade. After drying, the sample volume was determined by calculating the inner dimensions of the sample tube and the height of the empty portion of the sample tube. A consistent sample height was achieved by trimming the sample top to produce a reasonably flat surface. The volume of the empty portion was subtracted from the total volume of the sample tube to obtain the volume of the sample.

4.2.3 Atterberg limits

The liquid and plastic limits of 15 soil samples were measured following the general guidelines provided by ASTM D 4318. Most samples were prepared using the dry preparation method. Samples were air-dried and then pulverized with a mortar and pestle. In some cases, the sample was too hard to pulverize by hand and so the sample was soaked in water, broken apart, and then transferred to a pan where it was allowed to dry. The dried pieces could then be pulverized and sieved.

4.2.4 Drained residual direct shear

ASTM D 3080 guidelines were followed as much as was reasonable, but with some specific differences, mainly the rate of shearing. The general procedure used for this test is as follows:

An ELE International Digital Direct/Residual Shear Apparatus was used to carry out the direct shear testing (Figure 4-3). The desiccated nature of the samples required the specimens to be remolded into the shear ring (2.5-inch diameter). Samples were remolded at the plastic limit of the material as calculated from preceding consistency testing. Enough material was used so that the sample dry density would approximately match the average dry density of the material. The sample was compacted in three lifts in order to achieve the desired 1 in. sample thickness. A single sample was used throughout the whole test instead of replacing the specimen with a new one after each normal stress step. This method was deemed appropriate given the fact that residual strength values were desired and that shearing the same sample throughout the entire test would help the sample reach residual conditions. After the specimen was seated in the shear ring, the sample was flooded with water, and the first normal load was applied and the sample was allowed to consolidate for a period of at least 24 hours. Twenty-four hours was generally considered long enough based on observation of the vertical displacement dial. Submersion in
water was performed in order to simulate saturated conditions expected along the shear plane. After consolidation, the sample was sheared at a rate of 0.5 mm/min (0.02 in/min). This rate was chosen based on a study by Walker (1999), where drained residual shear testing was conducted on various soil and rock materials including clayey soil. By comparison, the suggested rate based on recommendations in ASTM D3080 is approximately an order of magnitude smaller. The faster rate was chosen for this study because it was shown to produce reasonable results in Walker (1999) and because the time requirement to run tests at the lower rate were deemed unreasonable given the available laboratory equipment. Furthermore, this rate of movement is expected to be similar to the irregular movements of the landslides at the site observed over time. The direct shear machine used in this study did not have a data logger, so measurements of the proving ring and horizontal and vertical displacement dials had to be taken by hand. Each group of tests for each normal load lasted between two and four hours. This amount of time would have increased to more than 12 hours if the slower rate was used. Shearing was stopped after there was no appreciable increase in shear stress over a total horizontal displacement of 0.06 in. It was assumed that an increase in shear strain not accompanied by an increase in shear stress indicated the sample had failed.

Figure 4-3. The direct shear machine used in this study with labels showing key components.

Horizontal displacement values at the failure point and when shearing was stopped generally ranged from 5-18% of the sample diameter. The shear direction was then reversed and the sample was quickly brought back to the original starting position. This process was repeated
a total of five times. After the fifth trial, the normal load was increased and the sample was allowed to consolidate for at least an additional 24 hours. Shearing was repeated an additional five times after which a final normal load was applied and the sample was allowed to consolidate. After five final trials of shearing, the sample was removed and dried in an oven so that the final water content of the sample could be measured. Samples were sheared for a total of 120-300% of the diameter of the sample. Including forward and backward motion, samples were sheared 240-600% of the sample diameter which equates to roughly 120-300 mm of total shear strain. Preliminary slope modeling showed the slide plane could be as deep as 40-45 meters. Therefore, an overburden of 40-45 meters was bracketed during direct shear testing.

The value of the shear stress at sample failure during each trial was taken and plotted against normal stress. Because the last four trials for each normal stress step produced relatively consistent results, the cohesion and friction angle values were computed using the values from the last four trials, while excluding data points from the first trial of each stress step.

The testing procedure described above includes several inherent assumptions including that remolded samples can be reasonably remolded to their in-situ density, any material lost during shearing (i.e. the volume of soil that squeezes out of the box during testing) does not have a significant impact on the measured strength, and the consistent peak identified in the last 4 trials approximates the residual shear stress.

Specific limitations of this approach include the fact that the sample is sheared in two directions instead of a single direction. Shearing in a single direction is preferred to reach residual conditions but the direct shear apparatus used in this study uses a forward and backward motion and can only measure shear stress in a single direction. Lastly, analog dials, which are moving during the test, are measured by visual inspection and therefore the measurements recorded are only approximate.

4.2.5 Hydrometer analysis

A hydrometer analysis was carried out on eight samples to calculate the clay fraction of the samples. Procedures outlined in ASTM D 422 were followed. In typical hydrometer tests, the material retained on the #200 sieve is dried and run through a stack of sieves to determine the grain size distribution of the material retained. However, for this study only clay and silt contents
were needed to characterize the soils because such a small percentage of the soil could be classified as a coarser material.

4.3 Numerical modeling of landslides in the complex

In order to assess the current stability, sensitivity to various input parameters, and effectiveness of potential slope mitigation methods, computer modeling was conducted using 2D limit equilibrium methods with RocScience Slide Version 2018 8.024. A 2D modeling approach was used because of the lack of topographical data required to create a 3D model.

Three different limit equilibrium methods were used to calculate the factor of safety and draw the global minimum surface. Based on the assumed planar shape of the slide plane for the different landslides, Spencer’s, GLE/Morgenstern-Price, and corrected Janbu method were used. These methods satisfy force equilibrium in both the x- and y-direction, and moment equilibrium. Abramson et al. (2002) suggests that for slide planes of an arbitrary shape, these methods can produce more accurate results. Ultimately, for the sensitivity analysis and analysis of mitigation options, the GLE/Morgenstern-Price method was used because it consistently provided the lowest FS of the three methods used. Using the lowest FS provides the most conservative evaluation of stability, which is appropriate for practical hazard assessments. In cases where the slide plane had a circular shape, the Bishop method was used.

Modeling was conducted for four landslides within the Cedar Pass Landslide Complex: the Prairie Island Landslide and Lateral Shear Landslide mapped in this study, and the Lower Wedge and Upper Wedge Landslide identified by the Park Service (see Figure 2-1). An additional model was created for the revised boundaries of the Upper Wedge Landslide based on this study.

4.3.1 Back analysis of model parameters

The creation of a slope stability model requires the input of a variety of parameters such as topography, water table elevation, stratigraphy, slide plane geometry, and the material properties such as unit weight and strength. Topography for each pre-failure profile was created using data collected in the field and from a 1/3 arc-second DEM (USGS 2013). The initial water table elevation was estimated based on a combination of field observations and borehole data (Kumar & Associates 1999; FHWA 2013; Yeh & Associates 2016). Initial Mohr-Coulomb
strength parameters were estimated based on results of laboratory tests carried out on samples collected for this study. In cases where the highway crossed the profile, a distributed load of 12 kN/m² was added to simulate traffic loading (FWHA 2013).

Due to the nature of these landslides being located within older landslide deposits, and the fact that original bedding has possibly been disrupted and is no longer intact, the stratigraphy of each profile was constructed from field observations and any relevant borehole data collected during previous geotechnical investigations. Field observations were relied on more heavily along profiles for which there was little geotechnical information available (e.g. profiles D-D’, E-E’ and F-F’).

In Slide software, a block search was used to constrain the slide plane geometry in every landslide investigated. A block search is a method to find a non-circular failure surface and is one way to force a general slide plane shape in the model. A block search uses one or more user-defined points, polygons, or polylines and the computed slide planes are forced to pass through these objects. In this study, a polygon was typically added near the head scarp. Entry and exit angles, which specify the range of angles at which possible slide planes can enter and exit the polygon, were defined by the author, but left with large enough ranges to provide some flexibility in the model. In the case where a distinct slide plane was located in the field, as was the case for the Lateral Shear Landslide, a polyline was added at that depth to force the slide plane created by the model to pass along that plane.

Three profiles for the Prairie Island Landslide were used to iteratively adjust slope stability model input parameters until they converged on specific material strengths, slide plane geometry and water table depth. The usefulness of employing a similar methodology of characterizing multiple unknowns within a slope model by analyzing more than one cross-section has been demonstrated by Santi (2014) and Scheevel (2017). An average factor of safety of 1.0 across all three profiles with individual values within 10% of 1.0 was desired. Back calculated strength values from the Prairie Island Landslide were used in the models for the three other landslides because of the similarities between soils present at each location. A single profile through the Upper and Lower Wedge landslides and the Lateral Shear Landslide were used to create the slope stability model for those slides.
4.3.2 Sensitivity analysis

A sensitivity analysis was carried out to evaluate the impact to the overall stability of changes in material properties, fluctuation of groundwater, and erosion within and near the toe. Material properties including unit weight, cohesion, and friction angle were individually and incrementally increased and decreased by 100% of the base value in order to calculate the percent change in the FS. To assess the impact of fluctuating groundwater levels, the water table was raised and lowered 1 m at a time, with the minimum water table depth coinciding approximately at the ground surface and the maximum water table depth below the slide plane. The maximum water table depth varied depending on the landslide. The sensitivity of the landslide to erosion at the toe was simulated by incrementally lowering the topography in the areas with little to no vegetation, which usually coincided with heavily channelized areas near the toe of the landslide. Stetler (2014), who studied erosion rates in Badlands National Park, notes that vegetation coverage, among other factors, can have a significant impact on the amount of erosion. Therefore, erosion in grass covered and other heavily vegetated areas was considered to be insignificant compared to erosion in barren areas. The topography was lowered by increments of 1.27, 2.54, 5.08, 10.16, 20.32, 30.48, and 60.96 cm (0.5, 1, 2, 4, 8, 12, 24 inches). Estimated average annual erosion rates within the park are 2.54 cm (1 in) (Stoffer 2003; Benton et al. 2015; NPS 2020), so simulated erosion equated to between 0.5 years and 24 years of erosion. The water table in the area of lowered topography was also lowered by the same amount in order to maintain a consistent water table depth.

4.3.3 Modeling the landslide toe when its location is ambiguous

One of the main sources of error when modeling a landslide with ambiguous boundaries is that it is difficult to constrain the location of the main slide plane when the upper and lower boundaries along the 2D profile can have large ranges. Typically, fieldwork or review of satellite or aerial imagery can help identify the head and toe of a landslide. In the case of the landslides investigated in this study, either the toe, the head, or both were difficult to see, even in the field. And without other geotechnical data to help constrain the location of the slide plane, the location of the head and toe is used to define the endpoints of the failure surface in the models. In order to circumvent this issue, the following procedure was developed.
In all models, two sets of limits were defined, with one set constraining the location of the landslide head, and the other containing the location of the toe. In the models for the Lateral Shear Landslide and Prairie Island Landslide, the upper limits were spaced close together in order to force the upper boundary of those landslides to pass through specific features identified in the field, such as a head scarp. In the models for the Upper and Lower Wedge landslides, upper boundaries were not confidently identified, and so the upper limits were left with a wider spacing to provide flexibility. The lower limits were initially left further apart, with one located at the base of the slope and the second located at some point further up the slope. A series of models were then run with different lower limit locations and spacing in order to identify trends in the models and a reasonable global minimum surface. Trends were used to constrain landslide boundary locations. Specific trends included where the most common location of the toe of the landslide was located by the software, and how stability changed with different locations given various degrees of freedom with the models. A model with a tight lower limit has less degrees of freedom than a model with a wide lower limit based on the number of possible slide planes the program can draw.

In the models where both the upper and lower boundaries were poorly constrained, the same procedure was employed, but the upper and lower limits were both adjusted incrementally in order to attempt to define a reasonable failure surface with a similar geometry to the failure surface of the Cedar Pass Landslide approximated from borehole and inclinometer data in Kumar & Associates (1999). Fortunately, in these cases, inclinometer data helped constrain the minimum depth at which a slide plane was located and therefore decreased the total number of possible model solutions.

4.3.4 Analysis of potential mitigation options

Various mitigation options were analyzed, including the construction of an earthen buttress, anchored tieback walls and piles. The effectiveness of these options was investigated through a statistical and probabilistic analysis. A range of values for cohesion and friction angle were assigned based on a combination of back-calculated values and laboratory testing from this study and values used by other investigators (Kumar & Associates 1999; FHWA 2013). This method helps account for some uncertainty in strength parameters. A uniform distribution was assigned to each range because there is not enough data to create a more traditional normal
distribution, which requires a standard deviation in order to define the distribution. Additionally, using a uniform profile means that any value within the assigned strength ranges is equally possible. Soil properties of the material used to construct the buttress were obtained from an FWHA (2013) report in which modeling was conducted to assess the effectiveness of a buttress constructed in 2015. The recommended strength parameters of tieback anchors and piles in Slide were used for those mitigation methods. The only change made was to increase shear strength of the piles to simulate a larger diameter pile.

The probabilistic analysis in this study first found the global minimum surface using the mean material property values and a deterministic analysis and then calculated the FS along that surface 1000 different times using different strength values chosen at random from the distribution of values assigned to each property. This method produces a probability of failure (PF) which demonstrates what percentage of strength property combinations would result in an FS lower than 1.0. This metric was used to quantitatively evaluate the effectiveness of the different mitigation measures by noting the decrease in the probability of failure in a slope once support was added.
CHAPTER 5
RESULTS

5.1 Laboratory testing

All of the samples tested, excluding the embankment fill, have been grouped together for characterization and modeling purposes. This approach was used to provide a range of initial values with the expectation that a single geologic unit would be modeled during the slope stability analysis due to the uncertainty in the stratigraphy in the landslides, and the uncertainty in the identities of the geologic units sampled.

Consistency testing showed that almost all the samples are classified as high plasticity clay (CH) using the United Soil Classification System (USCS) (Figure 5-1). The soils contained 97.5-99.7 percent fines with 35.8-46.6 percent clay. The one exception is the embankment fill sample (Sample 17) which contained only 80 percent fines and 26.8 percent clay. Excluding Sample 17, all of the soils generally showed very similar grain-size distributions (Figure 5-2). The plastic limit ranged from approximately 26 to 33 percent and plasticity index ranged from approximately 35 to 57 percent. These ranges generally agree with values from other studies involving soils in this portion of the park (Kumar & Associates, 1999; FHWA 2013; Zhang, 2013; Yeh & Associates, 2016) Previous work suggests that clay minerals present in the park include montmorillonite and illite which have high and moderate swell potential, respectively (Van Houten 1953).

Direct shear testing to estimate the residual strength properties of the soils was carried out on samples 7, 9, 11, 14, and 17. Shear stress vs. shear strain plots showed responses similar to those of normally consolidated clays (see Figure 5-3). Refer to Appendix B for more comprehensive data related to the direct shear testing. Some samples showed dilative behavior during the initial shearing of the sample, but in general most samples showed contractive behavior.

Total shear strain, including forward and backward movement applied to the samples, ranged from 124.2-301.6 mm. The total applied shear strain was dependent on how fast the sample failed during each trial. Applied normal stresses ranged from approximately 61-1240 kPa, with the maximum and minimum values varying slightly depending on the sample.
Table 5-1. Soil sample index properties obtained from laboratory testing. Samples 3 and 4 were extracted from the 2000 buttress and were excluded from most laboratory tests. Unit 1 is a nodular mudstone, Unit 2 is an Upper Scenic mudstone/claystone, Unit 3 is a Lower Poleslide siltstone, and Unit 4 is a Poleslide siltstone. Fill 1 is borrowed fill from the 2000 buttress, and Fill 2 is highway embankment fill.

<table>
<thead>
<tr>
<th>Unit</th>
<th>Sample No.</th>
<th>Unit Weight (kN/m³)</th>
<th>Atterberg Limits</th>
<th>Grain-size Analysis</th>
<th>USCS Class</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Liquid Limit</td>
<td>Plastic Limit</td>
<td>Plasticity Index</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>15.29</td>
<td>74.6%</td>
<td>32.9%</td>
<td>41.7%</td>
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<tr>
<td></td>
<td>2</td>
<td>15.14</td>
<td>87.1%</td>
<td>30.5%</td>
<td>56.6%</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>14.60</td>
<td>66.5%</td>
<td>27.9%</td>
<td>38.6%</td>
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<tr>
<td></td>
<td>Average</td>
<td>15.01</td>
<td>76.1%</td>
<td>30.4%</td>
<td>45.6%</td>
</tr>
<tr>
<td>Fill</td>
<td>3</td>
<td>15.90</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
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<td></td>
<td></td>
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<td>Average</td>
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<td></td>
<td></td>
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<td>6</td>
<td>14.90</td>
<td>63.7%</td>
<td>28.5%</td>
<td>35.2%</td>
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<td>7</td>
<td>14.70</td>
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<td></td>
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<td>64.8%</td>
<td>28.7%</td>
<td>36.1%</td>
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<td></td>
<td>9</td>
<td>16.62</td>
<td>70.2%</td>
<td>29.8%</td>
<td>40.5%</td>
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<tr>
<td></td>
<td>10</td>
<td>15.84</td>
<td>66.5%</td>
<td>28.1%</td>
<td>38.4%</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>15.75</td>
<td>66.2%</td>
<td>28.3%</td>
<td>37.9%</td>
</tr>
<tr>
<td>3</td>
<td>11</td>
<td>16.33</td>
<td>73.6%</td>
<td>29.8%</td>
<td>43.9%</td>
</tr>
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<td></td>
<td>12</td>
<td>15.53</td>
<td>72.7%</td>
<td>26.3%</td>
<td>46.4%</td>
</tr>
<tr>
<td></td>
<td>13</td>
<td>15.68</td>
<td>79.3%</td>
<td>30.3%</td>
<td>49.0%</td>
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<tr>
<td></td>
<td>14</td>
<td>15.46</td>
<td>73.2%</td>
<td>27.2%</td>
<td>46.0%</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>15.75</td>
<td>74.7%</td>
<td>28.4%</td>
<td>46.3%</td>
</tr>
<tr>
<td>4</td>
<td>15</td>
<td>16.27</td>
<td>75.8%</td>
<td>28.0%</td>
<td>47.8%</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>16.78</td>
<td>72.1%</td>
<td>28.7%</td>
<td>43.5%</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>16.53</td>
<td>74.0%</td>
<td>28.4%</td>
<td>45.7%</td>
</tr>
<tr>
<td>Fill</td>
<td>2</td>
<td>17</td>
<td>85.1%</td>
<td>28.7%</td>
<td>56.4%</td>
</tr>
</tbody>
</table>

Calculated friction angle and cohesion values were obtained by a best-fit linear regression through the data points for the last four trials of each normal stress step. The data points for the first trial of each normal stress step were disregarded because they were anomalously low compared to the other four trials. An explanation for these low values may be that in the first trial a shear plane has not been developed through the sample which results in excess pore pressures. In the last four trials, a shear plane has been created and pore pressures may be able to dissipate.
along that plane. The same process repeats for each normal load because reconsolidation of the sample results in the destruction of the existing shear plane developed in the previous step. See Figure 5-4 for an example.

![Plasticity Chart](image)

Figure 5-1. Badlands soils samples plotted on the plasticity chart.

Cohesion ranged from 27-48 kPa and the friction angle ranged from 18.5-31.3 degrees (Table 5-2). Non-zero cohesion values suggest that drained conditions were not met during testing. This apparent cohesion is a result of the development of pore pressures in the sample, something a truly drained test may eliminate. Stark and Eid (2005) suggest that the residual strength envelope of clays should pass through the origin, i.e. a cohesion value of 0, because when the residual condition is met, the orientation of the clay particles is such that it is difficult for particles to establish bonds between them resulting in the majority of strength coming from frictional resistance. However, the slowest movements measured within the complex likely
exceed the rate of shearing required for drained conditions to persist, and so it is expected that at least a small value of cohesion accurately models real conditions in the complex. Additionally, the failure envelopes could not be accurately characterized by forcing the cohesion intercept to pass through the origin. The use of a small value of residual cohesion has been presented in the literature (Lupini et al. 1981; Skempton 1985; Tiwari et al. 2005; Vithana et al. 2012). Furthermore, the contribution of cohesion to the overall stability is very small compared to friction angle values when considering the magnitude of normal stresses present on a slide plane that may be as deep as 45 m in one of the landslides mapped and likely at least that deep in the larger Cliff Shelf and Cedar Pass Landslide.

Figure 5-2. Grain-size distribution curves for the Badlands soil samples.
Figure 5-3. Shear stress versus horizontal displacement graph for Sample 11. The odd shape of the curves near the beginning is thought to be related to some mechanical issue with the direct shear device as it occurred in nearly every trial for every sample. In the legend, the first number refers to the trial number and the second value is the simulated depth (e.g. 1-40 meter means trial 1 with the sample consolidated under a load equal to 40 meters of overburden).

Table 5-2. Results of the direct shear test. Values were computed excluding the first trial from each normal stress step because for each step the first value was anomalously low compared to the values from the last four trials which generally agreed with each other. The average value reported is the average of samples 7-14. Sample 17 was excluded because it is not a natural soil.

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Cohesion (kPa)</th>
<th>Friction Angle (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>36.6</td>
<td>19.2</td>
</tr>
<tr>
<td>9</td>
<td>29.7</td>
<td>22.2</td>
</tr>
<tr>
<td>11</td>
<td>36.9</td>
<td>18.5</td>
</tr>
<tr>
<td>14</td>
<td>27.0</td>
<td>23.6</td>
</tr>
<tr>
<td>17</td>
<td>48.0</td>
<td>31.3</td>
</tr>
<tr>
<td>Average</td>
<td>32.6</td>
<td>20.9</td>
</tr>
</tbody>
</table>
Figure 5-4. Example of direct shear test results (Sample 11). The graph is scaled to a 1:1 X:Y ratio to show the true angle of the line. Note the relative consistency of the data points from the last four trials of each normal stress step (blue). The orange points are from the first trial. Cohesion and friction angle were calculated using a line fit through the last four points because of the relative agreement between points.

Overall, friction angle and cohesion values measured in this study are higher than expected based on ranges for these parameters provided by other studies and back-calculated values used for investigation in the park. This may be because a direct shear device, which shears a sample forward and backward, may not allow the sample to reach residual conditions. Specifically, Vithana et al. (2012) found that residual values of cohesion and friction angle obtained using a direct shear device compared to using a ring shear device were 2.1-3.2 and 1.6-1.9 times higher, respectively. If the adjustments reported by Vithana et al. (2012) are applied to the strength values of the Badlands samples, the resulting values are well within ranges reported by other investigators testing samples with similar materials (Table 5-3).
Table 5-3. Results of the direct shear test showing an adjustment for using a direct shear device instead of a ring shear device based on experimental data from Vithana et al. (2012). Average values disregard Sample 17 because it is embankment fill and therefore not representative of a majority of the soils in the complex.

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Effective Cohesion (kPa)</th>
<th>Effective Friction Angle (degrees)</th>
<th>Adjusted Effective Cohesion (kPa)</th>
<th>Adjusted Effective Friction Angle (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>36.6</td>
<td>19.2</td>
<td>11.4 – 17.4</td>
<td>10.1 – 12.0</td>
</tr>
<tr>
<td>9</td>
<td>29.7</td>
<td>22.2</td>
<td>9.3 – 14.1</td>
<td>11.7 – 13.9</td>
</tr>
<tr>
<td>11</td>
<td>36.9</td>
<td>18.5</td>
<td>11.5 – 17.6</td>
<td>9.7 – 11.6</td>
</tr>
<tr>
<td>14</td>
<td>27.0</td>
<td>23.6</td>
<td>8.4 – 12.9</td>
<td>12.4 – 14.8</td>
</tr>
<tr>
<td>17</td>
<td>48.0</td>
<td>31.3</td>
<td>15.0 – 22.9</td>
<td>16.5 – 19.6</td>
</tr>
<tr>
<td>Average</td>
<td>32.6</td>
<td>20.9</td>
<td>10.2 – 15.5</td>
<td>11.0 – 13.1</td>
</tr>
</tbody>
</table>

In addition to attempting to determine the residual friction angle directly through direct shear testing, we used an empirical relationship developed and revised by Stark and Eid (1994), Stark et al. (2005), and Stark and Hussain (2013) to estimate the drained residual friction angle from the liquid limit, clay fraction of the sample.

This study adopts Equation 2d (Equation 5-1) from Stark and Hussain (2013). The single input is the soil liquid limit, but several different equations exist for soils with varying clay size fractions, liquid limits and effective normal stresses. The use of equation 2d is appropriate for a soil with a clay fraction size between 25 and 45 percent, a liquid limit between 30 and 130%, and an applied normal stress of 700 kPa. Based on the results of the laboratory tests, clay size fractions averaged between 27 and 45 percent and liquid limit averaged from 66 to 85 percent. An equation considering an applied normal effective stress of 700 kPa was chosen for the calculation because 700 kPa is approximately the normal stress bracketed during the direct shear testing and therefore would provide the most applicable results.

\[
(\varphi'_r)_{\sigma'_n=700 \text{kPa}} = 28.05 - 0.2083(LL) - 8.183 \times 10^{-4}(LL)^2 + 9.372 \times 10^{-6}(LL)^2
\]  

(5-1)

where \( \varphi'_r \) = the drained effective residual friction angle

\( \sigma'_n \) = the effective normal stress for which the equation is applicable

\( LL \) = the liquid limit of the soil
Using Equation 5-1 provided a drained residual friction angle range of 10.2 to 13.4 degrees (Table 5-4). This range is similar to the range of 8.5-13 degrees estimated by Kumar & Associates (1999) using the same correlation; in this case, equations for the correlation were not available at the time and the graphical form of the correlation from Stark and Eid (1994) was used. Additionally, the values were estimated from one of the effective normal stress trend lines within the >50% clay fraction category. Laboratory testing results presented in Kumar & Associates (1999) only show a percent passing No. 200 sieve, so it is unclear whether or not the clay size fraction of soils samples was analyzed or considered when using the correlation. For this study, an emphasis was placed on the values estimated by the empirical relationship because of potential issues regarding the direct shear testing approach.

Table 5-4. Calculated drained residual friction angle values for different geologic units within the Cedar Pass Landslide Complex using equation 2d from an empirical correlation between drained residual friction angle and liquid limit developed by Stark and Hussain (2013).

<table>
<thead>
<tr>
<th>Unit</th>
<th>Average Liquid Limit</th>
<th>Average Clay Fraction</th>
<th>Drained Residual Friction Angle (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>76.1%</td>
<td>40.9%</td>
<td>11.6</td>
</tr>
<tr>
<td>2</td>
<td>66.2%</td>
<td>44.4%</td>
<td>13.4</td>
</tr>
<tr>
<td>3</td>
<td>74.7%</td>
<td>44.7%</td>
<td>11.8</td>
</tr>
<tr>
<td>4</td>
<td>74.0%</td>
<td>44.3%</td>
<td>12.0</td>
</tr>
<tr>
<td>Fill 2</td>
<td>85.1%</td>
<td>26.8%</td>
<td>10.2</td>
</tr>
</tbody>
</table>

5.2 Field reconnaissance observations and interpretations

The main portion of fieldwork was conducted from May-August 2018, but other site visits were made in December 2017 and April 2019.

5.2.1 Cliff Shelf Trail area

The Cliff Shelf area defined by this research encompasses the area to the east of the upper switchback of Badlands Loop Road and the Cliff Shelf parking lot (see Figure 3-2). Damage to park infrastructure includes settlement and cracking of the southeast side of the parking lot and adjacent sidewalk, and deformation of the wood and composite boardwalk at the beginning of the Cliff Shelf Trail.
A large scarp approximately 3 to 5 meters tall is located roughly 20-meters from the lower portion of the trail with several smaller (< 1-meter) scarps located directly below the boardwalk. The smaller scarps can be traced towards the parking lot where there is some minor offset, prevalent cracking of pavement, and separation of a wooden curb. This same damage was noticeably more significant in February 2020 (Figure 5-5). Cracks extend to the southwest side of the parking lot near a drop inlet for parking lot drainage. The wood curb behind the drop inlet is deflected at a downward angle toward the side of the parking lot experiencing settlement (see red arrow in Figure 5-5-B). Other landslide features observed in the area include multiple internal scarps, tension cracks, and generally hummocky topography (Figure 5-6; Figure 5-7).

Field observations indicate that movement of a larger landslide located south of the Cliff Shelf Trail, identified by the larger scarp, is destabilizing smaller blocks above (uphill) of the main landslide mass, and movement of the smaller blocks is causing the damage to the parking lot and trail (Figure 5-7). The toe of this landslide could not be identified in aerial photographs or in the field, so it was assumed to be located at the base of the slope, a common assumption made during landslide investigations (Schulz 2004). This assumption is largely based on the size of the landslide and the amount of vertical displacement shown by the height of the head scarp. This study has named the larger landslide the Prairie Island Landslide, as it has not been mapped and named in previous studies.

Movement of the Prairie Island Landslide appears to be mainly translational as indicated by many linear ridges and grabens near the top of the landslide, and the absence of a toe bulge, which would indicate potential rotation. These observations are consistent with an InSAR survey carried out in 1999-2002 by Anderson et al. (2004) that showed very few large areas of uplift consistent with the formation of toe bulges created by rotational landslides. However, it should be noted that due to the relatively slow movement of landslides within the complex (<0.5 meter/month), combined with relatively high erosion rates, geomorphic evidence of uplift may be significantly subdued or eliminated altogether.
Figure 5-5. A comparison of damage to the Cliff Shelf Trail parking lot. Both images are looking south. The red circle indicates the same joint in the wood curb in each image. In A, cracks are not as prevalent and have not been sealed. The clipboard is 36 cm on the long side. In B, there are more cracks (no sealant) and older cracks (sealant) have a larger aperture. Cracks extend across the parking lot to the drain on the opposite side (red arrow).

Figure 5-6. Panorama of the Prairie Island Landslide looking southwest. The Cliff Shelf parking lot is located in the upper right corner of the picture. Photo courtesy of M. Tello.
Figure 5-7. The boundaries of landslides in the vicinity of the Cliff Shelf parking lot identified by mapping landslide-induced geomorphic features. The direction of movement is indicated by the arrows.
5.2.2 Lateral Shear Landslide

The Lateral Shear Landslide is a region of instability encompassing approximately 3000 m² located approximately 75 meters south of the Cliff Shelf parking lot entrance.

The highway surface from between the parking lot entrance and a prominent nearby butte known as the Ridge of Resistance is experiencing several areas of pavement cracking and offset. Damage to the highway is recent, having occurred since the last road resurfacing, which was completed in late 2015 during the construction of a large earthen buttress downgrade of the Ridge of Resistance. Some crack systems extend for more than 5 meters and generally trend in a southwest to northeast direction closer to the parking lot, cutting diagonally across the highway to subparallel to the shoulders of the road along a short 20-meter stretch of highway above the area near the road sign indicating the entrance to the parking lot. Two obvious bumps have formed in the highway surface with one just east along the highway and one just west along the highway of the Ridge of Resistance (Figure 5-7). The offset in both bumps is such that the surface steps down as you travel east along the road. The western bump is also accompanied by a broken and uplifted segment of a concrete curb on the outside (southern) shoulder. The eastern bump does not appear to be affecting the concrete curbs on both shoulders of the road. However, a change in curvature of the outside curb is noticeable right at the location where the bump intersects the curb. It is unclear if the change in curvature is a result of horizontal movement of the highway structure or if the curb was originally constructed in this fashion as this location is coincident with the beginning of a switchback in the highway and the curb is not cracked or broken. As of February 2020, pavement cracking extending all the way across the highway was occurring in the locations of the road bumps, and the curb on the north side of the highway was broken and displaced in several places (Figure 5-8). Additionally, asphalt near the drop inlet and by the sign for the parking lot on the south side of the road was heaving and breaking apart. Other highway damage on the highway surface involved a concrete box drop inlet on the north side of the highway. The drop inlet appears to be separating from the curb as there is a several centimeter-wide crack between the box structure and the poured curb. During rain events, water and debris were observed flowing into this crack.
Figure 5-8. Looking south across the highway at the Ridge of Resistance in February 2020. The crack across the highway is potentially the western margin of the Lateral Shear Landslide. The road surface on the left side of the crack has been displaced downward relative to the road surface on the right side. Photo courtesy of A. Graber.

Directly adjacent to and north of the highway there are some minor scarp and tension crack features mainly in a location near the drop inlet. Other tension cracks exist further up the hill, closer to the parking lot entrance, on the inside of the highway curve. These cracks can be followed toward the highway where cracking is present in the same orientation on the pavement surface.

In the green space between the parking lot and the trailer/RV pullout on the highway for the parking lot, there are few noticeable landslide-induced features. The only exception is separation at the base of poured concrete stairs leading to the parking lot from the pullout. It appears that the poured curb at the base of the stairs has separated from the stair structure, as there is a >2 cm step down to the curb and a >2 cm gap between the bottom of the stairs and the curb. The gap is twice as wide as the expansion joint filler material installed during construction, but it is unclear if landslide movement caused the additional separation or offset to occur.
Field mapping could not definitively identify and confirm the boundary of the Lateral Shear Landslide as mapped by the National Park Service in 2016. An obvious head scarp is not easily identified and some scarp features around the road are not laterally continuous. There is significant offset and cracking in the highway surface at that location. By the end of the summer in 2018, one specific bump in the pavement had an offset of 0.3-meters or more, however, no offset was observed in the concrete curb on both shoulders of the highway adjacent to the bump. The NPS lowered the speed limit from 25 mph to 15 mph on the incline to Cedar Pass due to the severity of surface distress in this location and two more locations further up the highway. Some minor scarp-like features and open tension cracks are as close as 10-meters to the highway.

Active erosion is occurring in a channel to the south near the base of the highway embankment that could be causing the instability observed on the south side of the highway. A small amount of water was observed seeping into the channel from an area directly below and south of the highway a couple days after rainfall had occurred. This could be the result of groundwater seeping from underneath the highway or from the flow of water along piping features that may or may not be associated with buried culverts. National Park Service maps show at least two old culverts that daylight the embankment slope that were intentionally filled during the construction of a new storm-water capture and conveyance system installed in 2015.

An inclinometer installed on the south shoulder of the highway in the area showed movement at a depth of about 11 meters between October 2016 and July 2017 (Dominic Monarco, personal communication, July 2018). Based on this information, the current interpretation is that a landslide, moving towards the south, is responsible for the highway surface distress in this location and the landslide features located above and below the highway (Figure 5-7). With movement occurring at a depth of 11 meters below the south shoulder of the highway, the scarp located on the slope a few meters to the north of the highway has been identified as the head scarp.

### 5.2.3 Upper Wedge

The major feature of concern in this area is a zone of recurring offset in the highway surface (Figure 5-9). It appears that most, if not all deformation of the highway is occurring in the vertical direction with little horizontal offset observed. Directly to the west of the zone of offset is an area of scarps and tension cracking on a steep slope whose orientations match the
offset and cracking in the highway pavement (Figure 5-10). This slope has a maximum vertical rise of approximately 16 meters with an average slope angle of around 35 degrees and a maximum slope angle of 70 degrees. The uppermost scarp near the highway has a maximum offset of approximately 1.3 meters. The first signs of instability on this slope appeared sometime after 2003, as the scarps and tension cracks mapped in the field on the slope were not present in aerial photographs collected in 2003. An overview of Google Earth historical imagery shows the cracks developing in the slope as far back as 2011 with highway damage occurring in the same area at least before 2012. Photographs taken by park staff from July 23rd, 2012 show an area of modified pavement in the location of a recurring crack. The formation of the head scarp on the slope is most noticeable in images from 2016, which show a well-defined scarp near the top of the slope. The time period between 2013 and 2016 is coincident with the construction of a series of small retention ponds located just 50 m uphill and north along the highway on the east side of the road. There was also a period of normal to above normal precipitation between 2013 and 2015 that may have caused more movement on this slope as well as other areas in the complex, including the slope just downhill and west of the Cliff Shelf Parking Lot that was stabilized at the end of 2015. The Upper Wedge area has been subject to slope stability issues for the last 100 years. Wanless (1920) observed a landslide dammed pond occupying the area directly west of the present-day highway embankment. After rainstorms, the low gradient of the old pond bottom allows water to collect, and standing water was observed several times in this area, especially right at the base of the embankment fill.

The crack and settlement in the highway, which appears to be a continuation of the scarp feature on the west side of the road, does not continue onto the eastern shoulder of the road. There is a dip in the road on the southern side of the embankment fill, but no scarps or tension cracks were observed on either side of the highway in that area. No head scarp was observed; however, it is possible the scarp is buried under small talus fans beneath the cliffs of Millard Ridge that exhibit occasional rockfall, especially after/during rainfall events. Between April 2019 and February 2020, a larger volume rockfall than had previously been observed since December 2017 occurred. The source of the rockfall appears to be blocks of rock separating from Millard Ridge along subvertical joints. Additionally, water tends to pond in a catchment basin behind a concrete buttress on the east side of the road. This structure was built to collect surface water in a channel during the construction of the new storm water system in 2015. Infiltration of
the ponded water may increase groundwater levels in the embankment. Sometime between August 2018 and April 2019, the basin was lined to prevent infiltration.

Figure 5-9. Looking north up the highway in February 2020. The crack propagating across the highway is a recurring feature that the park has dealt with by adding or removing asphalt. The location of the crack is coincident with the approximate point at which the highway transitions on to a more substantial embankment fill. Scarps and tension cracks on the slope across the highway are visible. These features match the crack in the highway. Rockfall occasionally occurs in this area, especially after heavy rainfall. Some fallen rocks are visible on the shoulder in the right of the image. The covered orange pipe is an inclinometer that was installed in 2016 by Yeh & Associates and monitored by the FHWA. The top of the pipe was flush with the ground surface in April 2019. It is unclear whether the pipe has pushed up out of the ground or whether the FHWA extended it.

After rainfall events, a small amount of water was observed exiting an abandoned 61 cm (24 in) reinforced concrete culvert which used to convey water from the east (upstream) side of the embankment, under the embankment to the area located to the west. The inlet for this culvert is buried, suggesting water is either infiltrating from the ground surface to the culvert inlet.
The landslide features adjacent to the highway are thought to be related to small-scale instability in the steep slope that borders the north side of the ravine that the highway crosses in that area. No obvious landslide features were observed on the south side of the area that might suggest a landslide boundary such as the one mapped by the NPS in 2016.

Figure 5-10. The north slope of the Upper Wedge area. The slope in the foreground is moving to the right (south) and was mapped as the Upper Wedge Landslide in this study. The truck is passing across the highway embankment that is experiencing some deformation. Millard Ridge makes up the cliffs in the background.
The current interpretation of mapped features and other observations is that the recurring offset in the highway is caused by settlement in the embankment fill on which the highway is constructed, possibly related to movement in the natural slope adjacent to the highway (Figure 5-11). The topography under the highway alignment before construction is unclear, but it appears the highway was built across an old drainage channel. One specific field observation that supports the settlement theory is that the highway surface drops as it transitions onto fill from the north and then has some slight undulations before it transitions off fill to the south. Settlement of fill has been an issue in other parts of the park including the incline to Norbeck Pass and Dillon Pass to the west and have been attributed to consistent saturation of clay embankment soils and steep embankment slopes (FHWA 1999), unstable, fat clays (FHWA 2011), and piping of fines and differential frost heave (Parsons Brinckerhoff Quade & Douglas 2004). Therefore, the boundaries of the Upper Wedge Landslide as mapped by the Park Service have been revised to only include the slope shown in Figure 5-10. This smaller landslide is likely the result of the steepness of the slope and soils, but may also be related to large scale slumping of the Cliff Shelf paleolandslide, as the slope is located near the head scarp of the larger landslide.

5.2.4 Lower Wedge

A definitive boundary of the Lower Wedge Landslide was not observed during mapping. Landslide-induced geomorphic features are generally only observable on the north side of the slide area which include some minor scarps and tension cracks. Additionally, there is no obvious head scarp near the highway. The highway is experiencing significant surface distress at that location with a short section of multiple bumps, dips and cracks in the pavement. Deformation in the highway is not always laterally continuous and all features do not extend onto either shoulder. If a landslide does exist in that area, it is likely moving downslope to the west but it is still unclear based on only information collected in the field where the boundaries are located and what exactly is causing the deformation of the highway.
Figure 5-11. The boundaries of landslides in the Wedge area identified by mapping landslide-induced geomorphic features. The direction of movement is indicated by the arrows.
5.3 Numerical modeling

5.3.1 Prairie Island Landslide

5.3.1.1 Back analysis of model parameters

The Prairie Island Landslide was modeled as a homogeneous geologic unit, and so a single value for cohesion and friction angle was input into the model. A single unit was used because of the lack of borehole data for the landslide and because consistent stratigraphy within the landslide could not be identified.

Reasonable initial ranges of model parameters were developed using three possible sources, including lab testing completed for this study, testing/monitoring carried out by others at the site, and literature values (Table 5-5). These ranges were narrowed down using a back analysis as described below. A range of values was not given to unit weight, as values from all three sources were generally consistent. Average cohesion and friction angle values obtained from lab testing in this study were generally higher than ranges found within the literature, but were included as the maximum values for those parameters.

Table 5-5. Initial ranges of model parameters for the Prairie Island Landslide.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight</td>
<td>18.0 kN/m³</td>
<td>Kumar &amp; Associates 1999; FHWA 2013, Zhang 2013; Lab testing for this study</td>
</tr>
<tr>
<td>Res. Cohesion</td>
<td>0-36.2 kPa</td>
<td>Baum et al. 1998; Stark et al. 2005; Stark and Eid 2013; Lab testing for this study</td>
</tr>
<tr>
<td>Res. Friction Angle</td>
<td>5.0-20.9 degrees</td>
<td>Kumar &amp; Associates 1999; Wan &amp; Kwong 2002; Dewoolkar &amp; Huzjak 2005; Tiwari et al. 2005; Lab testing for this study</td>
</tr>
<tr>
<td>Water Table Depth</td>
<td>5-10 meters</td>
<td>Kumar &amp; Associates 1999, FHWA 2012</td>
</tr>
</tbody>
</table>

The initial water table elevation was chosen based on piezometric data provided in past geotechnical studies. A relatively shallow depth was chosen based on the hypothesis that higher ground water levels result in instability of the slopes within the landslide complex.

Detailed slide plane geometry is unknown for the Prairie Island Landslide, but previous observations (Kumar & Associates 1999; Anderson et al. 2004) have been made that landslides within the complex are translational. Additionally, Kumar & Associates (1999) showed through
borehole and inclinometer data that the Cedar Pass Landslide located on the north end of the complex has a steep initial slide surface (approximately 60 degrees) transitioning to a slide surface with a dip of around 2 degrees. Based on this information, the landslide models in this study relied on slide planes of a similar shape (see Figure 5-12).

An initial slope model along three profiles (Figure 5-13, 5-14, 5-15 see pages 57-59) through the Prairie Island Landslide using the maximum parameter values listed in Table 5-5 resulted in a FS over 2.5. Even with the water table at the surface (i.e. with ponding), the FS remained greater than 2. A reasonably consistent slide plane was identified among the three models despite the high FS values. This slide plane geometry ultimately remained relatively consistent even as other parameters were changed. Strength parameters and the water table elevation were adjusted iteratively until final “balanced” values were reached. Values were considered balanced when the average FS between the three profiles was approximately 1.0. Differences in the FS values among the three profiles were attributed to the variable topography of each profile.

Final calculated material parameters and water table depth are presented in Table 5-6. A small effective cohesion value of 5 kPa and an effective friction angle of 10.5 degrees were used in the remaining models. The value of the friction angle is close to the range of adjusted laboratory values, within the range of residual friction angles calculated for similar materials by other researchers (Baum et al. 1998, Kumar & Associates 1999; Dewoolkar & Huzjak 2005; Tiwari et al. 2005), and within the range of values predicted for these materials by the empirical correlation between liquid limit and drained residual friction angle developed by Stark and Hussain (2013). A residual friction angle of 10.5 degrees is slightly higher than the value of 8.5 degrees which was back calculated by Kumar & Associates (1999) for the Cedar Pass Landslide. A small value of cohesion was used instead of a value of 0 in recognition that these materials are expected to display a low level of cohesion, even under semi-drained conditions and high normal loads. Strength testing by Baum et al. (1999) for several large, slow moving landslides in clay-rich materials, showed that the residual drained effective cohesion along the slide plane ranged from 1.9-6.7 kPa and the residual drained friction angle ranged from 6-13.5 degrees. The cohesion and friction angle values used in this study are within those ranges.
Figure 5-12. Figure 4-2 from Kumar & Associates (1999) which shows the hypothesized shear surface of the Cedar Pass Landslide based on borehole and inclinometer data. The shape of this translational landslide was used to approximate the shear surface of the other landslides exhibiting similar behavior in other parts of the landslide complex.

Table 5-6. Final back calculated slope stability model parameters constrained based on three profiles through the Prairie Island Landslide.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight (kN/m$^3$)</td>
<td>18.0</td>
</tr>
<tr>
<td>Residual Cohesion (kPa)</td>
<td>5.0</td>
</tr>
<tr>
<td>Residual Friction Angle (degrees)</td>
<td>10.5</td>
</tr>
<tr>
<td>Depth to Water Table (m)</td>
<td>5.0</td>
</tr>
</tbody>
</table>

The groundwater depth was adjusted to approximately 5 m below the ground surface. This value approximately matches the highest ground water level observed in the complex (Kumar & Associates, 1999), and varies slightly within the profiles due to variations in the topography. Using a high groundwater level is appropriate for these landslides given the observed connection between landslide movement and higher than average precipitation which results in an increase in the groundwater elevation (FHWA 2002; FHWA 2012; FHWA 2013).
5.3.1.2 Stability

Figures 5-13, 5-14, and 5-15 show the three adjusted slope models for the Prairie Island Landslide, using the cross-section locations shown on Figure 4-2. The upper landslide boundary was tightly constrained by a set of limits based on the location of the head scarp, compared to the lower boundary of the landslide, which was allowed more flexibility because it was unclear in the field where the toe was located. Models were encouraged to produce basal shear surfaces with a dip of only a few degrees by using a block search. Some models were allowed to produce circular surfaces and generally showed deep slide planes with depths often greater than 75 m. Such models are unrealistic considering the observed translational nature of these slides. Additionally, shallower, large diameter, circular slip surfaces were modeled in an attempt to approximate a stepped failure plane, but these solutions consistently showed higher FS values than the deeper translational surface solutions.

A variety of shear plane exit points were modeled, including exit points located at or near the bottom of the slope and points near the midpoint of the slope to simulate a smaller landslide. Solutions with slide planes extending to the slope toe consistently produced lower FS values than those solutions with shallower slide planes exiting further up the slope. These deeper solutions are also supported by observations in the field that include disrupted topography all the way to the slope toe.

Comparison of the three profiles shows that middle profile (Profile E) has the highest FS and that right-side (west) profile (Profile F) has the lowest FS. The relative instability on the right side of the landslide may be related to the topography on that side. The distance between the toe of the slope and the head scarp is up to 75 m smaller than the transects through the middle and left side of the landslide. The shorter distance may be related to the fact there is a major drainage channel that cuts through the base of the slope in this location.

5.3.1.3 Sensitivity

The sensitivity of the Prairie Island Landslide to changes in material properties (namely unit weight, cohesion and friction angle), water table fluctuation, and erosion of the toe were investigated.
Figure 5-13. Slope model for the Prairie Island Landslide Profile D. A block search was used to help constrain the shape of the slide plane. The horizontal and vertical scales are in meters.
Figure 5-14. Slope model for the Prairie Island Landslide Profile E. A block search was used to help constrain the shape of the slide plane. The horizontal and vertical scales are in meters.
Figure 5-15. Slope model for the Prairie Island Landslide Profile F. A block search was used to help constrain the shape of the slide plane. The horizontal and vertical scales are in meters.
The results of the sensitivity analysis on material properties are presented in Figure 5-16. Similar trends are noted on each profile. Unit weight was varied between 13.5 and 22.5 kN/m$^3$ (-25 to +25 percent of the base value). The FS increased as much as 9% at the maximum value and decreased as much as 16% for the minimum value. Cohesion and friction angle were varied between zero and double the base value. This corresponds to a cohesion value ranging between 0 and 10 kPa and a friction angle ranging between 0 and 21 degrees. The FS was significantly impacted by changing the friction angle (up to a 98% change), compared to cohesion which produced a maximum of 12% change in the FS. The small effect of cohesion may be related to the small values of cohesion used, and the depth of the slide plane. Because of the high normal loads experienced along deeper slide planes, the friction angle is expected to have a larger impact on the stability.

To test the sensitivity to fluctuations in groundwater, the water table was lowered in 1 m increments from the ground surface to a depth of 25 meters. With the water table at the surface, the factor of safety decreased by 11% to 20%. When the water table was lowered to a depth of 25 m, the factor of safety increased by 38% to 47%. A water table depth of 25 m is close to the depth of the deepest borehole advanced within the landslide complex that did not encounter groundwater while drilling. Looking at the sensitivity of all three profiles and only considering the influence of groundwater, parts of the landslide become unstable; in other words, when the factor of safety drops below 1.0, when the water table is as shallow as 2 and 9 meters below the ground surface. Figure 5-17 presents the results of the water table sensitivity analysis for the Prairie Island Landslide.

The sensitivity of the Prairie Island Landslide to erosion of the toe of the landslide was investigated by lowering topography devoid of vegetation in the bottom third of the landslide by increments of 2.54 cm (1 in) to simulate average erosion over one year (Stoffer 2003; NPS 2020). A maximum of 61 cm (24 in) of erosion was modeled. Figure 5-18 shows the results of the erosion sensitivity analysis. All three profiles showed very little change in stability with an increasing amount of erosion. The maximum change in FS was only a little over 1%.
Figure 5-16. Sensitivity of the Prairie Island Landslide to changes in material properties including unit weight, cohesion and friction angle along Profile D, Profile E, and Profile F. The more horizontal the line is, the less sensitive the model is to that parameter. All three profiles show that the Prairie Island Landslide is most sensitive to changes in friction angle.
Figure 5-17. Sensitivity of the Prairie Island Landslide to changes in the ground water level. Each profile has relatively the same sensitivity to the water table, indicated by the similar shape of each line. Differences in the positions of the lines is a result of each profile having a slightly different base factor of safety. Red shading indicates factor of safety values <=1.

Figure 5-18. Sensitivity of the Prairie Island Landslide to erosion at the toe. Each profile shows minimal sensitivity to erosion. Differences in the positions of the lines is a result of each profile having a slightly different base factor of safety. Red shading indicates factor of safety values <=1.
5.3.1.4 Mitigation

Mitigation options for the Prairie Island Landslide are likely limited due to the size of the landslide. An earthen buttress placed at the toe would need to be twice the size of the buttress constructed just to the west in 2015 and up to 50% larger than the buttress constructed south of Cedar Pass in 2000. Buttress construction would also have to take into account the significant drainages located along the landslide flanks that include channels as deep as 10 meters in some locations. The drainage along the east flank of the landslide drains the entire area encompassed by the Cliff Shelf Trail.

For this study, investigations of corrective measures were focused on stabilizing individual unstable blocks located above the head scarp. Movement of these smaller blocks, likely caused by downhill movement of the Prairie Island Landslide, is responsible for the observed damage in Cliff Shelf parking lot and along early portions of the trail. The individual blocks were modeled as shallow rotational failures to simulate retrogression of the head scarp of the Prairie Island Landslide. Profile D for Prairie Island was used to create the model and evaluate mitigation options as no profile in the correct orientation passing through the parking lot was collected. Material property statistics assigned to the clay are shown in Table 5-7. The range of friction angle values was limited to the range of values used for these materials by other investigators in the park and the values obtained from the back analysis in this study.

Two mitigation methods were evaluated, including grouted tieback anchors and piles. These options were chosen because they increase the resisting forces along the slide plane of the upper blocks and do not require elements to be placed on the Prairie Island Landslide. For instance, the construction of a retaining wall below the parking lot would likely mean the foundation of that wall would be on the surface of the larger landslide, and the retaining wall could be damaged by movement of the larger landslide. The effectiveness of these mitigative structures on increasing stability was evaluated by the percent change in the probability of failure from FS values calculated using the Simplified Bishop method (Table 5-8). This method was chosen because of the hypothesized circular shape of the failure planes based on observations made in the field.

The base case produced a probability of failure of only 28% (Figure 5-19). This value is relatively low because the stability of this block is tied to the movement and stability of the Prairie Island Landslide located on the slope below. The probability of failure is expected to
increase if movement of the Prairie Island Landslide increases or if water table rises to or above the slide plane of the block.

Table 5-7. Statistical properties of the soils in the Lateral Shear and Prairie Island Landslide. Testing has not been carried out on shear plane materials, so unit weight was kept consistent with the other units. The mean cohesion and friction angle value are the back calculated values from this study. The minimum and maximum value of cohesion are the theoretical residual value and double the mean, respectively. The minimum and maximum friction angle is 0 to represent undrained conditions and about the maximum value used by other investigators in the park ([1], [2]), respectively.

<table>
<thead>
<tr>
<th>Material</th>
<th>Property</th>
<th>Unit</th>
<th>Distribution</th>
<th>Mean</th>
<th>Min</th>
<th>Max</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>Unit Weight</td>
<td>kN/m³</td>
<td></td>
<td>18</td>
<td></td>
<td></td>
<td>[1], [3]</td>
</tr>
<tr>
<td>Clay</td>
<td>Cohesion</td>
<td>kPa</td>
<td>Uniform</td>
<td>5</td>
<td>0</td>
<td>15</td>
<td>[1], [3]</td>
</tr>
<tr>
<td>Clay</td>
<td>Friction Angle</td>
<td>degrees</td>
<td>Uniform</td>
<td>10.5</td>
<td>8.5</td>
<td>27</td>
<td>[1], [2], [3]</td>
</tr>
<tr>
<td>Claystone</td>
<td>Unit Weight</td>
<td>kN/m³</td>
<td></td>
<td>18</td>
<td></td>
<td></td>
<td>[1], [2], [3]</td>
</tr>
<tr>
<td>Claystone</td>
<td>Cohesion</td>
<td>kPa</td>
<td>Uniform</td>
<td>5</td>
<td>0</td>
<td>20</td>
<td>[1], [3]</td>
</tr>
<tr>
<td>Claystone</td>
<td>Friction Angle</td>
<td>degrees</td>
<td>Uniform</td>
<td>10.5</td>
<td>8.5</td>
<td>27</td>
<td>[1], [2], [3]</td>
</tr>
<tr>
<td>Shear Plane</td>
<td>Unit Weight</td>
<td>kN/m³</td>
<td></td>
<td>18</td>
<td></td>
<td></td>
<td>[1]</td>
</tr>
<tr>
<td>Shear Plane</td>
<td>Cohesion</td>
<td>kPa</td>
<td>Uniform</td>
<td>5</td>
<td>0</td>
<td>10</td>
<td>[1]</td>
</tr>
<tr>
<td>Shear Plane</td>
<td>Friction Angle</td>
<td>degrees</td>
<td>Uniform</td>
<td>10.5</td>
<td>0</td>
<td>13</td>
<td>[1], [2]</td>
</tr>
</tbody>
</table>


A model was run with one row of anchors and the probability of failure dropped to 2.5%. Two rows of tieback anchors provided sufficient support to decrease the probability of failure to ~0% (Figure 5-20). This shows that the use of tiebacks may be effective in stabilizing the head scarp of the Prairie Island Landslide and the unstable blocks above.

The installation of reinforced piles along the edge downhill edge of the parking lot may help stabilize the area or the parking lot experiencing settlement due to movement of the Prairie Island Landslide if the piles are able to penetrate through the slide plane of the upper block and into underlying stable units. One row of piles lowered the probability of failure to 0.3%, suggesting that piles may also provide necessary support to stabilize the upper blocks of the Prairie Island Landslide (Figure 5-21). Additionally, piles can be installed with little disturbance of the natural landscape compared to other remedial measures.
Figure 5-19. Probabilistic analysis of a block located at the top of the Prairie Island Landslide. The model was created using a portion of Profile D from the Prairie Island Landslide. The horizontal and vertical scales are in meters.

Table 5-8. Summary of results of the effectiveness of different mitigation/support methods. A decrease in the probability of failure by 100% results in a probability of failure of ~0%. The initial probability of failure of the slope with no support was 30.3%.

<table>
<thead>
<tr>
<th>Support Type</th>
<th>Change in Probability of Failure (pf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tieback Anchors</td>
<td>Decreased by 100%</td>
</tr>
<tr>
<td>Piles</td>
<td>Decreased by ~100%</td>
</tr>
</tbody>
</table>
Figure 5-20. Slope stability model using grouted tiebacks to provide support to the slope. The Bishop Method was used to calculate the factor of safety. The horizontal and vertical scales are in meters.

Figure 5-21. Slope stability model using piles to provide support for the slope. The Bishop Method was used to calculate the factor of safety. The horizontal and vertical scales are in meters.
5.3.2 Lateral Shear Landslide

The Lateral Shear Landslide is located west and downhill of the Cliff Shelf parking lot (see Figure 5-7). A field investigation had difficulty identifying distinct boundaries of this landslide. Initially, it was unclear whether a landslide or some other process was causing highway surface distress in this area. However, an inclinometer installed by Yeh & Associates and FHWA in 2016 showed movement at approximately 11 m below the shoulder of the highway (Dominic Monarco, personal communication, July 2018). This shear surface was not sampled or noted in the borehole drilled before placement of the inclinometer. Figure 5-22 shows the slope stability model for the Lateral Shear Landslide.

The stratigraphy of the landslide was provided by a borehole log (Yeh & Associates 2016, Boring No. SI-103), which was incorporated in the slope stability model. The same soil properties back calculated in the analysis of the Prairie Island Landslide were assigned to every stratigraphic layer in the Lateral Shear model because the soils present in both landslides are very similar.

Figure 5-22. Slope stability model for the Lateral Shear Landslide. The shape of the water table was based off of piezometer data and observations made in the field. The vertical line passing through the landslide is the approximate location and depth of the inclinometer. The horizontal and vertical scales are in meters.
The water table was adjusted so that it was approximately 5 m below the highway. This value is higher than the highest water levels measured in the immediate area, but those measurements were collected during and at the end of a drier than normal 2-year period, suggesting that groundwater levels may have been lower than normal. The water table was moved to the surface or near the surface in areas where seepage was observed exiting the slope up to several days after rainfall events.

A shear plane with the same strength parameters as the surrounding material was added at the depth indicated by the inclinometer. With the other model parameters better constrained, the dip of the slide plane was adjusted until a FS of 1.0 was reached. The inserted shear plane was given a dip of about 4 degrees toward the slope below the highway, which matches the general geometry of other slide planes in the complex. An attempt was made to fit a circular failure surface, but doing so using the back calculated strength values and water table location produced unreasonably low FS values.

5.3.2.1 Stability

The lowest FS produced for the Lateral Shear slope stability model (Figure 5-22), with the parameters described in section 5.3.2, was 0.997 using the GLE/Morgenstern-Price method.

5.3.2.2 Sensitivity

Much like the Prairie Island Landslide, the Lateral Shear Landslide shows very little sensitivity to the unit weight of the soil. With unit weights ranging from 13.5 – 22.5 kN/m³, the total change in the factor of safety across that range was only 5%. The factor of safety decreased by 25% when the cohesion was lowered by 5 kPa to zero, and increased by 25% when the cohesion was increased by 5 kPa to 10 kPa. The maximum change in the factor of safety occurred when the friction angle was doubled to 21 degrees. This increased the factor of safety by 80%. When the friction angle was decreased to a value of zero, the factor of safety decreased by 75%. These results show that this landslide is more sensitive to changes in cohesion and slightly less sensitive to changes in friction angle than the Prairie Island Landslide. This is attributed to the fact that normal stresses within this landslide are less than in the Prairie Island Landslide because of the shallower slide plane. Figure 5-23 presents the results of the material properties sensitivity analysis.
Figure 5-23. Results of the material properties sensitivity analysis for the Lateral Shear Landslide. The more horizontal the line is, the less sensitive the model is to that parameter. The Lateral Shear Landslide is most sensitive to the friction angle of the soil but does show some sensitivity to the cohesion.

This landslide is reasonably sensitive to changes in groundwater, especially when the water table is between 0 and about 10 meters below the ground surface (Figure 5-24). From the baseline water table depth of 4.5 m, the factor of safety decreases by 18% when the water level is raised to the surface. When the water table is lowered by 5 meters, the factor of safety increases by about 34%. When the water table is greater than about 10 meters below the surface, the landslide exhibits little sensitivity to its depth. This is because at a depth greater than 10 meters, the water table is largely below the slide plane of the landslide.

This landslide shows limited sensitivity to erosion of the toe with erosion amounts ranging from about 1 cm to 61 cm (Figure 5-25)(<1% change). Historical images on Google Earth from September 25th, 2011 to June 15th, 2016 show up to about 5 meters of erosion into the channel on the right side of the landslide. When the model topography was adjusted to simulate this amount of erosion, the factor of safety decreased a little over 3%. One reason this landslide may not be very sensitive to erosion is that when material is lost, the volume of landslide material also decreases which acts to lower the driving forces. However, the slight increase in sensitivity with the removal of a larger amount of material in the channel shows that there is at
least the potential for the landslide to become more unstable if erosion of the slope below the highway continues.

Figure 5-24. Sensitivity of the Lateral Shear Landslide to fluctuations in the water table. The curve of the line flattens out when the water table is below the slide plane and steepens when the water table rises above the slide plane. Red shading indicates factor of safety values <=1.

Figure 5-25. Sensitivity of the Lateral Shear Landslide to erosion in the lower portions of the slope. The flatness of the line indicates very little sensitivity. Red shading indicates factor of safety values <=1.
5.3.2.3 Mitigation

Four different support methods were analyzed for the Lateral Shear Landslide, including an earthen buttress, a retaining wall, grouted tieback anchors and piles (see Table 5-9 for a summary of the results). The material properties, including cohesion and friction angle of each unit in each model (excluding buttress properties), were assigned input distributions in order to capture a range of potential material behaviors (Table 5-8). The base model produced a probability of failure of 68% (Figure 5-26).

The first mitigation measure analyzed was an earthen buttress. Results of modeling suggest that a buttress below the highway may be effective in stabilizing the portion of the highway in this area (Figure 5-27). It should be noted that when the model was allowed more flexibility in the location of the global minimum, the global minimum shifted below the buttress, exiting further down the slope. This may suggest that the addition of an extra mass of material...
near the top of the slope could cause new instability either below the buttress or along a deeper surface.

The effectiveness of a retaining wall at the base of the slope was investigated by adding a 3-meter-wide mechanically stabilized earth (MSE) wall. The wall material properties were assigned the same value as those from a deep patch modeled in FHWA (2013). A geotextile was placed in 0.3 m (1 ft) increments (see Figure 5-28 for model geometry and properties). The wall had a simplified design and was not anchored, however the retaining wall lowered the probability of failure to ~0%. Grouted tiebacks were added in a pattern near the base of the landslide to evaluate the effectiveness of this type of support. Five rows of tiebacks were added, which only decreased the probability of failure by approximately 50% (see Figure 5-29 for support properties). Lastly, two rows of piles spaced 2 meters apart were added on the downhill shoulder of the road to simulate support of the edge of the highway and add shear resistance by placing the piles through the shear plane of the landslide (see Figure 5-30). The probability of failure decreased only 10%. Analysis of piles and tiebacks suggests they may not be as effective mitigation strategies as a retaining wall or buttress (assuming the latter two structures can be founded on stable ground).

Table 5-9. Summary of results of the effectiveness of different mitigation/support methods. A decrease in the probability of failure by 100% results in a probability of failure of ~0%. The initial probability of failure of the slope with no support was 61%.

<table>
<thead>
<tr>
<th>Support Type</th>
<th>Change in Probability of Failure (pf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earthen Buttress</td>
<td>Decreased by 100%</td>
</tr>
<tr>
<td>Retaining Wall</td>
<td>Decreased by 100%</td>
</tr>
<tr>
<td>Tieback Anchors</td>
<td>Decreased by 48%</td>
</tr>
<tr>
<td>Piles</td>
<td>Decreased by 10%</td>
</tr>
</tbody>
</table>
Figure 5-27. Slope stability model of the Lateral Shear Landslide using an earthen buttress to provide support for the slope. The horizontal and vertical scales are in meters.

Figure 5-28. Slope stability model of the Lateral Shear Landslide using a retaining wall to provide support for the slope. The horizontal and vertical scales are in meters.
Figure 5-29. Slope stability model of the Lateral Shear Landslide using grouted tieback anchors to provide support for the slope. The horizontal and vertical scales are in meters.

Figure 5-30. Slope stability model of the Lateral Shear Landslide using piles to provide support for the slope and highway. The horizontal and vertical scales are in meters.
5.3.3 Upper Wedge Landslide

The Upper Wedge Landslide is a landslide mapped by the National Park Service and located on the north side of a butte known as the Matterhorn (see Figure 5-11). Field observations made for this study do not indicate a landslide with the boundaries mapped by the National Park Service. Nevertheless, the feature will still be evaluated to gauge the current and potential future stability. A topographic profile was collected through the landslide parallel to the direction of movement hypothesized by the Park Service. Stratigraphy was added based on borehole data from Yeh & Associates (2016), and material properties assigned to the clay and claystone were values back calculated from the Prairie Island Landslide. Material properties for the embankment fill match those used by Kumar & Associates (1999). While a sample of fill from this location was collected and its strength was tested in the lab, these values were discarded because of problems with the strength tests addressed in Section 5.1. The water table was adjusted so that it passed a point approximately 6 meters (21 ft) below the right shoulder of the highway. This approximately matches the shallowest level of water encountered during drilling (Yeh & Associates 2016).

5.3.3.1 Stability

A variety of methods were employed to identify a reasonable slide plane within the Upper Wedge model (Figure 5-31). Both shallow and deeper circular and non-circular slide planes were investigated. Shallower landslides are less likely, as a FHWA inclinometer installed on the east shoulder of the road showed no sliding movement to a depth 14.6 m (48 ft) between October 2016 and July 2017 (Dominic Monarco, personal communication, July 2018). Circular and non-circular slide planes deeper than the inclinometer are possible, however fitting a large radius circular plane underneath the depth of inclinometer with the head located at the base of Millard Ridge and a toe in the channel to the west resulted in a FS around 1.2, even with a relatively high water table and low material strength properties. Additionally, one would expect to see some evidence of uplift or a toe bulge in or near the channel due to the substantial rotational component a landslide of this shape would likely have. Another potential solution could be a landslide with a steeply dipping slide plane near the head and a low angle basal shear plane, similar to the geometry of the other landslides in the complex. This solution produced an
FS of around 1.4. In summary, modeling (and field mapping) could not confirm the presence of a landslide in the orientation originally mapped by the National Park Service.

Figure 5-31. Slope model used for the Upper Wedge Landslide mapped by the Park Service. The horizontal and vertical scales are in meters.

A model using a transect through the slope on the northern side of the drainage in the Upper Wedge area (see Figure 4-2, Profile A2-A2’ for model orientation), which is the landslide mapped as the Upper Wedge in this study, indicated a FS under 1.0 assuming a circular failure extending from the head scarp identified by mapping to the slope toe (Figure 5-32). This model was constructed to demonstrate that a landslide in this orientation, and separate from the landslide hypothesized by the Park Service, exists. The slide plane is likely circular due to the small size of the landslide combined with the amount of vertical offset observed along the head scarp. Strength values were left equal to the residual values back-calculated from the Prairie Island Landslide. To produce a landslide with a FS around 1.0, the cohesion and the friction angle had to be increased to within the lower bounds of typical peak strength values for these soils supported by Geotechnical Data Information (2013a; 2013b).

The results of the models from the Upper Wedge area support the conclusion that landslide-induced geomorphic features identified on the slope adjacent to the highway are more likely related to instability of that steep slope and movement occurring parallel to the highway and not to larger scale movement occurring in a direction perpendicular to the highway. The
smaller landslide is more recent, and the strength of the clay appears to be closer to peak values than residual values as indicated by slope modeling.

Figure 5-32. Slope model used for the Upper Wedge Landslide as mapped in this study. The horizontal and vertical scales are in meters.

5.3.4 Lower Wedge Landslide

5.3.4.1 Stability

The upper and lower boundaries of this landslide could not be definitively identified in the field, so the upper and lower limits were given large ranges. Inclinometer data from October 2016 to July 2017 showed no movement to a maximum depth of 22.5 m (74 ft) (Dominic, Monarco, personal communication, July 2018). A non-circular block search and a circular grid search were used in order to identify a slide plane located at a depth greater than 22.5 m below the highway and in the claystone unit identified in borehole data (Yeh & Associates 2016). See Figure 5-33 for the general Lower Wedge model profile.

A variety of different models were tested, including deep and shallow circular slide planes and deep and shallow non-circular slide planes. Using the back-calculated strength values for the clay and claystone, and strength values for the fill calculated by Kumar & Associates...
(1999), the lowest FS for the global minimum for most slide planes was generally greater than 1.2 to 1.3. Circular slide planes closer to the ground surface showed higher FS values than the deeper solutions. Several circular solutions with a FS of around 1.0 were found, but these surfaces were deemed as unrealistic given their pronounced, deep circular shape. For instance, the global minimum surface at its deepest point in these models was 24 m below the lower limit suggesting a significant rotational component not supported by field observations. A small circular failure in the downhill shoulder of the highway was modeled, but the lowest FS with those solutions was 1.3-1.4.

Several non-circular methods were employed and produced several more reasonable results with shallower slide planes. The solutions, however, still showed significant toe thrusts and did not pass below the inclinometer. Slide planes with a similar shape were drawn into the model to evaluate the stability of those surfaces, but resulting FS values were generally >1.2.

The summary of the modeling results is that a reasonable slide surface could not be identified using a variety of different computational methods and considering a variety of plausible slide plane shapes and locations. These results match observations in the field and the general conclusion that a localized landslide does not exist in this area.

Figure 5-33. Slope model used for the Lower Wedge Landslide as mapped by the Park Service. The horizontal and vertical scales are in meters.
5.3.5 Mitigation in the Upper and Lower Wedge areas

Specific mitigation options were not evaluated at the Upper and Lower Wedge areas because the unique landslides identified by the Park Service were not identified in the field and were not reproducible via slope modeling. Embankment fill deformation across the Upper Wedge area may be related to settlement or erosion of the fill, or movement of the Cliff Shelf Landslide. If damage is being caused by continuous movement of the Cliff Shelf Landslide, the best mitigative measure may be reconstruction of the highway in the area with a geotextile reinforced fill or a flexible pavement. The goal of these measures would be to reduce the frequency of required maintenance, as stabilization of the Cliff Shelf Landslide may not be feasible due to its large size. This same suggestion applies to the Lower Wedge area as well, where surface damage could not be attributed to a smaller, unique landslide in the area. If embankment issues are being created by small scale slope instability, erosion, or settlement, installing drainage in the embankment may help prevent the water table from rising after rain events and decrease the overall water content of the clay fill. The issue of extended residence time of ponded water in unlined basins adjacent to and above the highway fill, which may have contributed to damage in the area, was addressed by the Park Service by April 2019.
CHAPTER 6
DISCUSSION

6.1 Confidence of landslide identification in the field

This thesis has applied a confidence rating system in order to provide a quantitative way of expressing the likelihood that a landslide exists in the area, as opposed to some other explanation of the observed features. The confidence rating system used in this study was developed by the Oregon Department of Geology and Mineral Industries (Burns and Madin 2009). While it was designed for LiDAR-based mapping projects, it was adopted for this study based on the results of field mapping, as certain landslide features and boundaries were not always well-defined in some areas and were very identifiable in others. Each landslide is classified into a confidence category based on four main topographic characteristics associated with landslides (Table 6-1). The four characteristics are assigned a point value between 0 and 10, with a score of 0 meaning the feature is unidentifiable, and a score of 10 meaning the feature is clearly identifiable. The sum of the scores is used to assign a level of confidence.

Table 6-1. Confidence of landslide identification points and scale, adopted from Table 3 (Burns and Madin 2009).

<table>
<thead>
<tr>
<th>Landslide Feature</th>
<th>Points</th>
<th>Confidence</th>
<th>Total Points</th>
</tr>
</thead>
<tbody>
<tr>
<td>Head scarp</td>
<td>0-10</td>
<td>High</td>
<td>≥30</td>
</tr>
<tr>
<td>Flanks</td>
<td>0-10</td>
<td>Moderate</td>
<td>11-29</td>
</tr>
<tr>
<td>Toe</td>
<td>0-10</td>
<td>Low</td>
<td>≤10</td>
</tr>
<tr>
<td>Internal scarps, sag ponds or closed depressions, etc.</td>
<td>0-10*</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Applied a single time so that total points do not exceed 40.

The Prairie Landslide is one of the most well-defined landslides studied and the confidence in the identification of this landslide is rated as high (see Table 6-2 for confidence ratings for the landslides addressed in this study). The most readily identifiable features of this landslide include the head scarp, left flank, and internal scarps and other features. This landslide was previously unmapped, but movement of this landslide is likely responsible for much of the deformation occurring just above the head scarp. The lowest confidence was in the location of
the toe because of the highly eroded nature of the lower part of the slope, which likely reduced the visibility of geomorphic features usually associated with a landslide toe. The fact that this landslide could be identified with a high level of confidence made it an excellent candidate to develop a slope model that could be used to constrain material strength properties and other model parameters, which were recycled in the models for other landslides.

The Lateral Shear Landslide was mapped with a moderate level of confidence due to the overall lack of readily identifiable features. A slide plane identified by an inclinometer installed adjacent to the highway in this area helped constrain some of the boundaries based on its depth. The main issues affecting the overall confidence was the absence of a continuous head scarp and the slightly unusual nature of some of the damage in the highway, which included vertical offset and cracking of pavement across the highway creating a bump, with that same offset not impacting the poured concrete curb on either side of the highway. These cracks and bumps that crossed the highway were interpreted as being the left and right flanks even though these features were not always continuous on either shoulder of the highway.

The Lower Wedge Landslide is likely not a landslide, at least not in the orientation mapped by the Park Service (see Figure 2-1). This landslide was mapped in this study with an overall low confidence. There is no observable head scarp and only a change in slope angle that might indicate a toe. There were few tension cracks, scarps and depressions mapped in this area and any that were mapped were generally oriented in a direction parallel to any potential downslope movement. While a right flank may be tentatively drawn, there was no discernible change in topography on the left, or south side of the area indicating a landslide boundary. Additionally, computer models did not provide any information to help identify a landslide.

The Upper Wedge area has been divided into two different landslides. Upper Wedge NPS is the landslide mapped by the Park Service (see Upper Wedge Landslide on Figure 2-1). The north boundary of this landslide was defined by a scarp and area of instability located on the north slope of the small valley thought to be the body of the Upper Wedge Landslide. The head scarp was mapped along vertical jointing in Millard Ridge and the southern boundary was mapped at the base of the Matterhorn Butte. Movement of this landslide was predicted to be in a southwest direction and mostly perpendicular to the highway. Upper Wedge A is a landslide defined by this study and encompasses only the northern slope of the Upper Wedge area (Figure 5-12). This is the same area initially thought to be the northern flank of the Upper Wedge
Landslide as per the NPS. The distinction between the two is that Upper Wedge A is a separate landslide located on the steep slope on the north side of the valley and is not related to a larger landslide occupying the valley. Movement of the Upper Wedge A Landslide is southwards and into the valley.

Table 6-2. Confidence level of landslides mapped in the field and point values for primary landslide characteristics. Upper Wedge A refers to the landslide mapped by the National Park Service with an orientation of movement perpendicular to the highway. Upper Wedge B refers to the interpretation of the Upper Wedge Landslide from this study with direction of movement parallel to the highway.

<table>
<thead>
<tr>
<th>Landslide</th>
<th>Landslide Feature</th>
<th>Points</th>
<th>Confidence</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prairie Island Landslide</td>
<td>Head scarp</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>Flanks</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Toe</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Internal scarps, sag ponds or closed depressions, compression ridges, etc.</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td>30</td>
<td><strong>High</strong></td>
</tr>
<tr>
<td>Lateral Shear Landslide</td>
<td>Head scarp</td>
<td>4</td>
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</tr>
<tr>
<td></td>
<td>Flanks</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Toe</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td></td>
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<td>5</td>
<td></td>
</tr>
<tr>
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<td>21</td>
<td><strong>Moderate</strong></td>
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<td></td>
</tr>
<tr>
<td></td>
<td>Flanks</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Toe</td>
<td>1</td>
<td></td>
</tr>
<tr>
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<td></td>
</tr>
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<td><strong>Total</strong></td>
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<td>7</td>
<td><strong>Low</strong></td>
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<tr>
<td>Upper Wedge Landslide (NPS)</td>
<td>Head scarp</td>
<td>1</td>
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</tr>
<tr>
<td></td>
<td>Flanks</td>
<td>4</td>
<td></td>
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<tr>
<td></td>
<td>Toe</td>
<td>2</td>
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</tr>
<tr>
<td></td>
<td>Internal scarps, sag ponds or closed depressions, compression ridges, etc.</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td>10</td>
<td><strong>Low</strong></td>
</tr>
<tr>
<td>Upper Wedge A Landslide (this study)</td>
<td>Head scarp</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Flanks</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Toe</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Internal scarps, sag ponds or closed depressions, compression ridges, etc.</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td></td>
<td>35</td>
<td><strong>High</strong></td>
</tr>
</tbody>
</table>
A low confidence was assigned to Upper Wedge NPS because a head scarp and toe could not be identified, nor were there widely distributed internal features. The separation of blocks along nearly vertical joints along Millard Ridge can be attributed to the typical mechanism responsible for producing rockfall, and not the head of a landslide. This process of separation in the cliff was observed in many locations along Millard Ridge above the Cliff Shelf area. The distribution of scarps and tension cracks is limited to the northern slope valley, and there were no obvious landslide features on the south side of the valley. Therefore, it was concluded that slope movement in this area is likely confined to the northern slope of the valley and that scarps and tension cracks forming on the slope are caused by failure of that steep slope towards the valley.

It is interesting to note that the location of most landslide-induced geomorphic features is on the northern side of both the Upper and Lower Wedge areas, closest to Millard Ridge, which in this area is thought to be the head scarp of the Cliff Shelf paleolandslide. These features may suggest a more southerly direction of movement. It is possible that mapped features and highway damage in these locations are associated with continued or reactivated deformation at and near the head of the large paleolandslide. Survey data from 1999-2001, collected from monuments installed by Park and FHWA personnel showed a relatively consistent southwest direction of movement across much of the western side of the Cliff Shelf paleolandslide ranging from 1-13 mm/week (Kumar & Associates 2000, FHWA 2002) (Figure 6-1).

6.2 Parameter influence on slope stability in the Cedar Pass area

The landslides investigated show the highest sensitivity to friction angle, and to a lesser extent, cohesion. This sensitivity to friction angle suggests that while drained conditions may exist at times along the shear plane of the landslide, undrained conditions could significantly lower the stability of the slope and cause acceleration of movement. The formation of undrained conditions is likely linked to an increase in pore pressures caused by the input of water from snowmelt or rainfall into the landslide body.

Sensitivity of these landslides to changes in ground water was apparent, but as not large as sensitivity to friction angle. It is likely that fluctuations in ground water and their related impact on decreasing effective shear strength of the soils is important. This is supported by observations that more movement occurs after periods of wetter than average conditions. The impact of water is discussed more in Section 6.3.
Figure 6-1. Monitoring points for the Cedar Pass and Cliff Shelf Landslides showing relative magnitude and direction of movement (from FWHA 2002). The survey was conducted between April 7, 1999 and October 5, 2001.
Landslides in the Cedar Pass Landslide Complex show very little sensitivity to erosion, especially near the toe. Generally, it is expected that erosion of toe material would increase instability by removing buttressing materials at the base. However, it is likely that this erosion is simultaneously decreasing the driving forces by unloading upper/middle portions of the slope and placing more mass at the toe.

6.3 Cedar Pass Landslide Complex landslide characteristics and failure mode

Based on a review of previous geotechnical investigations, fieldwork and computer modeling conducted for this thesis, the primary mechanism of failure within the Cedar Pass Landslide Complex is translational sliding along low angle shear surfaces with the magnitude of horizontal movement greater than the magnitude of vertical movement. This is supported by observations made in the field for this study as well as borehole data (Kumar & Associates 1999) monument surveying (Kumar & Associates 1999; Kumar & Associates 2000) and an InSAR survey (Anderson et al. 2004). Sliding is possibly occurring on the low dipping surfaces between different geologic units; however, the continuity of these dipping surfaces is unknown. Sliding may also be occurring along stepped surfaces, although there is no evidence to support this, and computer modeling of approximations of a stepped surface produced higher FS values than those of a single, low angle slide plane.

The velocity of the landslides in the complex can be classified as moderate to extremely slow (Cruden and Varnes 1996) based on measurements that showed a maximum rate of 17.8 mm/day and a minimum rate of 0.14 mm/day (Kumar & Associates 1999). A review of photographs provided by the Park Service of highway damage and observations made during this study show that movement rates are likely closer to the lower end of this range, and possibly zero at times. The semi-continuous movement of landslides in the complex may be attributed to a mechanical feedback called dilatant strengthening which has been explained theoretically, tested in a laboratory setting and observed in the field (e.g. Iverson 2005; Schultz et al. 2009). Iverson 2005 and Schultz et al. 2009 describe dilatant strengthening as the process by which a mechanical feedback in the soil controls slope movement. An increase in ground water levels due to rain or snowmelt may increase pore pressures which initiates movement along a shear plane. Shearing can cause soil along the shear plane to dilate which lowers the pore pressures, therefore increasing the effective stress and shear resistance, ultimately slowing landslide.
movement. With time, reconsolidation of the sheared soils may then occur until increased pore pressures trigger the onset of faster landslide movement. This cyclic process depends on a variety of factors, and its efficiency is ultimately dependent upon the ability of the shear zone to dilate and reconsolidate in cycles without settling at the critical state density, and the amount of time it takes for pore pressures to develop, dissipate and redevelop (Iverson 2005). Dilatant strengthening has been used to explain movement patterns of landslides in fine-grained materials (Baum and Johnson 1993; Schultz et al. 2009), and may act as a velocity control of the landslides in the Cedar Pass Landslide Complex. Evidence to support that this mechanism may be controlling movement in the complex includes:

1. The semi-continuous, slow movement of the landslides observed in the field and by other survey methods.
2. The dilative response during shearing of some soil samples tested for this study. This response means that these soils were consolidated to a density greater than the critical state density. Soils tested were collected at the surface and remolded and recompacted in the lab to the density measured at the surface. It is reasonable to assume that soils along the shear plane, that may be as deep as 40 meters, are denser than the soils at the surface, and therefore may exhibit some dilative behavior.

Slope deformation and movement of landslides in the complex appears to be tied to periods of above normal precipitation. The reason precipitation likely impacts the movement of these slides is because the saturation of clayey soils within the landslides resulting from a rising water table can reduce the overall shear strength of that soil. Reduced shear strength can lower slope stability, resulting in movement. Movement at different rates can also reduce shear strength through the development of drained or undrained conditions. Under drained conditions, landslide movement is slow enough that excess pore pressures can dissipate. Under undrained conditions, water cannot escape fast enough leading to an increase in pore pressures and further reduction of shear strength.

Periods of accelerated movement, most notably in the late 1990s to early 2000s, and again in the early to mid-2010s were directly preceded by years with above normal precipitation. Most recently, 2018 and 2019 saw higher than average precipitation, with 2019 receiving more precipitation than any year in the last 50 years. Consequently, highway surface distress,
especially in the Cliff Shelf Trail parking lot and above the Lateral Shear Landslide as of February 2020 was more severe than at any point in the previous 5 years, which is the last time pavement along that portion of the highway was resurfaced. Since 2015, cracks and bumps in the highway had been forming and by the end of July 2018, maintenance had smoothed out the bumps by grinding down the pavement. In April 2019 (the approximate start of the rainy season in southwest South Dakota), there was a noticeable deterioration in highway surface conditions, but they had not returned to those observed before July 2018. However, by February 2020, surface conditions were worse than those observed in 2018. This is likely related to the fact that between April 2019 and December 2019, nearly 760 mm (30 in) of rain fell, which is approximately 450 mm (12 in) more than the average annual amount of precipitation over the past 50 years. Short term climate trends based on 50 years of precipitation data show that annual precipitation amounts have generally increased slightly over the last half century (Figure 6-2). This may explain why instability throughout the complex has become more widespread in that time. It also provides evidence that continued movement of already existing landslides, and the development of new landslides, is possible in the future.

![5-Year Precipitation Averages for Badlands National Park](image)

Figure 6-2. Annual precipitation averages divided into 5-year intervals from 1970-2019. A linear trendline through the data is used to show that annual precipitation has increased over the last 50 years.
6.4 Landslide mitigation

A variety of hazards exist within the Cedar Pass Landslide Complex and Badlands National Park in general, including erosion, swelling soils, frost heave, settlement related to piping, rockfall and landslides. Highway surface damage may be related to one or more of these hazards throughout the park. The landslide hazard is one of the most prominent hazards in the park, as slope instability exists on both the large and small scales, and Badlands Loop Road passes through some of these areas affected by this hazard. The landslide hazard in Badlands National Park is related primarily to infrastructure, specifically roads, and not necessarily to humans because the landslides are typically slow moving and failures do not occur catastrophically. However, it is prudent to assume that the possibility of more rapid failure exists, especially during periods of extremely wet conditions when an increase in landslide acceleration is possible. The main concern associated with this hazard in regard to human safety is the possibility of rapidly deteriorating highway conditions that may result in vehicle accidents. Of the four locations investigated in this study, the Upper and Lower Wedge regions and the Prairie Island Landslide likely pose the biggest challenges in managing the condition of the highway and Cliff Shelf parking lot. These challenges stem from the fact that highway surface distress in the Upper and Lower Wedge regions may be related to large scale deformation occurring in the Cliff Shelf paleolandslide and the fact that the Prairie Island Landslide is of sufficient size that stabilization of the entire landslide may not be feasible.

Buttresses appear to be the most effective method of remediation if the structure can be built against an intact butte (the 2015 buttress) or reasonably close to the base of a slope (the 2000 buttress). However, the long-term effectiveness of buttresses may suffer from erosion. This has been observed in the 2000 buttress, and the 2015 buttress is already experiencing similar problems and the highway above the structure is showing signs of distress in the form of multiple pavement offsets and cracking. Prairie Island Landslide would likely require such a large buttress and substantial work to control drainage that this is not a feasible solution for mitigation of that landslide. However, this study showed that tieback anchors and piles may be effective to stabilize the upper scarp of the Prairie Island Landslide. A drawback to piles is that they are generally better suited for soils that will not creep between individual piles (Abramson et al. 2002). The fine-grained soils in the park at least have the potential to cause this problem.
One of the most common methods of improving stabilization is decreasing the amount of water in the ground by installing horizontal drains. One of the biggest issues with horizontal drains is that the migration of fine-grained soils into the drain, such as the soils found at the Cedar Pass area, may plug and shorten their life spans. A potential alternative to the traditional horizontal drains is driven wick drains. Wick drains have several advantages, including that they are more resistant to clogging, are relatively cheap to install, and are able to accommodate slope deformations due to their flexibility. These drains could be used, especially in the highway embankment by the Upper Wedge Landslide and in the slope below the Lateral Shear Landslide. These types of drains have been shown to be effective at reducing water table heights in slopes and show life spans similar to conventional drilled horizontal drains, even in clay soils (Santi and Elifrits 2001). However, a limitation of wick drain use in the park is that the ideal material for driving wick drains has an SPT value less than 30 (Santi and Elifrits 2001). Many materials encountered in the landslide complex have SPT values greater than 30 and may create issues when driving the drains.

Controlling both surface water and groundwater is a difficult task in the Badlands. The fine-grained nature of the soils means that infiltration is low, runoff is high, and the potential for piping is very high. Piping is the evacuation of fine-grained particles underground by seepage which eventually results in the creation of an underground cavity (pipe). Over time, the pipes can become large enough that soils above collapse and create a sinkhole. The Cedar Pass Landslide Complex, along with many areas of the park, have high numbers of piping sinkholes and other piping features. Piping features were observed close to the highway, and it is possible that piping is responsible for some of the damage to the highway. This may be the case near the Lateral Shear Landslide where water has been observed seeping out of the slope below the highway and where a sinkhole was discovered on the inside of the bend in the highway during a construction project (Ellen Starck, personal communication, June 2018). Culverts may also increase the potential for piping, as water is normally directed to these locations and any gaps between the culvert and surrounding soil may provide a preferential path for water to flow. Because runoff rates can be so high, there is the potential for a significant amount of water to collect on a slope and run into open tension cracks on landslides, increasing the amount of water in the slope.

In areas such as the Upper and Lower Wedge, where surface deformation may be related to large-scale landslide movement of the Cliff Shelf paleolandslide, a more flexible pavement or
a supported subgrade that can withstand more deformation may be a preferable option in those locations that require constant maintenance. A flexible subgrade or pavement will not eliminate the need for maintenance, but may help reduce the frequency with which maintenance is required.

6.5 Limitations

This study was conducted using previously collected and available geotechnical data and extensive fieldwork conducted by the author. It should be noted that landslide investigations generally depend upon site specific data, which while available for some areas of the complex, were not available in others, most notably the Prairie Island Landslide. The accuracy of slope stability models is dependent on the ability to precisely constrain parameters such as water table elevation, material strength, landslide boundaries and slide plane location and shape. This study, using the method described in Section 4.3.1, was successfully able to constrain model parameters in order to provide reasonable results. However, more detailed data collected in the future related to subsurface conditions and material properties may provide a higher level of confidence in these results.

6.6 Recommendations for future studies

Futures studies within this landslide complex should focus on identifying rates of movement across the complex as rates are likely to vary spatially. This can be done using a variety of remote sensing techniques, including Interferometric Synthetic Aperture Radar (InSAR) and Light Detection and Ranging (LiDAR), which can efficiently provide data over a large region. LiDAR surveys can be used to create point clouds that can be differenced to show areas of the most movement and deformation. One specific issue with these surveys is that high erosion rates in some areas may mask landslide movement, especially if landslide movement is less than the magnitude of erosion. A survey of monitoring points throughout the complex may also be a useful method in checking whether or not landslide movement is occurring on a large scale. The last survey of this kind in the complex was conducted nearly 20 years ago.

Three of the four landslides discussed in this thesis have instrumentation installed, and it is recommended that an inclinometer be installed at the southeastern edge of the Cliff Shelf parking lot in order to identify the slide plane causing settlement in the parking lot. Additionally,
continued monitoring of piezometers and the installation of more piezometers, combined with movement data may help reveal whether or not dilative strengthening is controlling movement in the landslide complex.

Lastly, the analysis of mitigation options in this study was limited, and while it provided a comparison of certain methods, a detailed geotechnical investigation in specific areas and a unique design will be required in order to choose the best mitigation strategy.
CHAPTER 7
CONCLUSIONS

The stability and sensitivity to changes in material strength, groundwater fluctuations, and erosion of landslides in the Cedar Pass Landslide Complex was estimated using 2D limit equilibrium slope stability models. These models were also used to investigate the effectiveness of different mitigation techniques in improving slope stability. Individual landslide boundaries within the complex were refined using field observations and mapping. Three landslides identified by the Park Service and one landslide identified during this study were investigated. The slides, called the Prairie Island Landslide, Lateral Shear Landslide, Upper Wedge Landslide and Lower Wedge Landslide are located within the Cliff Shelf paleolandslide in the southern region of the complex.

Field observations could not confirm the existence of the Upper and Lower Wedge landslides in the orientation mapped by the Park Service. Computer modeling also failed to produce reasonable solutions for landslides in both these locations. Highway surface damage across the Upper and Lower Wedge areas is thus explained by deformation occurring along the upper boundary of the Cliff Shelf paleolandslide. In the case of the Upper Wedge area, highway surface damage may also be the result of small-scale slope instability adjacent to an embankment fill, or erosion/settlement within that fill.

Highway surface damage west of the Cliff Shelf parking lot and east of the 2015 buttress is caused by the Lateral Shear Landslide that encompasses the highway and slope immediately south of the highway above the major drainage channel. Monitoring shows a slide plane located approximately 11 meters below the south shoulder of the highway.

The Prairie Island Landslide is the largest landslide that was investigated and encompasses much of the slope below the Cliff Shelf Trail. Southern and downhill movement of this slide is causing smaller areas, or blocks, of instability above the head scarp as identified in the field. Movement of these blocks is causing the damage to the boardwalk and settlement of the southeastern side of the parking lot.

The landslides showed sensitivity to changes in ground water levels with up to a 47% change in Factor of Safety when ground water is raised to seasonally high levels. Undrained conditions in which the rate of loading exceeds the rate of pore pressure dissipation may cause periods of faster movement.
Some samples showed dilative behavior during shearing, so soils may be affected by
dilative strengthening, which would explain the cyclic nature of movement of these landslides.

Earthen buttresses can stabilize slopes in this area, as demonstrated by projects in 1999
and 2015. However, the long-term stability of these structures may be affected by piping and
erosion. Other mitigation options that may be practical are the installation of reinforced earth
retaining walls, tieback walls or pile walls, assuming these structures can be founded on stable
areas. Tiebacks and piles could be used to stabilize the head scarp of the Prairie Island Landslide
which is damaging the Cliff Shelf parking lot.

Traditional horizontal drains could become plugged over time due to the migration of
fines into the drains, but horizontal wick drains, which are more resistant to clogging, may be
effective in areas such as the Lateral Shear Landslide and the Upper Wedge embankment fill
where materials may be soft enough for wick drains to be driven into the slopes.

In areas with regular surface deformation, flexible pavement or a flexible road base may
be the most practical method to increase the amount of time between maintenance projects.

Precipitation data recorded over the last 50 years in the park shows an increase in the
average annual precipitation. The observed and modeled negative impact that an increase of
water into the landslide complex has on slope stability suggests that slope stability issues,
including damage to Badlands Loop Road, the Cliff Shelf parking lot, and other areas of
Badlands National Park, may persist.
REFERENCES CITED


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APPENDIX A
SUMMARIES OF PAST GEOTECHNICAL STUDIES (1998-2016)

See references cited for full citations for each study. The majority of reviewed documents were obtained from the National Park Service.

A.1 Kumar & Associates, 1998

In 1998, Kumar & Associates, a Denver-based consulting company prepared a report for the FHWA summarizing site observations made by the company as well as current surface and subsurface information concerning the landslide affecting the Badland Loop Road and a scenic overlook directly south of the Cedar Pass summit. The report also describes observations made by engineering geologist, Perry Rahn in 1993 and South Dakota Department of Transportation engineer Vernon Bump in 1996. Subsurface data was obtained from seven boreholes drilled in 1990 and the subsequent installation of piezometers in three of the boreholes. The borings in 1990 were advanced to a depth of 15.2-29.0 meters (50-95 ft). Logs were not detailed and were not useful in identifying the material encountered while drilling. The three piezometers showed water levels as shallow as 3 meters (10 ft) to as deep as 24.7 meters (81 ft) and both piezometers located downhill of the road sheared off at 4.9 and 12.8 meters (16 and 42 ft) by 1992.

As part of the report, Kumar & Associates also addresses the settlement in the outside shoulder of the highway below the Cliff Shelf Parking Lot by suggesting that the area of damage is probably a result of erosion along the downslope shoulder coupled with the fact that the road is built in landslide terrain. The report does mention the possibility that the distress is caused by the onset of larger-scale slope movement and they suggest remedial measures including improving drainage and replacing the road fill with mechanically stabilized fill.

Main conclusions:

● The landslide is generally made out of bedrock material with some bedrock blocks showing bedding that has been rotated out of horizontal.

● Review of aerial images revealed the presence of N 75 W tension cracks and N 65 to 80 W joints that are possibly controlling the location of the head-scarp of the slide. The slide is moving in a southerly direction.
• Erosion at the toe (which is difficult to locate) is causing the loss of resisting forces while fill placed for the scenic overlook and rockfall from Millard Ridge are adding weight and increasing driving forces.

A.2 Kumar & Associates, 1999

The Kumar & Associates report in 1999 is a follow up to the 1998 report that details the study done to support the design of a stability berm to stabilize the portion of the Cedar Pass landslide impacting Badlands Loop Road at the pass. The study included six borings at Cedar Pass (and four at the Cliff Shelf location discussed in the 1998 report), soil/rock sample collection, the installation of inclinometers, and the installation and survey of monuments established from Cedar Pass throughout the Cliff Shelf area.

Main conclusions:
• The Cedar Pass Landslide consists of high plasticity clay with weathered, sandy claystone at depth.
• Measurements of sheared inclinometers suggest the landslide is moving translationally along a shear plane with a dip of less than 2 degrees.
• Higher levels of precipitation may be responsible for increased movement with average horizontal movement rates on the order of 10-20 mm per day for the Cedar Pass Landslide.
• Monument movement data suggest the Cliff Shelf Landslide is no longer dormant.
• A potentially broken waterline may be partly responsible for localized movement downgrade of the Cliff Shelf parking lot.

A.3 Kumar & Associates, 2000

This report discusses the results of the assessment of current and future material losses within the Cedar Pass Landslide in preparation for the construction of a large stability berm that was constructed beginning in September 2000. The report addressed whether the construction of a berm was still a viable option for remediation as well as analyzing the possibility of rerouting the highway around the slide as an alternative option to the berm. The evaluation of material losses was carried out by comparing topographic profiles created in 1998 and 1999 constructed
from aerial images and survey points, as well as compiling movement rates based on monument measurements. While the report focused on the Cedar Pass Landslide, the Cliff Shelf Landslide is also briefly addressed and an investigation and monitoring plan is suggested.

Main conclusions:

- Significant movements of the Cedar Pass Landslide in the past several years have been observed after periods of above average precipitation.
- The advantages of road realignment above the head scarp include placing the highway on more stable ground and reducing the driving force on the landslide with the removal of the road.
- The disadvantages of realignment include eastward migration of the head scarp undercutting the new road due to continued southward movement of the landslide or possibly because of poor drainage along the new alignment allowing the infiltration of more water above the landslide mass.
- Movement rates and directions measured within the Cliff Shelf Landslide are variable suggesting a complex slide mechanism. The landslide may be a deep, translational-block slide but investigations up to the date of the report have not been able to confirm this. Boreholes advanced to approximately 10 meters did not encounter a distinct slide plane which suggests that movement is occurring at greater depths.
- Horizontal movement rates range from 1-13 mm per week.
- Resistant ridges and intact buttes within and at the edges of the landslide may be causing the variations in direction of movement and movement rates.

A.4 Federal Highway Administration, 2002

In the early 2000s the Federal Highway Administration conducted a study to assess the feasibility of re-routing SR-240 around the Cedar Pass Landslide Complex. The study developed four preliminary alternative alignments with two additional alignments produced from modifications to two of the preliminary routes. A cost estimate for each route was also calculated to address the economic feasibility of constructing any of the proposed routes. When developing new routes, the following factors were considered including:

1. length of the new roadway deviating from the existing route
2. total length of the alignment
3. height and volumes of cuts and fills
4. visual experience
5. area of land required outside the park boundary
6. stream crossings
7. impact on traffic flow through the eastern portion of the park
8. other environmental impacts

The study was conducted by using aerial imagery and field verification. Geologic hazards such as slope stability and groundwater problems were evaluated by a Federal Highway engineer. All alignments were designed to minimize the amount of distance spent on the Badlands Wall due to the amount of distress observed in the roadway at Cedar Pass and Norbeck Pass (the pass located approximately 9.7 road kilometers (6 mi) west of Cedar Pass). A description of each route is followed by a summary of the advantages and disadvantages of each route. Proposed alignments were discussed with park staff in 2001 with an associated field trip along the new alignments to observe the routes. The results of the discussion and field trip were used to revise some of the preliminary alignments and select the most desired options.

A.5 Anderson et al., 2004

InSAR surveys were conducted from 1999-2002 using satellite data to study land deformation and movement along the highway through the Cedar Pass area. The results of the InSAR survey were compared with measurements obtained from survey points installed by Federal Highway Administration personnel. Ground-based observations made at the Cedar Pass Landslide in 1999 showed horizontal and vertical movement rates on the order of over 10-20 mm/day to less than 1 mm/day. Generally, horizontal movement rates were greater than vertical rates. Movement within the Cliff Shelf Landslide was less than 1 mm/day and areas of upward and downward movement were randomly distributed across the study area. InSAR observations in 1999 showed deformation rates within the Cliff Shelf Landslide similar to the rates obtained from monument survey data.

Ground observation in 2000 showed that the movement within the Cliff Shelf Landslide had increased slightly, with movement rates now a little above 1 mm/day. The report notes that
almost all of the upward movement recorded was occurring near the toe of the landslide. InSAR-derived movement rates were similar to those recorded the previous year. InSAR also revealed that some areas outside of the two landslides had also moved. The observation period in 2000 was preceded by a period of above normal precipitation in the spring, but normal precipitation throughout the summer.

Observations made by InSAR in 2002 showed very little to no movement in both the Cedar Pass and Cliff Shelf Landslide. However, the 2002 observation period was the shortest out of any in the entire study, and it was an “exceptionally dry year.”

Main conclusions:
- New areas of movement both inside and outside the mapped landslides were identified.
- The mode of deformation in the area does not appear to be rotational as evidenced by the lack of large areas showing upward movement.

A.6 Parsons Brinckerhoff Quade & Douglas, 2004

This study summarizes work done by Parsons Brinckerhoff Quade & Douglas for the FHWA to rehabilitate the highway surface along approximately 24.1 km (15 miles) of Badlands Loop Road, including the portion of the highway through the Cedar Pass Landslide Complex. Part of the scope of work included the evaluation and recommendations for addressing road surface distress caused by slope movement in the Cedar Pass area. A subsurface investigation was conducted with the drilling of two boreholes (DB-1, DB-2) in the road surface in the landslide complex. These boreholes were advanced to a maximum depth of 8.1 meters (26.5 ft) and encountered clay fill followed by weathered claystone from 1.4-3.0 meters (4.5-10 ft) below the road surface to the bottom of the borehole. Samples collected within the claystone were moist to very moist, which the investigators attributed to surface water infiltration or a perched water table. Roadway fill materials are mostly fine-grained clays with a moderate to high swell potential and moderate to high plasticity.

Main conclusions:
- Roadway distress is possibly caused by wear-and-tear due to traffic loads, swelling of subgrade soil and differential heaving, frost heave and thaw and other temperature-related
processes, deep-seated landslide movement, and piping of fine-grained materials in the embankment fill.

- The recommended method to minimize roadway damage in the Cedar Pass caused by slope movement is to provide adequate drainage away from the road surface and subgrade materials and expect frequent maintenance. At the time, it was not considered economically feasible to mitigate landslide movement.

A.7 Federal Highway Administration, 2012

This technical memorandum details the results of a subsurface investigation within the Cliff Shelf Landslide conducted by FHWA, specifically addressing the slope movement observed directly west and downhill of the Cliff Shelf parking lot. In August 2010, an embankment failure was observed followed by surface distress in the highway which had recently been resurfaced. As a result, a subsurface investigation was proposed and carried out which included 6 borings advanced to depths of 15.2-19.8 meters (50-65 ft). Inclinometers were installed in two of the borings and piezometers in three other boreholes. Geologic units encountered in the holes generally followed the pattern of a thicker sequence of clay with a thinner sequence of highly weathered, weak siltstone, underlain by highly weathered, weak to strong claystone. Groundwater was not encountered during drilling but it was encountered at roughly 9.1 meters (30 ft) below the ground surface in subsequent visits to take inclinometer and piezometer measurements. Groundwater levels fluctuated between 0 and about 1.2 meters (0-4 ft) in the later spring to mid-summer time period. Inclinometer data points to a slide plane between 9.1-10.7 meters (30-35 ft) and at 15.2 meters (50 ft) below the highway.

Main conclusions:

- Two slide planes are present, with one located between 9.1-10.7 meters (30-35 ft) depth and the second at 15.2 meters (50 ft). The failure surfaces and tension cracks indicate the landslide is moving translationally.
- Landslide movement is likely caused by fluctuations in groundwater level between the wet and dry seasons and that movement will likely continue to occur until water levels drop.
- Mitigation of this landslide would be “significant” and “without guaranteed results.”
• They recommend continuing to monitor installed instruments, field reconnaissance to estimate the extent of the slide, controlling surface water to prevent infiltration into open cracks, regular maintenance of the roadway surface, or the installation of a deep patch or shoulder stabilization.

A.8 Federal Highway Administration, 2013

This report by FHWA discusses the Cliff Shelf Landslide, specifically the smaller section located directly west of the Cliff Shelf Trail parking lot impacting the downslope shoulder of the highway. The report addresses this landslide by presenting the results of a subsurface investigation as well as recommendations of mitigation options to stabilize the highway. Remedial measures analyzed include improving drainage, excavation of failed materials and construction of an earthen buttress, construction of a retaining structure, and deep foundations for the road and shoulder.

As in the previous report, the sensitivity of the landslide to higher than normal rainfall is emphasized. Based on an analysis of annual precipitation in the park and noting that previous major slide movements occurred in 1998 and 2011, the report states that a “definite” correlation exists between slope movement and periods of above average precipitation. The report goes on to say that there is a likely a delay in the onset of slope movement after the period of high precipitation because water transmission in the fine-grained soils is low and therefore it may take months before the slide plane becomes saturated and the clays lose their strength. It was hypothesized that groundwater flows along the contact between the clay, siltstone and claystone encountered in borings and that the groundwater is likely recharged from surface water that ponds due to the irregular topography of the area.

An analysis of four mitigation options (mentioned above) was carried out utilizing slope stability modeling. The first option was to continue maintenance. The report states that this option did not lower the risk of significant landslide movement but that the alternative of stopping maintenance operations would lead to the highway becoming impassable within 5 years. The second option was to install a deep patch beneath the road surface to improve the road foundation. The third option considered was the construction of both a deep patch and buttress. The fourth option was to realign the highway upslope of the landslide. The downside of this option was that a realignment would lead to a steeper road grade and tighter curves requiring a

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reduction in speed limit. More importantly, this option only protected the road in the near future but did not protect against other movement further upslope. This has been a common conclusion of engineers that relocating the road within the landslide is likely not a permanent solution unless the entire area is avoided. The ramifications of such a reroute were addressed in a 2002 realignment study conducted by the Federal Highway Administration (see above). Ultimately, the third option was chosen and a buttress and deep patch were constructed in 2015.

Main conclusions:

- Extended periods of higher than average precipitation is likely causing movement.
- Slope stability analysis showed that the stability of the slope decreased as the groundwater level rose into the clay layer which is the soil unit encountered to about 12.2 meters (40 ft) beneath the road surface.

A.9 Zonge International, Inc., 2013

This report summarizes a geophysical study conducted in the Cliff Shelf area with the objectives of measuring the conductivity, strength and stiffness of landslide material as well as to locate any vertical or lateral inhomogeneities within the landslide mass that may suggest the location/depth of a slide plane. Specifically, investigators hoped that conductivity could be correlated with water content. Additionally, researchers interpreted p-wave and s-wave velocities of 1310-1430 m/sec (4300-4700 ft/sec) and 274 m/sec (900 ft/sec), respectively, obtained from the seismic surveys as representing the stiffness contact where the slide plane was most likely to occur. These values were based on velocities measured at depths where displacement was measured in adjacent and nearby inclinometers.

The study was conducted using seismic refraction, Multi-Channel Analysis of Surface Waves (MASW), and electrical resistivity along five survey lines located along the highway directly west of the Cliff Shelf Trail Parking Lot and along lines trending northwest from the curve in the highway by the parking lot.

Main conclusions:

- An abrupt change in p- and s-wave velocities was detected in sections of two profiles.
- Researchers concluded that variations in resistivity are likely related to changes in subsurface moisture content because of the relatively homogeneous nature of the
subsurface materials in the study area. Testing revealed areas of anomalously low resistivity near the surface in two of the three resistivity survey lines which are thought to be related to piping features.

- Stiffness contours from p-wave velocities, thought to show the depth of the slide plane, ranged from 6.1-24.4 meters (20-80 ft) below ground surface. Shear wave results seemed to put the slide plane at a similar depth with the 274 m/sec (900 ft/s) contour being roughly 9.1-18.3 meters (30-60 ft) below ground surface in most cases.

**A.10 HDR Engineering, 2014**

This memorandum provides an overview of the parameters used to design a new stormwater conveyance system along Badlands Loop Road in the Cliff Shelf area extending towards Cedar Pass. The new system was designed to capture surface water on the uphill, north and east sides of the road to prevent runoff from entering the slide area on the downhill, south and west sides of the highway. All captured water was designed to be transported south of the project area and away from the existing slide mass.

The project consisted of plugging the 8 existing cross drain culverts north of the Cliff Shelf parking lot with grout or concrete and installing a storm drain line in the east ditch along the highway. The drain system was designed based on a 10-year flood event peak flow rate of approximately 2.35 cubic meters (83 cubic feet) per second originating from an approximately 6.5-hectare (16-acre) basin. Basin delineation was determined using USGS topographic maps.

Construction of the storm drain system was part of other proposed improvements including the reconstruction of a section of the highway and the construction of an earthen buttress in the late summer and fall of 2015.

**A.11 Yeh & Associates, 2016**

This report summarizes the drilling and instrumentation completed by Yeh and Associates Inc. in 2016. This is the most recent work completed that is available for review.

Drilling was completed in October 2016 and included advancing 9 boreholes to depths ranging from 15.5-27.7 meters (51-91 ft). Investigation focused on the landslides identified as Upper and Lower Wedge, Lateral Shear and the area exhibiting highway distress adjacent to the 2015 buttress. Locations were along the road from approximately mile marker 3.95 to mile
marker 4.3. Subsurface samples were collected by a Modified California sampler or split spoon sampler in borings PZ-103 and SI-105 located at the Lateral Shear and Upper Wedge, respectively. Instrumentation installed included 5 inclinometer casings, 2 vibrating wire and 2 open stand pipe piezometers.

Summary of investigation results:

- Groundwater was not encountered near the 2015 Buttress and Lateral Shear Landslide. Borehole depths in those locations ranged from 13.7-27.4 meters (45-90 ft) below ground surface. At the Lower Wedge, groundwater was encountered at depths of 15.5 and 18.3 meters (51 and 60 ft), and at Upper Wedge, at depths of 6.1 and 15.2 meters (20 and 50 ft). The report mentions that there is unverified information of springs to the east of Upper and Lower Wedge landslides. Groundwater variations were explained as being dependent on seasonal changes and properties of the site surface and groundwater drainage. Investigators note that seasonal perched zones may exist.

- In regards to the subsurface geology at the Upper and Lower Wedge landslides, drilling typically encountered more sandy material underlain by clay and then claystone and siltstone. While the claystone and siltstone were described as bedrock, the lateral continuity of the bedrock is largely unknown due to the fact that materials encountered are thought to have been displaced.
# APPENDIX B

## LABORATORY RESULTS

Table B-1. Results of the direct shear test.

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<th>Trial</th>
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<th>Sample 7</th>
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<th>Load (kg)</th>
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<th>Load (kg)</th>
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Figure B-1. Shear failure envelope for Sample 7 with computed cohesion and friction angle values.

Figure B-2. Shear stress vs shear displacement graph for Sample 7.
Figure B-3. Shear failure envelope for Sample 9 with computed cohesion and friction angle values.

Figure B-4. Shear stress vs shear displacement graph for Sample 9.
Figure B-5. Shear failure envelope for Sample 11 with computed cohesion and friction angle values.

Figure B-6. Shear stress vs shear displacement graph for Sample 11.
Figure B-7. Shear failure envelope for Sample 14 with computed cohesion and friction angle values.

Figure B-8. Shear stress vs shear displacement graph for Sample 11.
Figure B-9. Shear failure envelope for Sample 17 with computed cohesion and friction angle values.

Figure B-10. Shear stress vs shear displacement graph for Sample 17.
Table B-2. Direct shear test parameters. Sample 11 was the first sample tested and a bracketed depth of 45 meters was chosen. This value was revised in the other four tests to 40 meters so the lower end of the bracket was decreased to 5 meters.

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APPENDIX C
BADLANDS NATIONAL PARK PRECIPITATION DATA

Badlands National Park Precipitation
Station: Interior 3 NE
State: SD
County: Jackson
Station Index No.: 39-4184-05
Measurement: Inches
Current Location: Elevation: 2440 ft. (743.7 m.) Lat: 43.7483 N Lon: -101.9413 W
Table C-1. Tabulated monthly precipitation data from 1970-2019, in inches, for Badlands National Park, recorded at the Reifel Visitor Center. Red cells indicate missing/incomplete data. Estimated values for those months are taken from the 50-year average for that specific month. Records were obtained from the National Oceanic and Atmospheric Administration National Centers for Environmental Information. Retrieved from https://www.ncdc.noaa.gov/cdo-web/datasets/GSOM/stations/GHCND:USC00394184/detail.

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**Table C-1 continued**
Figure C-1. Annual precipitation from 1970-2019, recorded in millimeters, for Badlands National Park, recorded at the Ben Reifel Visitor Center. The orange line indicates the 50-year annual average of 459 mm.
Figure C-2. Annual precipitation from 1970-2019, recorded in inches, for Badlands National Park, recorded at the Ben Reifel Visitor Center. The orange line indicates the 50-year annual average 18.1 inches