March 16, 2017

Kadrmas, Lee & Jackson, Inc.
4585 Coleman Street
Bismarck, North Dakota 58503-0431

Attn: Ms. Jen Turnbow and Mr. Troy Ripplinger

RE: PRELIMINARY GEOTEchnICAL DESIGN MEMORANDA,
US-85 - I-94 TO WATFORD CITY BYPASS, PROJECT NO. 9-085(085)075, PCN
20046, BILLINGS, MCKENZIE, AND STARK COUNTIES, NORTH DAKOTA

We are pleased to submit our preliminary geotechnical design recommendations for the above-referenced project. The enclosed technical memoranda address the following:

- **Technical Memorandum TM-1, Landslide Considerations, Long X Bridge:** Alignment selection and design issues related to landslides along the south bank of the Little Missouri River.

- **Technical Memorandum TM-2, Preliminary Cut and Fill Slope Recommendations:** Recommended slope angles and discussion of slope stability issues for the portion of the alignment passing through landslide-prone badlands terrain.

- **Technical Memorandum TM-3, Preliminary Foundation Considerations, Long X Bridge:** Foundation alternatives, including preliminary axial design parameters, for the proposed Long X Bridge improvements.

- **Technical Memorandum TM-4, Slope Stability Analyses Methodology:** Description of the slope stability analyses used to develop the cut and fill slope recommendations provided in TM-2.
We appreciate the opportunity to be of service to you on this project. If you have any questions or require further information, please contact me at 303-825-3800.

Sincerely,

SHANNON & WILSON, INC.

Gregory R. Fischer, Ph.D., P.E.
Senior Vice President

Encl: Technical Memorandum TM-1, Landslide Considerations, Long X Bridge
      Technical Memorandum TM-2, Preliminary Cut and Fill Slope Recommendations
      Technical Memorandum TM-3, Preliminary Foundation Considerations, Long X Bridge
      Technical Memorandum TM-4, Slope Stability Analyses Methodology
TECHNICAL MEMORANDUM TM-1

LANDSLIDE CONSIDERATIONS, LONG X BRIDGE
This memorandum discusses preliminary considerations with respect to the location and length of the proposed Long X Bridge improvements on US Highway 85 (US-85) over the Little Missouri River. This document supersedes our December 29, 2015 version of this memorandum.

EXISTING BRIDGE

The location of the existing Long X Bridge is shown in Figure TM-1-1. The bridge is a two-lane, 950-foot long, three-span, truss structure reportedly supported by driven pipe and H piles. The bridge was constructed circa 1960.

The North Dakota Department of Transportation (NDDOT) has indicated that records from the 1960s and 1970s describe some movement at the south end of the bridge. However, measurements of the movement are not available. Additionally, NDDOT indicated that their records contain photos showing distress to the end posts at the sound end of the bridge that is indicative of movement of the south abutment to the north. However, the movement has not apparently affected the main span of the bridge.
REGIONAL GEOLOGY

Surficial deposits near the existing bridge are mapped\(^1\),\(^2\) as Quaternary-age alluvium and landslide deposits. Mapped landslide deposits are shown in Figure TM-1-2. As indicated in the figure, a relatively large landslide complex is mapped on the south bank of the Little Missouri River. The mapped landslide complex includes the south abutment of the existing bridge and is indicated to terminate between the south abutment and adjacent pier. Laterally, the landslide complex is mapped extending about 800 feet downstream and 2,000 feet upstream of the bridge. The extent of the mapped landslide is not exact and should be considered relative to the map scale and the mapping methods (review of aerial imagery). Based on our site observations (see below), the landslide toe appears to extend further north towards the Little Missouri River than shown in the geologic maps.

On the north side of the bridge, only alluvial deposits are mapped near the existing structure. Bedrock underlying the site is mapped as the Sentinel Butte Formation, which is described as “alternating beds of grayish brown to gray sandstone, siltstone, mudstone, claystone, and lignite.”

SITE CHARACTERIZATION

Engineers and geologists from Shannon & Wilson completed multiple reconnaissances of the project alignment between October 2014 and October 2016. Details of these reconnaissances are available in the project Subsurface Characterization Report (SCR)\(^3\). As indicated above, a site plan showing the locations of mapped and observed landslide features is provided in Figure TM-1-2.

Additionally, Shannon & Wilson completed eight subsurface explorations (designated RP-126-02 through RP-126-08, and RP 126-02A) near the location of the bridge. The locations of the borings are shown in Figure TM-1-1. The boring logs and laboratory test results for samples

\(^2\)Surface Geology, Lone Butte, North Dakota Quadrangle, 2003, Murphy, E.C., scale 1:24,000, North Dakota Geological Survey.
collected from the borings are provided in the project Geotechnical Data Report (GDR). A generalized subsurface profile presenting stick logs of the borings is provided as Figure TM-1-3. A discussion of the geologic units shown in the subsurface profile is provided in Technical Memorandum TM-3. Three of the borings (RP-126-02A, RP-126-03, and RP-126-08) were completed as inclinometers to monitor potential slope movement. Data from the inclinometers are provided in the GDR.

Based on the reconnaissance, subsurface explorations, and instrumentation, geotechnical characterization pertinent to the evaluation of potential bridge locations is summarized below:

- The toe of the large, deep-seated, ancient landslide complex is near the south pier of the existing bridge (Unit Qls1). However, based on the borings, landslide debris appears to extend some distance into and beneath the river channel (Unit Qls2), where the material is mantled by alluvium. The landslide debris buried in the river channel may be related to runout of ancient landslide movement into the ancient river channel.

- Near the bridge, on the south bank of the river the landslide debris appears to be approximately 35- to 40-feet thick (see Figure TM-1-2).

- Beginning about 150 feet west of the bridge and extending about 1,000 feet further upstream, there is significant recent landslide activity in and near the riverbank. Scars associated with the recent activity are indicated in Figure TM-1-2.

- All three inclinometers completed near the bridge indicated slope movement over the monitoring period (approximately two months for inclinometer RP-126-02A and one year for RP-126-03 and RP-126-08). In each inclinometer, the movement occurred at the bedrock contact (approximately elevation 1,940 feet), at a rate of about 0.3 inch per year. The inclinometers indicated movement oriented about 20 to 30 degrees east of north (see Figure TM-1-1).

**LANDSLIDE CONSIDERATIONS – ALIGNMENT SELECTION**

We understand that KLJ is considering several alignments for the proposed bridge. For the evaluation herein, we considered potential alignments located within approximately 100 feet (east or west) of the existing bridge alignment. The location of the proposed bridge alignment should consider two potential landslide mechanisms: 1) relatively shallow, small landslides localized to the south bank of the Little Missouri River and 2) the relatively large, deep-seated
landslide complex that appears to terminate near the south bank of the Little Missouri River. Each of these landslide mechanisms is discussed below.

**Local Bank Failures**

As shown in Figure TM-1-2, we observed several recent active slope failures in the south bank of the Little Missouri River. In our opinion, these active failures are likely the result of the river scouring and undercutting landslide deposits located at the nearby outside bend in the river channel. Based on the alignment of the river channel, the apparent lack of recent slope failures within about 150 feet of the existing bridge, and the flatter topography in the vicinity of the existing bridge, it is our opinion that the risk of shallow slope failures under existing topographic conditions is low within 100 feet of the existing alignment. However, there is a slight preference for the east side of the existing bridge, because bank failures were not observed downstream of the existing bridge.

Even if localized bank landsliding would occur, because such failures would be relatively shallow, the bridge foundations could be designed to accommodate lateral loading caused by such a failure. Localized grading (flattening of riverbanks) and slope armoring could also be completed to improve stability of the riverbanks.

**Deep-Seated Landslide**

Recent inclinometer readings indicate that movement has occurred near the south abutment of the existing bridge, at a depth of about 38 feet. We interpret this movement as being associated with the deep-seated landslide complex described above. If this movement was extrapolated using the rate observed in the inclinometers near the bridge between 2015 and 2016, movement over the approximately 55-year lifespan of the bridge would be approximately 16 inches. However, it is unlikely that the landslide moves at a constant rate as indicated from our limited monitoring period. Further, there is no indication that this magnitude of movement has occurred at the south abutment. While past movement is suspected at the south abutment, as discussed above, the magnitude of this movement is uncertain. The structure has apparently tolerated any movement with relatively minor distress. In our opinion, there are several possible explanations for the lack of significant distress:

- The landslide movement has been episodic and relatively minor over the life of the bridge (the recent inclinometer data is not representative of long-term movement).
• The south abutment and south pier abutment of the bridge have been capable of resisting the landslide movement (the foundations have not been displaced sufficiently to be noticeable and/or the landslide has flowed around the bridge substructure elements).

• Some combination of the above.

Based on the reconnaissance and the inclinometers, it is not clear if the movement indicated at the bridge abutment (inclinometer RP-126-02A) is associated with a failure constrained to the area within a few hundred feet of the river bank or with a much larger failure, potentially encompassing the mapped extents of the landslide, which extend over 1,500 feet south of the river bank. To assess which of the aforementioned failure mechanisms most likely caused the movement in the inclinometer, we completed slope stability analyses to determine the more critical (less stable) of these two failure surfaces. The analyses suggest that the most critical failure mechanisms are 1) a large failure encompassing the extent of the mapped landslide and 2) small failures located within about 100 feet of the river bank. The analyses indicated that intermediate failure surfaces were more stable. The analyses further suggest that the movement observed at the inclinometers is the result of movement of the relatively large mapped landslide. Movement indicated by inclinometer RP-126-03 could also be associated with localized instability of the riverbank.

To the extent that the large mapped landslide is active, design considerations for the proposed bridge are unlikely to depend on whether the bridge is located to the east or west of the existing alignment, because the mapped landslide deposits extend well beyond the proposed alignment limits. However, the alignment located to the west could involve significant approach cuts into the lower portion of this landslide complex, which could result in a decreasing stability (or destabilizing) condition. Conversely, the alignment to the east will generally result in the placement of shallow approach fill, which would be expected to result in a neutral or increasing stability (stabilizing) condition. Additionally, the landslide deposits appear to decrease in thickness to the east. As such, it is our opinion that the east alignment option presents geotechnical advantages over the west option.

**LANDSLIDE CONSIDERATIONS – BRIDGE LENGTH**

KLJ has requested that we evaluate the potential to utilize a structure that is shorter than the existing bridge. Specifically, KLJ has indicated a preference to move the south abutment toward the river. As indicated on Figure TM-1-2, the toe of the landslide complex appears to terminate near the south pier, and the thickness of the landslide deposits decreases towards the river (see
Figure TM-1-3). This would suggest that locating the proposed abutment north of the existing abutment (shortening the bridge) may be preferable. However, moving the abutment to the north will likely require constructing an approach fill. Constructing such an approach fill would result in decreased stability for potential riverbank slope failures.

Considering stability issues and landslide mechanisms at the site, it is our opinion that the south abutment of the proposed bridge could be located no more than 100 feet north of the existing abutment location. However, if the abutment is moved north from the existing location, measures may be required to address stability of the riverbank near the bridge, as described below. Regardless of the abutment location, stability mitigation could be required to address the movement associated with the large mapped landslide complex at the site. Potential mitigation measures, for both small and large slope failure, are discussed below.

**STABILITY MITIGATION**

**Deep-Seated Landslide**

If further characterization confirms that the movement observed in inclinometer RP-126-02A is indicative of movement of the large mapped landslide near the proposed bridge, mitigation could be considered to decrease the likelihood of the movement affecting the proposed bridge. Potential mitigation measures include (in order of lowest to highest risk):

- The landslide could be avoided by locating the bridge near the mapped downstream limit of the landslide (near the confluence of Freed Creek and the Little Missouri River). Additional characterization would be necessary to better define the downstream limit of the landslide. We understand that this alternative is likely infeasible due to right-of-way, environmental, and other non-geotechnical considerations.

- Mass grading could be completed to unload the head of the landslide complex. A suitable location to waste the excavated material would be required. We understand that this alternative is likely infeasible due to right-of-way, environmental, and other non-geotechnical considerations.

- The bridge foundations could be designed to allow the landslide to flow around/between foundation elements and to resist the associated lateral loading. This would likely require the installation of drilled shafts and ground anchors through substructure or foundation elements. Based on preliminary numerical modeling, the unfactored lateral loading due to landslide movement may be on the order of 75 to 125 kips per foot of structure width, for shaft spacing ranging from 6 to 3 diameters, respectively. For preliminary design,
this load could be applied as an equivalent triangular pressure distribution increasing from zero at the ground surface to a maximum at the slip surface, elevation 1,940 feet.

**Shallow, Localized Bank Failures**

If the south abutment is moved north of the existing location, measures may be required to mitigate or accommodate the decrease in riverbank stability associated with the approach fill, depending on the final bridge location and geometry of the approach fills. The bridge foundations, approach fills, and/or associated retaining structures could be designed to address riverbank slope stability issues using one or more of the following techniques:

- Approach fills could be constructed using lightweight fill, such as slag or expanded polystyrene foam (geofoam).
- Bridge foundations could be designed to improve slope stability as laterally loaded elements. This may require utilizing larger diameter, deep-foundation elements or ground anchors installed through the substructure or foundation elements.
- Retaining structures could be supported by deep foundations.

**ADDITIONAL CONSIDERATIONS**

Continued monitoring of the existing inclinometers should be completed to better characterize landslide activity near the proposed bridge. Additionally, the project would benefit from the installation of additional inclinometers to better define the limits of landslide activity. The sooner that these additional inclinometers can be installed and monitored, the more useful they will be for final design. As such, consideration should be given to installing additional inclinometers so that data can be collected and used in the final design of the bridge and approaches. Assuming the final alignment is located within approximately 100 feet of the existing alignment, appropriate locations for the proposed inclinometers would largely be independent of the selected alignment because of the relatively large size of the mapped landslide complex. Potential locations for additional inclinometers include proposed substructure locations associated with the east alignment option, the relatively flat portion of the mapped landslide west of the existing alignment between approximately Stations 6660+00 and 6665+00, and the farmstead area to the east of the existing alignment.

Encl:
Figure TM-1-1: Site and Exploration Plan
Figure TM-1-2: Mapped Landslides and Site Observations
Figure TM-1-3: Generalized Subsurface Profile A-A’
NOTES
1) Site plan adapted from KLJ files
   Photo_North_NDDOT\Prelim_McKenzie.dgn and
   control_mckenzie.dgn.
2) Stationing of existing alignment provided in KLJ file
   control_mckenzie.dgn.
LEGEND

Mapped landslide deposit
(Murphy and Gonzalez, 2003; Murphy, 2003)

Active head scarps observed by Shannon & Wilson.

Approximate landslide limits based on field reconnaissance by Shannon & Wilson.

NOTES
1) Site plan adapted from KLJ files
   Photo_North_NDODOTPrelim_McKenzie.dgn and control_mckenzie.dgn.

2) Stationing of existing alignment provided in KLJ file control_mckenzie.dgn.

US-85 - I-94 to Watford City Bypass
Project No. 9-085(085)075, PCN 20046
McKenzie County, North Dakota

MAPPED LANDSLIDES AND SITE OBSERVATIONS
March 2017 23-1-01453-230

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants
FIG. TM-1-2
TECHNICAL MEMORANDUM TM-2

PRELIMINARY CUT AND FILL SLOPE RECOMMENDATIONS
TO: Jen Turnbow and Troy Ripplinger (KLJ)
FROM: Greg Fischer, Bill Laprade, David Vara
DATE: March 16, 2017
RE: TECHNICAL MEMORANDUM TM-2, REV. 1, PRELIMINARY CUT AND FILL SLOPE RECOMMENDATIONS, US-85 – I-94 TO WATFORD CITY BYPASS, PROJECT NO. 9-085(085)075, PCN 20046, MCKENZIE COUNTY, NORTH DAKOTA

This memorandum provides preliminary recommendations for permanent cut and fill slopes for the proposed widening of US Highway 85 (US 85). Specifically, this memorandum addresses slope stability issues for the portion of the US 85 alignment which passes through landslide-prone badlands terrain (Station 6390+00 to 6780+00). All stationing referenced herein refers to the existing alignment and stationing provided by KLJ in the file control_mckenzie.dgn. In preparing this memorandum, we considered cross-sections provided by KLJ on November 23, 2016 in the following files: Xsec_McKenzie_4LaneFlush_Realignment.dgn, Xsec_McKenzie_4LaneFlush_trail.dgn, and Xsec_McKenzie_LongX_W.dgn. We also reviewed cross-sections provided on January 6, 2017 in the file Xsec_McKenzie_4LaneFlush_170106.dgn. This document supersedes our March 17, 2016 version of this memorandum.

PRIOR LANDSLIDES

As part of our evaluation to assess appropriate slopes for cuts and fills, we reviewed North Dakota Department of Transportation (NDDOT) records documenting prior landslides and associated repairs. These records are summarized below.

Original Highway Construction

The original construction of US 85 through the badlands was documented in a 1960 report. The report describes two landslide areas, one between approximately Stations 6559+00 and 6570+00

1 North Dakota State Highway Department, 1960, Construction Report, Landslide Area, U.S. Highway 85, McKenzie County, North Dakota, Construction Department.
(referred to as the south slide), and one between approximate stations 6587+00 and 6597+00
(referred to as the north slide).

**Lower Landslides**

The report indicates that landslide activity initiated at both landslide areas after placing
fill on the downhill (east) side of the alignment (the fill consisted of material excavated to
construct the terraced slope above the roadway). The initial landslide at the north slide area (see
Figure TM-2-1) was described as having a width of about 500 feet along the alignment, an area
of about 10 acres, and sliding northeast towards Freed Creek. The head scarp of the landslide
was reportedly approximately 30 feet high and located about 150 east of the design centerline.
The construction records indicate that fill was being placed at a slope of 2H:1V (horizontal to
vertical) at the time of the failure, and that the toe of the landslide extended to Freed Creek at
approximately elevation 2,030 feet. The thickness of the fill and the initial topography prior to
construction is uncertain.

Less information is available on the initial south slide (see Figure TM-2-2). The report
indicates that the initial south slide involved an area of five acres. The report also indicates that
the landslide “close[d]” Freed Creek, suggesting sliding was occurring near the creek elevation,
approximately 2,160 feet. Aside from describing the location and area of the initial south slide,
the report contained little other substantive information on the slide.

The report indicates that at the north slide area, measures would be required to “buttress
the next slump block west that would be crossed by the highway.” The report further describes
this slump block as being “bounded by a slip plane that crosses at station 128 [approximate
current station 6593+00], inters [sic] the backslope to terrace 1 [approximate elevation 2,245
feet] then crosses the centerline at station 132 [approximate current station 6589+00].” Based on
the limited information provided in the report, characteristics of the slump block such as the slip
surface depth and geometry are uncertain.

Because of the two lower landslides and the central slump block at the north slide area,
the alignment was shifted to the west, with a horizontal change of 77 feet near the north slide and
68 feet near the south slide. To stabilize the slides, the contractor constructed large buttresses
below the roadway. Construction of the buttresses included realigning Freed Creek east of its
natural alignment. At the north slide, the upper two-thirds of the buttress was typically
constructed at slopes of 3.75H to 4H:1V, while the lower portion was apparently relatively flat.
Based on our interpretation of the available construction documents and subsurface explorations, the north buttress is estimated to have a maximum thickness of about 65 feet. The buttress at the south slide was constructed at slopes of about 3H:1V and is estimated to have a maximum thickness of about 45 feet.

Based on our recent field observations, the fill slopes appear to be stable. However, the slopes exhibit erosion features including rilling, gullyng, internal erosion, and sink holes. The density of vegetation on the fill varies with location.

**Upper Landslides**

The construction report documents several “slump blocks” that developed while excavating for the terraced back slope near the north slide (Stations 6581+00 and 6591+00) and the south slide (Station 6559+00). The slump blocks reportedly displaced on a “nearly horizontal” slip plane at about elevation 2,360 feet, which was described as “wet blue slick-sided clay” and “blue-green sandy clay above lignite coal.” The elevation of the slip plane was also reported to correspond to the “natural or geologic terrace that was eroded across slump blocks.”

The report provides limited information describing the final cut slope configuration above elevation 2,360 feet. However, in the vicinity of the south slide, it appears that the contractor removed the slump blocks (slide debris) and utilized cut slopes of 1.3H:1V without benches above the roadway, based on a cross-section near station 6565+00 that was included with the report. Based on a photo of the completed project included in the construction report, near the north slide it appears that the contractor removed the slide debris by excavating a near-horizontal bench near elevation 2,360 feet that extended into the hillside until encountering intact bedrock, where the bench was blended into the natural slope. Below elevation 2,360 feet, the final cut slope generally consisted of 40-foot high, 1H:1V cuts with 15-foot wide benches.

Our recent field observations indicate that the 1.3H:1V cut slopes excavated above elevation 2,360 feet are unstable. Although the cuts below elevation 2,360 feet appear to be stable, the cuts have experienced significant erosion. Additional information regarding the performance of the cut slopes is described in the section Cut and Fill Slope Recommendations.
Other Information

The report indicates that “springs issue from sand above lignite in the backslope station 121 to 126 [current station 6595+00 to 6599+00]. The present flow may be classed as seepage and this is of minor concern to date.” The report also indicates that the “spring area station 121 to 126 was subcut and backfilled with blue sandy clay. The total zone subcut and reconstructed extends from station 120 to 138 [current Station 6582+00 to 6600+00].”

1999 Slide Repair

We reviewed plans for Project SNH-7-085(036)124, prepared by NDDOT\textsuperscript{2}. The plans present earthwork and grading improvements to mitigate a slope failure and slope erosion between approximately Stations 6570+00 and 6574+00 (see Figure TM-2-2). As indicated by Figure TM-2-2, this landslide appears to be beyond the limits of the landslides that occurred during original highway construction.

The slope failure in this area occurred on the downhill (east) side of the roadway. The head scarp, which was approximately 15-feet high, appears to have been located about 35 feet east of the edge of pavement. The plans indicate that the slope failure was about 125-feet wide and toed-out approximately 250 feet east of the edge of the pavement (approximately elevation 2,220 feet). Prior to the failure, the ground was typically sloped at about 2H:1V. No subsurface explorations were completed in the slide mass.

The repair for the slide consisted of excavating the slide debris and in its place, rebuilding a geosynthetic-reinforced 2H:1V slope. The project also included surface and subsurface drainage improvements and erosion control measures.

Our recent field observations indicate that the slide repair appears to be stable and well vegetated. However, signs of internal erosion and instability were observed in the ground adjacent and immediately below the repair.

\textsuperscript{2} NDDOT, 1999, SNH-7-085(035)124 Highway 84 Slide Repair and Erosion Control, design plan set, March 8.
2002 Slide Repair

We reviewed plans for Project NH-7-085(044)123, prepared by NDDOT\(^3\). The project included grading and earthwork to repair an approximately 500-foot wide landslide between Stations 6543+50 and 6548+50 (see Figure TM-2-3).

The slope failure apparently occurred on the west side of the roadway, in a 2H:1V fill slope with heights ranging from about 30 to 65 feet. The lower portions of the slope were on the order of 3H:1V, but it is unclear if the flatter portions of the slope were native or fill materials. The base of the failure apparently dipped to the north, from elevation 2,375 feet at the south end of the failure to elevation 2,320 near the north end of the failure. Given that the Sentinel Butte Formation has essentially nil dip, the variability in the elevation of the base of the failure suggests that the failure was confined to fill materials. No subsurface explorations were completed in the slide mass.

The repair for the slope failure consisted of excavating the slide mass, and in its place rebuilding a geosynthetic-reinforced 3.5H:1V slope. The project also included widening and realigning the roadway, which required additional cuts and fills. Between Stations 6506+00 and 6517+00, the existing 2H:1V fill slopes on the east side of the roadway were reportedly flattened to 3.5H:1V. Between Stations 6528+00 and 6533+00, the roadway was widened to the east with 3.5H:1V fill slopes. From Stations 6538+00 to 6546+00 and Stations 6552+00 to 6557+00, existing cuts on the east and west side of the roadway, respectively, were enlarged with widened cuts up to 145-feet high, which were excavated at slopes of 1.1H:1V to 1.5H:1V (average slope of about 1.4H:1V), depending on the location. Additionally, the project included surface and subsurface drainage improvements and erosion control measures.

During our recent field reconnaissance, we did not observe any indications of instability in the cuts and fills completed as part of the 2002 repair. However, both the cut and fill slopes exhibit erosion, including rilling, gullies, and signs of internal erosion. Significant erosion is present on the 3.5H:1V fill slope between stations 6528+00 and 6533+00.

\(^3\) NDDOT, 2002, Project NH-7-085(044)123, Grading, Slide Repair, Aggregate Base Course, Hot Bituminous Pavement, Internal Pipe Joint Seals and Incidentals, design plan set, July 26.
2011 Repair

We reviewed plans for Project SER-7(063)127, prepared by NDDOT\(^4\). The project included grading, earthwork, and erosion control improvements between approximately Stations 6717+00 and 6767+00 to repair distress associated with slope movement. Between approximately Stations 6725+00 and 6750+00, the roadway alignment was shifted a maximum of 120 feet to the east. The location of the project is shown in Figure TM-2-4.

Based on Google Earth Pro\(^5\) imagery from 2011, there were two areas of pavement patches, which are suggestive of landslide movement, in the prior alignment: one offset about 100 feet west of the current alignment between Stations 6736+00 and 6738+00 (referred to as the south area) and one offset about 100 feet west of the current alignment between Stations 6741+00 and 6743+00 (referred to as the north area). (In the vicinity of this area, we understand that the alignment was originally located further to the east, in an area referred to as Horseshoe Bend. Chronic landslide activity in Horseshoe Bend resulted in a substantial realignment in the 1980s of the roadway to near its current location.)

The plans provide relatively limited characterization of the landslide. However, at the north area of distress, imagery from 2011\(^5\) show a visible head scarp adjacent to the west shoulder of the roadway. Based on our interpretation of the imagery, the landslide at this location likely involved both embankment fill and native materials. At the southern area of distress, a pavement patch is visible going back as far as 1995 (the oldest Google Earth Pro imagery for this area), which suggests slope movement was occurring in this area for many years prior to the 2011 repair. However, a distinct head scarp is not visible at the southern area of distress in any of the available Google Earth Pro imagery. Nevertheless, in our opinion it is likely that both areas of distress are associated with the same landslide. Prior to failure, the embankment in this area was at a 2H:1V slope.

The repair of the landslide consisted of excavating the slumped embankment material and shifting the roadway alignment a maximum of about 115 feet to the east. The new alignment was constructed on 3H:1V fill slopes that were benched into the existing embankments. The construction also included erosion control and surface drainage improvements.

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\(^4\) NDDOT, 2011, Project SER-7-085(063)127, Slide Repair, Base, HBP, design plan set, July 15.

In 2011, NDDOT installed an inclinometer near the toe of the embankment slope, on the west side of the failure area (approximately Station 6740+00, 220 feet left) to monitor slope movement. NDDOT completed periodic readings of the inclinometer until it apparently sheared to failure at a depth of about 70 feet in October 2015. The data obtained by NDDOT indicate that ground movement has been ongoing since the repairs were completed. The inclinometer, which has top and bottom elevations of approximately 2,190 and 2,108 feet, respectively, indicates slip zones at the following approximate elevations (it is possible that landslide movement is occurring below the bottom of the inclinometer):

- El 2,168 feet,
- El. 2,160 feet,
- El. 2,128 feet, and
- El. 2,120 feet.

In addition to the movement indicated by the inclinometer, a pavement patch is visible between Stations 6736+00 and 6738+00. We also observed relatively minor rilling and erosion on the surface of the recently constructed embankments. A failure in the cut slope between Stations 6734+00 and 6736+00, which was excavated as part of the 2011 project, is also visible. Recent landslide activity, including debris flows, is visible in the slope above the roadway between Stations 6742+00 and 6748+00.

**SUBSURFACE CHARACTERIZATION AND WEAK LAYERS**

Landslide activity in sedimentary bedrock, such as the Sentinel Butte Formation found along the project alignment, is often controlled by near-horizontal seams of comparatively weaker material. For example, multiple landslides were reported to have occurred during the original construction on a weak layer near elevation 2,360 feet.

To identify potential weak layers, we compared laboratory test results to our field observations and documentation of past failures. Specifically, we utilized the liquid limit (LL), an index property that can be correlated with soil/rock strength (higher LL is indicative of lower strength). Figure TM-2-5 summarizes the statistical distribution of LL values obtained from our laboratory testing efforts. Based on data for the weak layer identified near elevation 2,360 feet during the original construction, we assumed that LLs greater than 120 (which correspond to the 95th percentile for the samples we tested, indicating LLs near the maximum for all samples tested) might correlate with weaker layers.
In Figure TM-2-6, we plotted the LL versus elevation adjacent to a generalized stratigraphic column we developed for the alignment. LL values greater than 120 are plotted in red. We then compared this plot to our field observations and past performance of slopes along the alignment to identify potential weak layers (high plasticity zones), which are summarized below. In addition to considering LL, we also considered the color of the bedrock. As previously discussed, the weak layer at elevation 2,360 feet was described in the original construction records as green in color. Similarly, we observed that this layer and other high LL layers tended to be greenish gray, while comparatively lower LL layers were gray (see Figure TM-2-6). To account for potential variability and uncertainty in the locations of the high plasticity zones, we have specified elevation ranges for the zones, as indicated below.

- **Zone A – El. 2,340 to 2,400:** This zone is associated with the previously described failures that occurred at approximately elevation 2,360 feet while excavating cut slopes during the original highway construction. This zone may also correspond to landslide activity occurring at the natural slope immediately north of the Horseshoe Bend area of the alignment (approximately Station 6742+00 to 6748+00).

- **Zone B – El. 2,160 to El. 2,180:** This zone is similar to the creek elevation at the base of the lower south slide (approximately Station 6559+00 to 6570+00) that occurred during the original construction of the highway. Therefore, it is possible that the lower south slide occurred in this zone. This zone also corresponds to a shear zone indicated by the NDDOT inclinometer installed near the 2011 landslide.

- **Zone C – El. 1,980 to El. 2,070:** This zone includes the elevation (2,030 feet) reported for the base of the lower north slide (approximately Station 6584+00 to 6597+00) that occurred during the original construction of the highway. Therefore, it is possible that the lower north slide occurred in this zone. This zone also appears to correlate with the base of two large ancient landslides, one observed between Stations 6612+00 and 6627+00 and the other between Stations 6636+00 and 6647+00.

- **Zone D – 1,900 to 1,940:** This zone includes the slip surface elevation (1,940 feet) of a relatively large landslide near the south bank of the Little Missouri River (see Technical Memorandum TM-1), as well as several smaller failures in the south bank of the river in the vicinity of the Long X Bridge. The elevation of the river channel at the Long X Bridge is similar to this elevation range.

Based on our field observations, we identified several slope failures that do not correspond to the layers described above. As examples, these include relatively large landslide bowls on the west side of the roadway between Stations 6600+00 and 6611+00 and the cut slope failure on the east side of the roadway between Stations 6734+00 and 6736+00.
SLOPE STABILITY ANALYSES AND FACTORS OF SAFETY

To assess appropriate cut and fill slope angles, we completed slope stability analyses for selected representative slope cross-sections. We focused our initial stability analyses at locations where landslides have previously occurred and back-analysis could be completed to calibrate the stability model. Details of these analyses are provided in Technical Memorandum TM-4.

Based on AASHTO\textsuperscript{6} requirements, slopes that do not support structural elements (e.g. bridge substructures) are typically designed for a resistance factor (RF) of 0.75. The resistance factor is essentially the inverse of the factor of safety (FS) for slope stability. As such, a FS of 1.3 is typically used. For slopes that support structural elements, AASHTO indicates a RF of 0.67, which is equivalent to a FS of 1.5 (a higher FS may also be used when there is greater uncertainty in the slope stability model used for design).

The FS values indicated by AASHTO are typically applied to a slope that has not yet been constructed or to an existing slope that has not failed. For a large landslide mass that is undergoing movement or has failed, it is often cost prohibitive or even infeasible to mitigate the slope to an FS of 1.3 or 1.5. Therefore, some agencies and entities consider lower FS values in such instances.

The NDDOT has indicated a preference to design cut and fill slopes constructed for the project to an FS of 1.3; however, they have also indicated a desire to select design FS values based on the performance of the existing slopes. Specifically, NDDOT has indicated that FS values less than 1.3 are acceptable in areas where the existing slopes have performed satisfactorily and designing to an FS of 1.3 would have significant cost and constructability ramifications, as discussed in the following section.

CUT AND FILL SLOPE ANGLE RECOMMENDATIONS

Our recommended slope angles are summarized in Table TM-2-1. We developed our preliminary recommendations for cut and fill slopes for the proposed widening of US 85 based on:

- Review of previous landslides and repairs,
- Field reconnaissance we completed from 2014 through 2016,

\textsuperscript{6} American Association of State Highway and Transportation Official (AASHTO), 2016, AASHTO LRFD Bridge Design Specifications: customary U.S. units (7th ed. with 2016 Interim Changes), Washington, D.C.
In general, we anticipate that 2H:1V cut slopes and 3H:1V fill slopes will provide adequate slope stability (FS of at least 1.3) for most of the project. However, in some areas of the alignment, as indicated in Table TM-2-1, flatter slopes or other measures will be required to provide adequate slope stability. Additionally, as discussed with NDDOT, consideration was given to designing some slope areas with FS values less than 1.3, based on the performance of the existing slopes. Areas deviating from the typical cut slopes of 2H:1V and fill slopes of 3H:1V and areas where FSs less than 1.3 may be considered are discussed below.

The recommendations provided below were developed for proposed cut and fill slopes. Except where noted in the following sections, these recommendations do not address stability of existing landslides outside of the anticipated grading limits based on the cross-sections provided by KLJ. Additionally, these recommendations do not address stability of the relatively large landslide mapped near the south bank of the Little Missouri River (see Technical Memorandum TM-1).

**Station 6436+00 to 6440+00**

The existing 2H:1V fill slope on the west side of the roadway in this reach is exhibiting instability and minor erosion. Therefore, we recommend utilizing flatter fill slopes to buttress the existing fill. In our opinion, 4H:1V fill slopes are appropriate for this reach.

**Station 6506+00 to 6552+00**

We anticipate that 3H:1V fill slopes and 2H:1V cut slopes will generally provide adequate stability. However, if it is necessary to place fill on the active landslide between approximately Stations 6547+00 and 6550+00 (see Figure TM-2-4), flatter slopes or other measures to improve slope stability will be required. Based on cross-sections provided by KLJ, we do not anticipate that fills will extend onto the active landslide.

**Stations 6557+00 to 6612+00**

This reach corresponds to the locations of the slope failures that occurred during the initial construction of the highway. Some of the largest cuts and fills for the proposed construction will
be located in this reach. This reach presents the most challenging slope stability issues for the proposed construction, in our opinion.

**Cut Slopes**

The weak layer encountered near elevation 2,360 feet will have a substantial effect on stability of the proposed cut slopes in this reach. Several slope failures have occurred in the existing cuts above elevation 2,360 feet, which were initially cut at about 1.3H:1V. These failures above elevation 2,360 feet tend to toe-out above the underlying section of the slope, resulting in landslide debris cascading down the lower portion of the slope. NDDOT has indicated that these debris flows have been an ongoing maintenance issue. As such, NDDOT has indicated a preference to remove landslide debris and to flatten the slope to provide a FS of 1.3, regardless of the need to excavate the slope to accommodate the selected roadway alignment.

Below the existing terrace near elevation 2,360 feet, the previously constructed benched cut slopes have eroded and sloughed to varying degrees, although the slopes generally appear to be stable. In some locations, the benches are no longer distinguishable based on comparison to photographs of the original construction¹, and the slope is nearly continuous at an average slope of approximately 1H:1V to 1.3H:1V. Numerous large gullies and signs of internal erosion are also present on the face of the cut slope. The erosion appears to be most severe on east-facing slopes, while the north-facing slopes have performed better. Because the slopes have performed adequately, excluding erosion, NDDOT has indicated that existing slopes could remain in their current configuration, regardless of the calculated FS. As such, KLJ has proposed an alignment that does not require excavation of the existing cut slope below elevation 2,360 feet.

To provide a FS of 1.3 for the slope above elevation 2,360 feet, we recommend using the excavation configuration shown in Figure TM-2-7, which includes a 100-foot wide bench at elevation 2,340 feet. The purpose of the proposed bench is to remove landslide debris and material that has been disturbed by prior landslide movement, such that the proposed cut will be located in comparatively more competent, stable bedrock. Additionally, the proposed bench will provide access for installation of horizontal drains or other features to improve drainage above the weak layer at elevation 2,360 feet (see section Additional Considerations).

Based on the previously described discussions with NDDOT and KLJ, the existing cut slope below the proposed bench at elevation 2,340 feet may remain in its existing condition. However, we recommend implementing drainage improvements to reduce the severity of erosion
on the slope. Additionally, any landslide debris on the slope face below elevation 2,340 should be removed.

**Fill Slopes**

Fill slopes steeper than approximately 2H:1V have experienced significant stability issues in this reach. These stability issues were mitigated by constructing buttresses with slopes typically between 3H:1V and 4H:1V. In locations where stability issues have not occurred, the fill slopes are generally sloped at about 3.5H:1V or flatter.

We recommend utilizing fill slopes of 3.5H:1V in this reach. To waste material generated from cuts, we recommend additional slope flattening where feasible, particularly at locations where failures previously occurred (Stations 6559+00 to 6574+00 and 6587+00 to 6597+00). At locations where 3.5H:1V slopes are not feasible, we recommend utilizing walls instead of a steeper slope. The use of a slope steeper than 3.5H:1V would result in a decrease in stability compared to existing conditions. However, the use of a wall would require placing less fill, which would have a smaller effect on slope stability. In some locations, it may be necessary to construct walls with lightweight fill (e.g. geofoam) to maintain a minimum FS of 1.3. During final design, the stability of walls and the need to utilize lightweight fill should be further evaluated.

At the location of the 1999 landslide repair (between Stations 6570+00 and 6574+00), where instability has been observed below the repair, an option more robust than lightweight fill may be required to prevent uphill propagation of the failure such that it affects the proposed alignment. This could be accomplished by constructing a cantilevered or tied-back soldier pile and lagging wall. The lower portion of the slope could also be flattened to improve stability. Ground anchors could also be considered to improve stability of the landslide debris below the 1999 landslide repair.

**Stations 6612+00 to 6656+00**

This reach traverses several large ancient landslides. Although the majority of cuts and fills in this reach will be relatively small, a few larger cuts in landslide debris may be required. To reduce the likelihood of local instability and re-activating ancient landslides, we recommend utilizing 3H:1V cuts in this reach. Based on cross-sections provided by KLJ, we anticipate that 3H:1V fill slopes will provide adequate stability in this reach.
Stations 6735+00 to 6736+00

A relatively small failure has occurred in a cut slope on the east side of the roadway in this reach. We recommend flattening the failure to 4H:1V.

Stations 6736+00 to 6745+00

Because of chronic instability of fill slopes in this area, we recommend utilizing 4H:1V fill slopes on the west side of the roadway. On the eastside of the roadway, where fill slopes would be oriented in the opposite direction relative to the landslide movement, we recommend 3H:1V fill slopes. Based on discussions with KLJ, we understand that cut slopes are not proposed in this reach.

Stations 6745+00 to 6750+00

Cut Slopes

This reach includes an active slope failure in the natural slope above the roadway. As previously discussed, this failure appears to be associated with a weak layer near elevation 2,360 feet. Based on discussions with NDDOT, we understand that this failure is resulting in ongoing maintenance issues. As such, NDDOT prefers to remove debris associated with the landslide and flatten the slope to provide an FS of 1.3 for the slope above elevation 2,360 feet, regardless of the need to excavate the slope to accommodate the selected roadway alignment.

Below elevation 2,360 feet, the existing slopes above the roadway appear to be stable, although a portion of the slope may be mantled by a thin layer of landslide debris associated with overlying slope instability. Because the slopes have performed adequately, NDDOT has indicated that existing slopes could remain in their current configuration, regardless of the calculated FS. As such, KLJ has proposed an alignment that does not require excavation of the existing cut slope below elevation 2,360 feet.

To provide a FS of 1.3 for the slope above elevation 2,360 feet, we recommend using the excavation configuration shown in Figure TM-2-8, which includes a 100-foot wide bench at elevation 2,340 feet. The purpose of the proposed bench is to remove landslide debris and material that has been disturbed by prior landslide movement, such that the proposed cut will be located in comparatively more competent, stable bedrock. Additionally, the proposed bench will
provide access for installation of horizontal drains or other features to improve drainage above the weak layer at elevation 2,360 feet (see section Additional Considerations).

Based on the previously described discussions with NDDOT and KLJ, the existing slope below elevation 2,360 feet may remain in its existing condition. However, we recommend removing any landslide debris on the slope and implementing drainage measures to reduce erosion on the slope.

**Fill Slopes**

As previously discussed, instability of the fill slope in this reach has resulted in roadway distress. In 2011, a landslide occurred in this area, resulting in the realignment of the road and regrading the embankment to 3H:1V slopes. Since the repair, ground movement has apparently continued to occur based on inclinometer readings and roadway distress in the area.

Because of the large size of the adjacent landslide, which is mapped extending south to approximately Station 6694+00, it will be challenging to increase the stability of the landslide to an FS of 1.3. Further, we anticipate it will be challenging to achieve even a small increase in the FS that could stop landslide movement in this area. Stabilization of the landslide to achieve a FS of 1.3 would require either significant grading or significant structural improvements. The grading option would require construction of a buttress within the boundaries of the adjacent Theodore Roosevelt National Park (the Park). We anticipate that the FS could be increased to 1.3 with a major grading effort. While we recognize the challenges of working in the Park, if this could be negotiated, it would also allow a wasting area for the excess cut material (i.e. provide dual benefits). Similarly, the optimum location for the latter (structural) option would also be inside the Park boundaries, although disturbance from this option would be significantly less compared to the grading option. It may be feasible to locate a structural option near the edge of the right-of-way, but it would be a substantial structure and likely would only be feasible to achieve a small (less than 10 percent) improvement in the FS if placed at this boundary. A greater improvement could be achieved by locating the structure inside the Park boundaries, but the resulting FS would still likely be less than 1.3.

Considering the ongoing slope movement and large landslide complex mapped in this area, we recommend 4H:1V fill slopes on the west side of the roadway and 3H:1V fill slopes on
the east side of the roadway in this reach. Based on preliminary cross-sections prepared by KLJ, we understand that relatively minor fills will be required through this reach. As such, we anticipate that the proposed fill slopes will have a nominal effect on stability of the existing landslide. However, slope movement will likely continue at a rate similar to existing conditions.

**Stations 6750+00 to 6780+00**

This reach marks the transition from the Little Missouri River Valley and badlands terrain to relatively flat upland terrain. We understand that several of the existing cuts may need to be widened through this reach to accommodate the proposed alignment. The existing cuts are generally sloped at about 2.5H:1V and have been performing satisfactorily. Therefore, we recommend sloping the proposed cuts at 2.5H:1V.

For fills, we recommend slopes of 4H:1V on the west side of the roadway (see previous discussion for Stations 6745+00 to 6750+00) and 3H:1V on the east side of the roadway.

**ADDITIONAL CONSIDERATIONS**

**Drainage**

Proper surface and subsurface drainage will be critical to slope stability and reducing the potential for erosion (see below). Key drainage recommendations are summarized below:

- Grading and drainage should be designed to reduce the flow of water on the faces of slopes. This should include the use of ditches to intercept surface water above slopes.
- Where benched slopes are utilized, the benches should be graded to direct water to a ditch on the inboard side of the bench.
- In several locations, fills for the existing alignment have blocked natural drainage paths resulting in water being impounded behind embankments (Stations 6549+00, 6609+00, and 6625+00). Ponded water should be drained and drainage measures (e.g., inlets and storm drains) should be installed or repaired as necessary to prevent water from becoming impounded again. Because of the existing topography, the installation of such drainage measures may require the use of trenchless techniques.
- In locations where slope stability will be controlled by weak layers, we recommend installing subsurface drainage features to relieve potential pore water pressures and to improve slope stability. Appropriate drainage features could include horizontal drains, springhead drains, and trench drains excavated parallel or perpendicular to a...
given slope. Subsurface drainage features should also be installed at locations where seepage is observed during earthwork activities.

Erosion

Many cut and fill slopes, as well as natural slopes, along the alignment exhibit substantial erosion, including rilling, gullying, and signs of internal erosion. To assess the erosion potential of soil/rock along the alignment, we completed pin hole dispersion and sodium absorption ratio testing of selected soil samples. The laboratory test results are generally indicative of dispersive/erodible soils, i.e. materials that are substantially more susceptible to erosion than typical earth materials. In general, erosion is considered a maintenance issue. However, severe erosion, if not repaired, can lead to slope instability.

The drainage measures discussed above will be important to reduce the likelihood and severity of potential erosion. Existing erosion control features, such as straw wattles and erosion control blankets, have achieved varied success, based on our observations. Additional measures to mitigate erosion of dispersive soils and reduce the need for future maintenance are summarized below:

- Slopes higher than 50 feet should be benched. For preliminary design, we recommend assuming 15-foot wide benches with a horizontal spacing of 50 feet between adjacent benches. The slope of the steps between adjacent benches should be determined to provide an overall slope equal to the values recommended herein.
- Flatten slopes as much as feasible.
- Dispersive soils tend to have poor natural fertility, making it difficult to establish vegetation. Blending and placement of topsoil and fertilizer on the surface of slopes can support the establishment and growth of vegetation.
- Surficial soils on fill slopes could be chemically stabilized by blending materials such as cement, gypsum, or fly ash.
- Ditches and similar drainage features could be lined with graded riprap or concrete for erosion protection. Additionally, non-dispersive soils should be used to backfill around such features to prevent undermining.

FINAL DESIGN

The recommendations presented herein are preliminary. Further subsurface characterization, monitoring of instrumentation, and slope stability analysis will be required to determine final
recommendations for the proposed cut and fill slopes. We anticipate that this work will be completed during the design phase of the project.

Encl:
Table TM-2-1: Recommended Cut and Fill Slope Angles by Station
Figure TM-2-1: Original Construction North Slide Repair
Figure TM-2-2: Original Construction South Slide Repair and 1999 Slide Repair
Figure TM-2-3: 2002 Slide Repair
Figure TM-2-4: 2011 Slide Repair
Figure TM-2-5: Liquid Limit Histogram and Distribution
Figure TM-2-6: Generalized Stratigraphic Column and Atterberg Limits
Figure TM-2-7: Permanent Cut Slope, Typical Section Sta. 6557+00 to 6612+00
Figure TM-2-8: Permanent Cut Slope, Typical Section Sta. 6745+00 to 6750+00
### TABLE TM-2-1
RECOMMENDED CUT AND FILL SLOPE ANGLES BY STATION

<table>
<thead>
<tr>
<th>Station Range</th>
<th>Recommended Slope Angle</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Begin</td>
<td>End</td>
<td>Cuts</td>
</tr>
<tr>
<td>6390+00</td>
<td>6436+00</td>
<td>2H:1V</td>
</tr>
<tr>
<td>6436+00</td>
<td>6440+00</td>
<td>2H:1V</td>
</tr>
<tr>
<td>6440+00</td>
<td>6506+00</td>
<td>2H:1V</td>
</tr>
</tbody>
</table>
| 6506+00       | 6552+00                 | 2H:1V   | 3H:1V     | 1) 3.5H:1V fill slopes between approximately Stations 6528+00 and 6538+00 have experienced significant erosion.  
2) If fill is placed on active landslide on east side of roadway between approximately Station 6547+00 and 6550+00, flatter fill slopes and additional measures will likely be required. |
| 6552+00       | 6557+00                 | 2H:1V   | NA (Fills not Proposed) | - |
| 6557+00       | 6612+00                 | -       | 3.5H:1V   | 1) See Figure TM-2-7 for recommended cut slope configuration.  
2) Flatten slopes as much as possible to waste excavated material. |
| 6612+00       | 6656+00                 | 3H:1V   | 3H:1V     | -          |
| 6656+00       | 6735+00                 | 2H:1V   | 3H:1V     | -          |
| 6735+00       | 6736+00                 | 4H:1V   | 3H:1V     | Flatten slope failure on east side of roadway to 4H:1V. |
| 6736+00       | 6745+00                 | NA (Cuts not proposed) | 3H:1V (east side of roadway)  
4H:1V (west side of roadway) |
| 6745+00       | 6750+00                 | -       | 3H:1V (east side of roadway)  
4H:1V (west side of roadway) | See Figure TM-2-8 for recommended cut slope configuration for slope failure adjacent to roadway. |
| 6750+00       | 6780+00                 | 2.5H:1V | 3H:1V (east side of roadway)  
4H:1V (west side of roadway) | Bench slope where slope height exceeds 50 feet. |

**NOTES:**
1) Slope recommendations are preliminary. Further subsurface investigation, monitoring of instrumentation, and analyses may result changes to the above slope angles.
2) See memorandum for additional slope stability considerations.
### TABLE TM-2-1
RECOMMENDED CUT AND FILL SLOPE ANGLES BY STATION

<table>
<thead>
<tr>
<th>Station Range</th>
<th>Recommended Slope Angle</th>
<th>Comment</th>
</tr>
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<tbody>
<tr>
<td>Begin</td>
<td>End</td>
<td>Cuts</td>
</tr>
<tr>
<td>6390+00</td>
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<td>6750+00</td>
<td>6780+00</td>
<td>2.5H:1V</td>
</tr>
</tbody>
</table>

### NOTES:
1) Slope recommendations are preliminary. Further subsurface investigation, monitoring of instrumentation, and analyses may result changes to the above slope angles.
2) See memorandum for additional slope stability considerations.
NOTES

1) Site plan adapted from KLJ files.
   Photo_North_WDOTPreim_McKenzie.dgn and
   control_mckenzie.dgn.

2) Stationing of existing alignment provided in KLJ file
   control_mckenzie.dgn.
NOTES:

1) Data shown above are for all bedrock samples tested by Shannon & Wilson.

LIQUID LIMIT
HISTOGRAM AND DISTRIBUTION

US-85 - I-94 to Watford City Bypass
Project No. 9-085(085)075, PCN 20046
McKenzie County, North Dakota

March 2017

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. TM-2-5
NOTES

1) Liquid limits greater than 120 percent are shown as red on the atterberg limit plot.

2) Zones A through D refer to high liquid limit zones within the stratigraphy (high occurrence of LL greater than 120%)

FIG. TM-2-6

Sentinel Butte Formation (Tertiary)

<table>
<thead>
<tr>
<th>Elevation (ft)</th>
<th>Lithology</th>
<th>Plastic Limit and Liquid Limit (%)</th>
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<tr>
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<td>0</td>
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<tr>
<td>2540</td>
<td>Green-Gray Claystone</td>
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<tr>
<td>2500</td>
<td>Coal A</td>
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<tr>
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<tr>
<td>1900</td>
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US-85 - I-94 to Watford City Bypass
Project No. 9-085(085)075, PCN 20046
Billings, McKenzie, and Stark Counties, ND

March 2017
SHANNON & WILSON, INC.

GENERALIZED STRATIGRAPHIC COLUMN WITH ATTERBERG LIMITS

FIG. TM-2-6
NOTES

1) Slope benches should be graded such that water does not flow down the face of the slope, which could exacerbate erosion and slope instability.

2) Benches not required for slopes less than 50-feet high.

3) Remove any landslide debris on slope below elevation 2,340 feet.

4) Excavation not required for natural bowls near Sta. 6600+00 and between Sta. 6505+00 and 6612+00
NOTES

1) Slope benches should be graded such that water does not flow down the face of the slope, which could exacerbate erosion and slope instability.

2) Benches not required for slopes less than 50-feet high.

3) Remove any landslide debris on slope below elevation 2,340 feet.
TECHNICAL MEMORANDUM TM-3

PRELIMINARY FOUNDATION CONSIDERATIONS, LONG X BRIDGE
This memorandum discusses preliminary foundation recommendations for the proposed Long X Bridge improvements on US Highway 85 (US-85) over the Little Missouri River. This document supersedes our June 24, 2016 version of this memorandum.

SUBSURFACE EXPLORATIONS

We completed seven explorations (designated RP 126-02 through RP 126-08) near the location of the Long X Bridge. The locations of the borings are shown in Figure TM-3-1 and a generalized subsurface profile is shown as Figure TM-3-2.

Subsurface conditions encountered in the explorations generally consisted of a sequence of alluvium and landslide debris over Sentinel Butte Formation bedrock. A relatively large landslide complex along the south bank of the Little Missouri River was observed in the field and LiDAR hill-shade images and confirmed in our borings (see Technical Memorandum TM-1). The landslide deposits were also encountered in our borings adjacent to the south abutment and pier. Based on our borings, the landslide deposits likely terminate near the north pier of the existing bridge, but, as discussed below, landslide debris beneath the river channel are not likely currently moving. On the north side of the bridge, only alluvial deposits are mapped near the existing structure and were encountered in our explorations. Bedrock underlying the site is mapped as the Sentinel Butte Formation, which is described as “alternating beds of grayish brown to gray sandstone, siltstone, mudstone, claystone, and lignite.”
Based on geologic origin and geotechnical engineering properties, we divided the subsurface profile into the geologic units described below:

- **Qls1 (landslide debris):** This unit typically consisted of stiff to hard, fat clay (which frequently appears to be disturbed bedrock) with varying sand content. This unit includes material that is actively moving (see Technical Memorandum TM-1).

- **Qls2 (landslide debris):** This unit typically consisted of stiff to hard, lean to fat clay (which frequently appears to be disturbed bedrock) with varying sand content. This unit likely includes landslide debris associated with ancient landslide activity that flowed into an ancient river channel. It is unlikely that this material is actively moving, based on the location of this material in the river channel and the depth of presently moving ground.

- **Qal1 (alluvium):** This unit typically consisted of loose to dense, silty and clayey sand with interbedded medium stiff to very stiff silt with sand and lean clay with sand.

- **Qal2 (alluvium):** This unit typically consisted of stiff to hard, sandy lean clay interbedded with layers of medium dense, silty sand.

- **Qal3 (alluvium):** This unit consisted of medium dense to very dense sand with varying percentages of clay and silt along with isolated layers of hard fat clay.

- **Tsb (Sentinel Butte Formation bedrock):** The bedrock consisted of claystone, sandstone, and thin layers of coal. When classified as a soil, the claystone consisted of very stiff to hard fat clay and the sandstone consisted of very dense, clayey sand.

**EXISTING BRIDGE AND FOUNDATIONS**

The location of the existing Long X Bridge is shown in Figure TM-3-1. The bridge is a two-lane, 950-foot long, three-span truss structure constructed circa 1960. Plans\(^1\) provided by the NDDOT provided limited information on foundations used to support the bridge, as discussed below.

The plans required the installation of two test piles (one 14x73 H-pile and one 12 ¾-inch diameter steel-encased pile [concrete-filled closed-end pipe pile]) at each pier. The plans further indicated that the production pile type for each pier would be selected by the Engineer. Hand

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written notes indicating pile types were included on the plans. We assume that these notes indicate the pile types selected during construction.

Based on the annotated bridge plans, the abutments were supported by battered 12x53 steel H-piles. The north pier was supported by 12 ¾-inch diameter steel encased piling with 0.31-inch wall thickness. The south pier was apparently supported by 14x73 H-piles. The piles at the north pier were apparently 35-feet long with an approximate tip elevation of 1,879 feet. The lengths of the piles at the other pier/abutments are unknown. The design loads for the abutment and pier piles were 94 and 130 kips, respectively.

**DRIVEN PILES**

We anticipate that driven H-piles, steel encased concrete (SEC) piles (also known as closed-end pipe piles), and open-end pipe (OEP) piles are all feasible foundation options for the Long X Bridge. We understand that H-piles are routinely used on NDDOT projects. We are not aware of any NDDOT projects that have utilized OEP piles. Although not used recently, NDDOT has indicated that SEC piles have been used on prior projects.

Plots of estimated factored axial resistance versus pile penetration are provided for subsurface profiles corresponding to each boring completed adjacent to the Long X Bridge in Figures TM-3-3 through TM-3-8. The figures provide charts for several different H- and pipe pile sizes, as requested by KLJ. The geotechnical factored axial resistance values presented in the figures were estimated using static analysis methods and include a resistance factor of 0.45 in accordance with AASHTO\(^2\). The factored axial resistances have been limited to a value corresponding to 0.33 times the steel yield strength in accordance with the NDDOT Design Manual\(^3\).

Although a resistance factor of 0.65 may be applied to the nominal resistance determined using dynamic testing during pile installation (additional design and installation considerations are discussed below), in accordance with AASHTO Section 10.7.3.3, we recommend estimating pile lengths using the static-estimated resistance and the associated resistance factor (0.45 in this

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\(^3\) North Dakota Department of Transportation, 2013 Standard Specifications for Highway Bridges Design Manual, accessed at: [https://www.dot.nd.gov/manuals/design/designmanual/designmanual.htm](https://www.dot.nd.gov/manuals/design/designmanual/designmanual.htm)
The static-based resistance factor accounts for variability and uncertainty in the calculated and actual pile resistance. The 0.65 resistance factor is intended to be applied to the nominal resistance value determined using dynamic testing, and the 0.65 value reflects the variability between the dynamic-estimated resistance and the actual resistance.

**H-Piles**

H-piles are typically designed for a combination of side resistance and end bearing, or as end bearing piles when driven to a suitable bearing layer. Because of their relatively low displacement, H-piles can be driven through dense/hard layers with less resistance than displacement piles. However, in some instances H-piles may “plug” during driving, resulting in increased driving resistance, but also increased end bearing. H-piles may be subject to horizontal deviation if obstructions (e.g. boulders) are encountered, but are generally less susceptible to damage during difficult driving than a comparable pipe pile.

H-piles could be driven to bear in the bedrock. Alternatively, where bedrock is deeper (at the northern portion of the river crossing) the piles could be “hung up” above the bedrock, with the piles bearing in either the Qal2 or Qal3 Unit. Piles bearing in the Qal2 or Qal3 Unit would likely have less axial resistance than piles driven to bedrock. As such, piles hung up in Qa2 or Qa3 Unit may be preferred if the foundation loads are relatively light. We do not recommend terminating the piles in the Qls1 or Qls2 Units.

**Pipe Piles**

Pipe piles may be installed as either closed- or open-end. We understand that on NDDOT projects, pipe piles are typically driven closed-end and filled with concrete after driving (SEC piles). Similar to H-piles, open-end pipe piles can be driven through dense/hard layers with less resistance and lower driving stresses than an equally sized closed-end pipe pile due to their lower displacement. Closed-end piles may be designed assuming end bearing develops across the full cross-sectional area of the pile, while the amount of end bearing mobilized by an open-end pipe pile will depend on the degree of plugging (i.e. the extent that soil forms a solid section at the end of the pile). In general, open-end pipe piles are designed assuming a plug does not develop, unless test piles or experience indicate otherwise. SEC and OEP piles typically have diameters ranging from 12 to 36 inches. Although less common, OEP piles may have diameters up to 72 inches. OEP piles would be advantageous if deeper penetration is required to reach bedrock or to achieve lateral stability.
SEC piles should be driven through the Qls1 and Qls2 Units and bear on either bedrock or in the Qal2 or Qal3 Units. Along the northern half of the crossing, where the bedrock is relatively deep, SEC piles may encounter refusal or high driving resistance in the overburden. To reduce the likelihood of refusal above the design tip elevation or damage during installation, it may be preferable to utilize OEP piles, particularly if relatively deep penetrations are necessary for lateral stability.

Other Considerations

If piles are selected for design, we recommend conducting a wave equation analysis of pile driving (WEAP) to confirm that it will be feasible to drive the selected pile section to the required axial resistance. The WEAP evaluation will estimate stresses in the pile during driving and will provide a relationship between the blow count and estimated pile resistance for the assumed pile driving hammer. If desired, we can conduct the WEAP evaluation.

A design-phase pile installation and load testing program could be considered to confirm pile driveability and axial resistance, particularly if the design team prefers to utilize a pile type/size that is not routinely used on NDDOT projects. Design-phase pile testing can reduce risk for the designer, owner, and potential bidders.

For piles driven to bedrock, we recommend including pile protection (tips, points, or shoes) to reduce the likelihood of damage occurring during driving. The selected pile protection should fit flush with the outside of the pile section such that a gap is not created between the pile and the ground during driving, which could cause a substantial reduction in axial resistance and increase pile lengths.

Corrosion should be considered if steel piles are selected for the bridge. Typically, a sacrificial thickness of steel is included in the design to account for potential corrosion.

We recommend conducting dynamic testing during pile driving to evaluate axial resistance, to establish driving criteria, and to confirm installation damage does not occur. Conducting dynamic testing will also allow the use of higher resistance factors in accordance with the AASHTO, as indicated above. Vibration monitoring should also be conducted during construction to confirm that vibration levels satisfy project requirements.
Advantages and Disadvantages

Advantages for driven piles include:

- Dynamic testing can be completed to confirm axial resistance and to determine driving criteria.
- The driving resistance (blow count) provides a way to evaluate the axial resistance of each pile.
- The NDDOT and local contractors are familiar with driven piles.
- Pile driving equipment is relatively easy to manage in overwater construction.

Disadvantages for driven piles include:

- Vibration from pile driving can negatively affect adjacent structures, including the existing bridge.
- Pile groups require the construction of a pile cap. Construction of the pile cap may require a sloped or shored excavation on land. In the river channel, a cofferdam may be required.
- Noise generated during pile driving can negatively affect aquatic life, and restrictions may be placed on the pile driving schedule.
- Unexpected obstructions, variations in depth of rock, or piles not developing adequate resistance can result in pile quantities greater than estimated.
- Driven piles typically have less lateral load capacity than drilled shafts, unless battered. However, we do not recommend using batter piles where large ground movements are possible, such as in landslide terrain.

DRILLED SHAFTS

We anticipate that drilled shafts will be a feasible foundation alternative for the Long X Bridge. However, we understand that the NDDOT does not typically utilize drilled shafts.

The diameter of drilled shafts typically ranges between 3 and 8 feet, with lengths up to 200 feet. Drilled shafts for the bridge could be socketed in the bedrock or hung up in the Qal3 Unit. We do not recommend terminating drilled shafts in the Qls1 or Qls2 Units. For preliminary design, drilled shafts can be analyzed using the axial resistance parameters provided in Table TM-3-1.
TABLE TM-3-1
RECOMMENDED AXIAL DESIGN PARAMETERS
FOR PRELIMINARY ANALYSIS OF DRILLED SHAFTS

<table>
<thead>
<tr>
<th>Geologic Unit</th>
<th>Nominal Unit Side Resistance (ksf)</th>
<th>Resistance Factor - Side Resistance</th>
<th>Nominal Unit End Bearing (ksf)</th>
<th>Resistance Factor - End Bearing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Qls1</td>
<td>1.0</td>
<td>0.45</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Qls2</td>
<td>1.0</td>
<td>0.45</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Qal1</td>
<td>0.2</td>
<td>0.55</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Qal2</td>
<td>1.1</td>
<td>0.45</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Qal3</td>
<td>1.7</td>
<td>0.55</td>
<td>35</td>
<td>0.50</td>
</tr>
<tr>
<td>Tsb</td>
<td>5.0</td>
<td>0.50</td>
<td>75</td>
<td>0.60</td>
</tr>
</tbody>
</table>

NOTE:
1) Nominal resistance should be taken as the nominal unit side resistance times the circumferential area of the shaft plus the unit end bearing times the base area of the shaft. At the strength limit, the nominal resistances should be multiplied by the specified resistance factors.
2) The specified resistance factors should be reduced by 20% for a non-redundant drilled shaft.

Base Grouting

Rock-socketed drilled shafts would have greater axial resistance than a shaft of the same diameter bearing in the Qal3 Unit. Additionally, rock-socketed shafts would have a stiffer load-settlement response under axial loading than a shaft bearing in the alluvium. However, drilled shafts bearing in the alluvium could be base grouted to increase the unit end bearing of the shaft and to reduce post-construction settlement.

Base grouting essentially consists of pumping neat-cement grout under high pressure to the base of a drilled shaft after the drilled shaft concrete has cured. A grout delivery system that allows grout to be pumped from the top of the shaft to the soil below the base of the shaft is installed with the rebar cage. Grout pumped through the delivery system under pressure densifies soils beneath the bottom of the shaft, increasing the unit end bearing and the stiffness of the soil. As such, base grouting is most effective in granular soils such as those encountered in the Qal3 Unit. The incremental cost associated with extending the drilled shafts to bedrock in lieu of grouting should be considered in the evaluation of base grouting.
Maximum grout pressures for base grouting are typically on the order of 800 pound per square inch. The reaction to resist the uplift pressures generated during grouting is developed in side resistance. Therefore, base grouting should only be considered where the shaft is long enough to mobilize adequate side resistance such that upward movement of the shaft does not occur during grouting.

**Construction Considerations**

Drilled shaft excavations at the site will encounter a variety of materials, including stiff clays, saturated sands, and very low strength bedrock. Construction of drilled shafts will require the use of temporary casing or drilling fluid (or a combination thereof) to maintain excavation stability. At locations where the drilled shafts will extend through saturated cohesionless alluvium, the use of drilling fluid will likely be required to prevent heave at the base of the excavation caused by differential water pressure. Drilled shaft construction techniques could vary depending on the contractor’s preference and location (i.e. subsurface conditions).

Drilled shafts constructed at the site will most likely require tremie (underwater) concrete placement. We recommend completing integrity testing of such shafts to confirm the quality of the concrete. Testing could consist of cross-hole sonic logging (CSL), thermal integrity profiling (TIP), or gamma-gamma logging (GGL). CSL and GGL methods require access tubes to be attached to the rebar cage. Sacrificial TIP sensors can be tied to the rebar cage and cast into the concrete, or testing could be completed through conventional access tubes. The test frequency is typically established by the owner. Based on our experience, integrity testing of every non-redundant shaft and 25 percent of redundant shafts is reasonable. However, access tubes should be installed in every shaft to facilitate integrity testing in the event of an abnormal event during construction (e.g. delay in concrete placement).

**Load Testing**

AASHTO allows the use of higher resistance factors if axial load testing is completed, which can result in a substantial reduction in drilled shaft quantities. Assuming load testing is completed, a resistance factor of 0.70 may be used for loading in compression, compared to resistance factors ranging from 0.35 to 0.60, depending on the particular design method, if testing is not completed. Additionally, because of the NDDOT’s limited experience with drilled shafts, load testing may be desired to reduce the uncertainty in the axial resistance of drilled shafts constructed at the site.
Because of the relatively high test loads required for drilled shafts, a compression load test would require a substantial reaction frame supported by additional piles/shafts or dead weight. Therefore, it is often more cost-effective to complete a bi-directional (Osterberg cell, commonly referred to as O-cell) load test.

The O-cell essentially consists of a hydraulic jack that is sandwiched between two steel plates, each having a diameter slightly smaller than the shaft to be tested. The apparatus is then cast into the drilled shaft. After the concrete cures, the jack is pressurized, cracking the concrete into an upper and lower segment. As the pressure to the jack is increased, the drilled shaft is loaded bi-directionally, with the upper and lower segments providing the reaction force for each other. Instrumentation installed with the shaft measures displacements and strains, which can be utilized to determine the distribution of soil resistance.

Advantages and Disadvantages

Advantages for drilled shafts include:

- Generally, drilled shafts are less susceptible to corrosion than driven piles.
- Drilled shaft construction results in less noise and vibration than pile driving.
- Drilled shafts can connect directly to bridge columns (i.e. a cap is not required), eliminating the need to excavate and construct a pile cap.
- Drilled shafts typically have greater axial and lateral resistance than driven piles. This may be particularly advantageous at the south abutment where landslide movements and forces may need to be accommodated by the bridge foundations.

Disadvantages of drilled shafts include:

- There is less certainty in the axial resistance of drilled shafts compared to piles because each pile is essentially load tested during driving.
- Static load testing of drilled shafts is more expensive and time consuming than equivalent dynamic testing of driven piles.
- Construction of drilled shafts, particularly when tremie concrete placement is required, can be challenging and requires good construction QA processes, including observation by the geotechnical engineer and integrity testing.
- Drilled shaft construction usually requires several pieces of large equipment, which can make overwater construction more challenging than for driven piles.
ADDITIONAL CONSIDERATIONS

Refer to technical memorandum TM-1 for a discussion on the design concerns concerning the potential shallow and deep-seated landslide issues. As previously discussed, drilled shafts can typically carry greater lateral loads than piles.

Encl:
Figure TM-3-1: Site and Exploration Plan
Figure TM-3-2: Generalized Subsurface Profile A-A’
Figure TM-3-3: Long X Bridge, Boring RP 126-02, Driven Pile Option, Summary of Axial Resistance
Figure TM-3-4: Long X Bridge, Boring RP 126-04, Driven Pile Option, Summary of Axial Resistance
Figure TM-3-5: Long X Bridge, Boring RP 126-05, Driven Pile Option, Summary of Axial Resistance
Figure TM-3-6: Long X Bridge, Boring RP 126-06, Driven Pile Option, Summary of Axial Resistance
Figure TM-3-7: Long X Bridge, Boring RP 126-07, Driven Pile Option, Summary of Axial Resistance
Figure TM-3-8: Long X Bridge, Boring RP 126-08, Driven Pile Option, Summary of Axial Resistance
**NOTES:**

1. Axial pile resistance indicated in plots has been limited based on the structural resistance of the steel pile section in accordance with the NDDOT Design Manual. The limiting structural resistance was assumed to be equal to 0.33 times the steel area times the yield strength of the steel. For H-piles, we assumed a yield strength of 50 ksi. For pipe piles, we assumed a yield strength of 45 ksi.

2. Plotted axial geotechnical resistance was estimated using static methods and includes a resistance factor of 0.45, per AASHTO (2014). During installation, dynamic testing should be completed in accordance with AASHTO and a resistance factor of 0.65 may be applied to the nominal resistance indicated by dynamic testing.

3. The analyses were performed based on recommendations presented in the AASHTO LRFD Bridge Design Specifications (2014), Design and Construction Driven Pile Foundations (FHWA, 2006), and our experience. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 widths/diameters center-to-center). Once final pile group sizes and spacing are determined, the axial resistance of the pile group should be re-evaluated.

4. See memorandum for additional foundation considerations and a description of geologic units.
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4. See memorandum for additional foundation considerations and a description of geologic units.
TECHNICAL MEMORANDUM TM-4

SLOPE STABILITY ANALYSES METHODOLOGY
This memorandum discusses the methodology used for completing slope stability analyses to develop the preliminary recommendations for permanent cut and fill slopes, which were provided in Technical Memorandum TM-2, for the proposed widening of US Highway 85 (US 85). All stationing referenced herein refers to the existing alignment and stationing provided by KLJ in the file control_mckenzie.dgn.

BACKGROUND

To assess appropriate cut and fill slope angles, we completed slope stability analyses for selected representative slope cross-sections. We first completed back-analyses at locations where slope failure have occurred (see Technical Memorandum TM-2 for a discussion of the prior slope failures). Because these slopes have failed, a factor of safety (FS) of 1.0 can be assumed at the time of failure. We used the back-analysis to calibrate the slope stability model and to assess selected shear strength parameters, as discussed in the following section. We then used these stability models developed from the back-analysis to complete slope stability analyses for the proposed cuts and fills.

We completed the slope stability analyses using the software SLOPE/W 2016\(^1\). To calculate FS values, we used the method of slices by Morgenstern and Price\(^2\), which satisfies both force and moment equilibrium. We represented pore water pressures by a piezometric line.

\(^1\) Geo-Slope International, Ltd., 2016, SLOPE/W 2016, Version 8.16
BACK-ANALYSIS

We completed the back-analyses at locations where prior failures occurred and where sufficient information was available to develop a slope stability model. The locations we considered are summarized below. Further discussion of the landslides that occurred at these areas is provided in Technical Memoranda TM-1 and TM-2.

- Failures in cut slopes above elevation 2,360 feet (approximately Stations 6560+00 to 6590+00). Selected output from our analyses for this case are provided as Figures TM-4-2 and TM-4-3.
- Failures in fill slopes during the original construction of the roadway (approximately Stations 6560+00 to 6567+00 and Stations 6590+00 to 6595+00). Selected output from our analyses for this case are provided as Figures TM-4-1 and TM-4-4.
- Large natural landslide near the south bank of the Little Missouri River (approximately Stations 6660+00 to 6670+00). Selected output from our analyses for this case are provided as Figure TM-4-5.
- Large natural landslide on west side of roadway near Horseshoe Bend (approximately Stations 6730+00 to 6760+00). Selected output from our analyses for this case are provided as Figure TM-4-6.

Slip Surface Geometry

As discussed in Technical Memorandum TM-2, stability of the aforementioned landslide areas is likely controlled by thin layers of relatively weak (higher plasticity) bedrock. Therefore, we completed our stability analyses assuming five-foot-thick layers of weaker material at elevations where sliding was indicated by inclinometers, where movement was suggested by prior reports by others (see TM-2), or where we interpreted landslide movement based on our site observations and subsurface characterization.
Material Properties

For our analyses, we divided the subsurface profile into the units listed below, with the specified properties.

### SOIL/ROCK PARAMETERS USED FOR STABILITY ANALYSES

<table>
<thead>
<tr>
<th>Unit</th>
<th>Total Unit Weight, $\gamma$ (pcf)</th>
<th>Effective Stress Friction Angle, $\phi'$ (deg.)</th>
<th>Effective Stress Cohesion, $c'$ (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill</td>
<td>125</td>
<td>25</td>
<td>100</td>
</tr>
<tr>
<td>Overburden (Landslide Debris)</td>
<td>120</td>
<td>25</td>
<td>0</td>
</tr>
<tr>
<td>Bedrock (Sentinel Butte Formation)</td>
<td>125</td>
<td>22</td>
<td>1,500</td>
</tr>
<tr>
<td>Slip Surface</td>
<td>125</td>
<td>Non-Linear Shear Strength Envelope (See Below)</td>
<td></td>
</tr>
</tbody>
</table>

For the overburden and bedrock, we selected shear strength parameters based on correlations with plasticity\(^3\) and our judgment. We determined the selected cohesion value for the bedrock based on back-analysis of existing stable cut slopes to a minimum FS of 1.1.

For the slip surface material, we considered two strength conditions, the fully softened shear strength and the residual shear strength. Essentially, the fully softened shear strength, which is greater than the residual shear strength, would be applicable to the weak layer before significant landslide movement occurs. The residual shear strength would be applicable after significant landslide movement has occurred. For both the fully softened and residual shear strengths, we utilized non-linear shear strength envelopes based on correlations with the liquid limit and clay-

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sized fraction\textsuperscript{3}. Based on laboratory testing of selected samples of the comparatively weaker material, we assumed a liquid limit of 175 and a clay-size fraction of 50 percent for input into these correlations.

For initial analyses, we assumed the entire length of a given weak layer was at fully softened conditions. However, the analyses indicated FS values greater than 1.0, suggesting that the actual shear strength of the weak layer (or a portion of the layer) at the time of failure was less than the fully softened strength. This is consistent with literature\textsuperscript{4} indicating that a portion of the slip surface in stiff clays, claystones, and shales (the type of material present along the alignment) may be at a residual condition prior to failure occurring.

Residual conditions may develop near the toe of the landslide due to prior smaller slope failures or due to localized concentration of strain caused by unloading of the slope. Therefore, we assumed residual conditions for a portion of the weak layer near the toe of the slope. We selected the length of material at a residual condition such that an FS of 1.0 was back-calculated for a slip surface representative of the actual failure surface. If residual conditions were assumed for the entire weak layer of an existing slope, the analyses indicated instability for relatively large failures that were not representative of the observed failures, further suggesting that only a portion of the weak layer was at a residual condition at the time of the initial failure.

**Groundwater Conditions**

We initially selected a piezometric surface representative of groundwater conditions encountered in the subsurface explorations and measured in the groundwater instrumentation. We then adjusted the piezometric surface as necessary to produce and FS of 1.0 for the assumed failure surface.

**ANALYSIS OF PROPOSED SLOPES**

After completing the back-analyses, we used the calibrated soil, rock, and groundwater parameters to complete stability analyses of the proposed slopes at corresponding locations. To account for the existing slope failures (in both cut and fill slopes), we reduced the shear strength

to residual conditions along the portion of the back-calculated failure surface coincident with the weak layer.

As previously discussed, excavation and unloading can cause strain and deformation near the toe of a given cut slope, resulting in the strength of material near the toe decreasing from fully softened to residual conditions. To account for this behavior, at cut slopes where the weak layer at elevation 2,360 feet controlled stability, we assumed a portion of the weak layer near the toe of the slope will transition to residual conditions due to excavation. We assumed the zone of residual material would extend a horizontal distance from the toe of the slope equal to approximately 60 percent of the proposed cut height. This distance is similar to the length of residual material we determined from back-analysis of initial failures in the cut slopes (excluding cases where multiple, progressive failures have resulted in a greater length of residual material near the toe of the slope). Additionally, this distance, which is based on performance of the existing slopes, is likely conservative in our opinion, because the proposed slopes are flatter than the existing and prior slopes. In general, shear strains near the toe of a slope will decrease with the slope of the cut, suggesting the length of material that could transition to a residual condition would be shorter for a flatter slope.

The results of our stability analyses are presented in Technical Memorandum TM-2 as recommended cut and fill slope angles, along with other slope stability considerations. Technical Memorandum TM-2 includes discussion of the FS values assumed for the proposed slopes. Selected slope stability outputs for a critical fill section and two critical cut sections in the Badlands reach of the alignment are included as Figures TM-4-7 through TM-4-9.

Encl:
Figure TM-4-1 Station 6565+00 – Back-analysis of Fill Failure During Original Construction
Figure TM-4-2 Station 6567+00 – Back-analysis of Current Cut Slope Failure
Figure TM-4-3 Station 6585+00 – Back-analysis of Current Cut Slope Failure
Figure TM-4-4 Station 6594+00 – Back-analysis of Fill Failure During Original Construction
Figure TM-4-5 Long X Bridge – Back-analysis of Current Slope Failure
Figure TM-4-6 Horseshoe Bend – Back-analysis of Current Slope Failure
Figure TM-4-7 Station 6560+00 – Proposed 3.5H:1V Fill Slope
Figure TM-4-8 Station 6567+00 – Proposed 2.25H:1V Cut Slope
Figure TM-4-9 Station 6585+00 – Proposed 2.25H:1V Cut Slope
Station 6565+00: Back-analysis of Fill Failure During Original Construction

Name: Failure

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Figure TM-4-1
Station 6565+00 - Back-analysis of Fill Failure During Original Construction
Station 6567+00 - Back-Analysis of Current Cut Slope Failure

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Figure TM-4-2
US-85 - I-94 to Watford City Bypass
23-1-01453-210
Date: 3/6/2017
File Name: 6585+00.gsz
Name: Existing

Figure TM-4-3
Station 6585+00 - Back-Analysis of Current Cut Slope Failure
### Approx. Station 6594+00: Back-analysis of Fill Failure During Original Construction

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![Figure TM-4-4](Image)

Station 6594+00 - Back-Analysis of Fill Failure During Original Construction
US-85 - I-94 to Watford City Bypass
23-1-01453-210
Date: 1/6/2017
File Name: Section 1 (Parallel to slide).gsz
Name: Existing (2)

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Figure TM-4-5
Long X Bridge - Back-Analysis of Current Slope Failure
US-85 - I-94 to Watford City Bypass
23-1-01453-210
Date: 3/10/2017
File Name: HSB - STA 6745 - Back Analysis.gsz
Name: Existing

US-85 - I-94 to Watford City Bypass
23-1-01453-210
Date: 3/10/2017
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Name: Existing

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Figure TM-4-6

Horseshoe Bend - Back-Analysis of Current Slope Failure

Figure TM-4-6
Horseshoe Bend - Back-Analysis of Current Slope Failure
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**US-85 - I-94 to Watford City Bypass**

**23-1-01453-210**

**Date:** 3/10/2017  
**File Name:** 6560+00.gsz  
**Name:** Proposed_3.5:1  
**3.5H:1V Fill Slope Option**

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**Figure TM-4-7**  
Station 6560+00 - Proposed 3.5H:1V Fill Slope