November 2019

Appendix E

GEOTECHNICAL RISK EVALUATION

SD44/PLATTE-WINNER BRIDGE CORRIDOR STUDY AND ENVIRONMENTAL ASSESSMENT

Agreement No. 410583, Work Order PD-19-16

Project Nos. HP5596(19); P0044()290, PCN 05X0

PROJECT MEMORANDUM

GEOTECHNICAL RISK EVALUATION

To: Study Advisory Team (SDDOT, FHWA, SDGFP)

From: Ezra Ballinger, Wes Dickhut and Scott Mackiewicz (Braun Intertec Corporation); Tim Thoreen (HR Green)

Date: 7/6/2017_v2

Document Summary

This memorandum provides an overview of geotechnical impacts on roadway and bridge construction for consideration on the SD44/Platte-Winner Bridge and Corridor. The memorandum was originally submitted for consideration on April 25, 2017. The current version includes response to comments received from SDDOT Bridge and additional evaluation performed since the last submittal.

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Introduction

The HR Green/Braun Intertec Corporation Team (Task Team) has reviewed readily available topographic, geologic and geotechnical information within the project area and existing bridge plan information to evaluate the geotechnical risk(s) associated with altering the existing roadway and bridge alignment to another location within the project study area. The purpose of this geotechnical study was to aid in evaluating the existing alignment and selecting potential alternative alignments for the project in coordination with traffic, roadway and structure design criteria. The Task Team can use the results of this study to focus on the alignment(s) that prove most viable from a geotechnical standpoint and develop roadway and bridge design concepts for South Dakota Department of Transportation (SDDOT) and all the project stakeholders' consideration.

Geologic Overview

The geologic units and surficial soil types within the project study area are depicted on the attached Figures 1 and 2, respectively. In general, the project study corridor can be differentiated in three different geologic areas: 1) west approach, 2) Lake Francis Case/Missouri River crossing and 3) east approach, which are discussed in more detail below.

West Approach

The surficial soils across the majority of the west approach area (i.e., west of the Lake Francis Case/Missouri River crossing) are derived from weathering of the underlying Pierre Shale. The Pierre Shale is composed of several different members, including the Elk Butte, Mobridge, Sully, Gregory and Sharon Springs Members, all of which are comprised primarily of bentonitic clay shale. Where the soils have developed on slopes of about 5H:1V or flatter, as for that portion of the study corridor outside of the Missouri River valley, they are relatively stable and suitable for growing crops or grazing cattle. Along the dissected western valley, the slopes are generally steeper than 3H:1V and Pierre Shale has been exposed due to erosion from the river over millennia. The resulting slopes are comprised of a thin mantle of soil overlying the Pierre Shale or exposed Pierre Shale, which are often marginally stable. The slope instability and landslide potential results from the weathering of the Pierre Shale, which creates soils that are strength-sensitive to buildup of pore water pressures. Slopes steepened by a loss of soil at the toe through erosion, sliding, construction or other means can experience progressive failures that become increasingly larger with time.

The south-central portion of the West Approach area has surficial soils associated with the Oglalla Group, which was formed on top of the Pierre Shale. The Oglalla Group is comprised of interbedded clay, sandy siltstone and sandstone and includes the Ash Hollow, Valentine and Fort Randall Formations. The soils associated with weathering of these formations are predominantly silty in nature.

Lake Francis Case/Missouri River

The soils immediately adjacent to the edge of Lake Francis Case and beneath the river valley are comprised of alluvium deposited as the Missouri River has meandered through the valley as shown in Figure 2. The surficial alluvium is generally comprised of silts, sandy silts, clayey silts and silty sands. These soils generally have no continuity in their layering across a given site due to the complex process of erosion and deposition by the river. These near-surface soils are normally a combination of materials deposited by slack water during flooding and

tend to be finer-grained cohesive and moderately cohesive materials. Below the cohesive materials, the alluvium typically becomes coarser-grained, having been deposited in faster-flowing water. The material below this depth generally grades from sandy silts and silty sands, to medium to coarse sands and gravel/cobbles near the bottom of the river channel.

Below the alluvium is the Niobrara Formation, which is comprised of white to dark gray argillaceous chalk, marl, and shale that contains thin, laterally continuous bentonite beds, chalky carbonaceous shale, minor sand, and small concretions. The Niobrara Formation is commonly termed the Niobrara “chalk” and will be encountered at about 100 feet below the current bottom of the lake. Several outcroppings of the Niobrara Formation have been mapped along the edges of the river bank within the study corridor, as shown on Figure 2.

East Approach

The east approach alignment is underlain by surficial soils derived from the Pierre Shale or exposed Pierre Shale, similar to the conditions encountered along the west approach alignment, until the grades climb out of the river valley on the east end of the project area. The soils encountered outside of the river valley or in drainage channels running toward the river within the east approach are comprised of Glacial Moraine and Glacial Outwash. These soils were created through the advancement and subsequent retreat of the glaciers that terminated on the east/north side of the Missouri River. Moraine soils were pushed up in front of glaciers as a mix of ice, sand, clay, silt and gravel and were left in place as the glaciers retreated and the ice melted. Outwash soils were deposited by outward flowing water from beneath the glaciers as they melted and are typically comprised of similar particle sizes that precipitated out of the flowing water. Surficial soils created by erosion and repeated working of the glacial soils frequently results in a clay or mud soil that is conducive to growing crops or pastureland as seen in Figure 2.

Corridor Grades

As part of our study, we also reviewed the relative steepness of slopes (i.e., slope gradient) along the alignment. The slope gradient coupled with the mapped geologic conditions provides an indication of the potential for slope instability. The slope gradients along the project study corridor are presented on the attached Figure 3. After discussion with the project team, we have represented slope gradient in terms of low (<3 degrees, 19H:1V), medium (3 to 8 degrees, 19H:1V to 7H:1V) and high (>8 degrees, 7H:1V). The gradients along the entirety of the project corridor are generally high within about 2 to 6 miles of Lake Francis Case/Missouri River. The grades generally climb from the roadway elevation of about 1410 feet at the bridge ends to plateau elevations on the east side of the corridor at about 1700 feet and on the west side of the corridor at about 2100 feet. The plateau grade is relatively similar regardless of how far away from the river the plateau is reached, i.e. roadway elevations will need to increase about 600 to 700 feet over a distance of 2 to 6 miles.

Existing Roadway History

Since the roadway was constructed, numerous repairs and maintenance projects have been performed to maintain traffic through the corridor safely. The repairs and maintenance have included: 1) landslide mitigation, 2) drainage and grading projects, 3) realigning portions of the roadway, 4) grading and paving overlays, and 5) repairs to the bridge structure.

Landslides and Mitigation/Repairs

The identified landslides that have been mapped along the study corridor are presented in Figures 1 and 3. Landslides at these locations have been primarily caused by poor drainage that leads to a buildup of pore water pressures, leading to failure of the marginally stable soils derived from shale. Often a large storm event and the subsequent increased runoff also results in head cut erosion of a drainage channel(s). Subsequent migration of the head cut up the drainage channel results in loss of support at the toe of roadway embankments, which then results in slope instability. Repairs and mitigation methods that have been implemented to increase the stability of existing landslides or reduce the potential for new slides include:

- Check dams to reduce the speed of water flowing through the project area;
- Placement of sand underdrains on and off the roadway corridor to reduce the amount of water under the roadway;
- Installation of numerous drainage control structures such as concrete culverts and chutes on and off the roadway alignment to control water flow;
- Removing soil weight at the top of susceptible slopes;
- Placement of toe berms at the base of susceptible slopes;
- Grading and maintenance to roadway ditches to reduce the channel flow gradient which could result in erosion;
- Fence line berms to aid in controlling water flow; and
- Roadway realignment away from sliding areas.

Many of these repairs have been performed as much as 1 mile off of the centerline to redirect drainage away from and avoid concentrated flows at the roadway. Due to the frequent repairs and ongoing monitoring that has been performed along the existing roadway, existing corridor soils are well understood, the driving forces for slides have been properly characterized and slide prone areas are well known. SDDOT Materials & Surfacing staff characterize the SD44 roadway in this study area as currently being in the most stable condition they have ever seen.

Platte-Winner Bridge

The existing bridge is supported on foundations comprised of prestressed, cylindrical-shell piles filled with concrete and sand after placement. From review of construction records, the driven pile installation was aided by jetting, which resulted in a concentration of larger particles (cobbles and boulders) near the river bottom that were not carried to the surface by the jetting procedure. The concentration of these coarse materials resulted in difficulty obtaining the design depths for the driven piles. The concentration of the large cobbles and boulders surrounding the existing foundation elements will likely influence installation of new foundation in close proximity to existing foundations.

In addition, we further understand that two (2) of the pile caps have cracked as a result of damage caused by wind and ice loading and have been subsequently repaired. The damage and repair resulted in a bridge closure that required a detour of more than 70 miles.

Alternative Roadway Alignments

From a geotechnical perspective, the Task Team believes it is not prudent to consider new SD44 roadway alignment alternatives beyond that necessary to reconstruct the bridge and return to the existing alignment. We have based our recommendation on the following:

- Comparison of geology and soil types shown on Figures 1 and 2 indicate that the subsurface materials are generally consistent north and south of the existing alignment within the study area, indicating that landslides would likely be anticipated on alternative alignments similar to those on the existing alignment.
- The bottom and top elevation for roadway grades are generally fixed because of the bridge elevation and top of river valley elevation – thus shortening the horizontal distance of the climb will result in steeper grades of the roadway.
- The subsurface conditions and potential slope instability effectively limit grades, which would result in switchbacks and deep fill sections.
- The landslide issues along the existing roadway alignment have been evaluated, are understood and have been effectively addressed by years of observation, monitoring, and millions of dollars in repairs. Altering the alignment effectively abandons these repairs and starts over on a new, more unknown area.
- If a new alignment is selected and pursued, the existing alignment will still likely need to be maintained by local jurisdictions to allow continued access for residents and Game, Fish and Parks users. If the roadway were to be turned over to a local jurisdiction, they are unlikely to have the resources to conduct the necessary maintenance program given the concerns noted above.
- The roadway is characterized as being in the best shape of its history by those responsible for its maintenance.
- Known issues on the current SD44 roadway that are under consideration for repair are relatively minor realignments and could be performed as part of the corridor upgrades.

Bridge Alignment Considerations

A new bridge will be constructed in the vicinity of the existing bridge. Adding width to the existing bridge is not desirable given concerns with regard to the bridge construction, the degradation of the substructure as a result of wind and ice loading, and the age of the existing foundations. In addition, the existing bridge will remain in place until a new one is constructed to avoid detouring the users 70 miles as is necessary when the bridge is closed. As new bridge alignments are evaluated by the design team, we are considering the minimum offsets from the existing bridge foundations due to the construction issues that have been previously documented. It is our recommendation that a new bridge be maintained as close to the existing bridge as feasible in order to keep potential roadway realignments on either end of the existing bridge as short as possible. In addition, alternative bridge alignments will result in lengthening the bridge an increasing amount the farther the bridge alignment is shifted off the current one.

As part of our review of the Materials & Surfacing archives, we noted a drawing that was prepared during litigation for the site. The document is titled *Diagram Showing Extent of Air Jet Sorting at Bent No. 18,* prepared by South Dakota Department of Highways under project WAR 008 Sec.1, dated January 3, 1965. Indicated on the drawing is a zone of influence for jetting when piling was placed that extends from the bottom of the air jets, shown as 36 feet below the lake bottom, upward to the bottom of the lake bottom. At locations nearest the air jetting tip, larger

materials were concentrated and at locations farther away, as the velocity of the air dissipated, smaller particles were concentrated. This results in concentrations of cobbles and boulders closer to the bottom of the air jets and concentrations of sand and gravel farther away from the bottom of the jets.

Based on the referenced sketch, the concentration of cobbles and boulders was predicted to have occurred within an area bounded by the tip of the piles at the bottom of the air jets and projecting at a 45 degree (1H:1V) upward from the bottom of the air jets to the bottom of the lake elevation. This area is anticipated to be highly problematic during the course of new foundation installations.

The concentration of sands and gravels was predicted to have occurred within an area bounded by the tip of the piles at the bottom of the air jets and a 60 degree (1.73H:1V) upward projection from the bottom of the air jets to the bottom of the lake elevation . We anticipate that within this area there will be a potential for encountering some foundation difficulties, however foundations can likely be advanced within the area.

Table 1 presents the radius from the centerline of the existing roadway at each Pier pile cap for avoidance of cobbles/boulders and sands/gravels. Our calculations were based on air jets extending to Niobrara Formation or 40 feet below the lake bottom at the centerline of the pile cap and roadway foundation, whichever was shallower. At the two Sill's, the minimum offset is calculated as 26 feet, to be at least three pile diameters away from the outermost pile supporting the abutment as these areas would not have been jetted.

Table 1: Minimum Offset Distances

	Minimum Radius from Bridge Centerline at Columns	
	Avoid Concentrated Cobbles and Boulders (feet)	Avoid Concentrated Sands and Gravels (feet)
Pier 2	25	30
Piers 3 to 9	65	95
Piers 10 to 24	70	100
Piers 25 & 26	65	95
Piers 27 & 28	55	75

The distances in Table 1 are the minimum offsets at the lake bottom. Projecting upward at a 10H:1V potential batter for foundation elements, results in an additional 8 horizontal feet at the bottom of pile cap elevation (1358 feet) where the river is deepest. For planning purposes at this time, the design team recommends maintaining a minimum offset distance of 100 feet from centerline of the existing bridge to the edge of the new bridge for the Piers 3 to 26. On the east side of Pier 26 and west side of Pier 3 the new bridge may shift closer to the existing alignment to reduce impacts to the existing roadway, as geometric design allows.

Causeway

As a result of conversations that emerged from the public input meetings, there is interest in considering a causeway as one means of reducing the bridge length. A causeway has been used effectively at the I-90 crossing over the Missouri River further north at Chamberlain, South Dakota.

The existing bridge elevation is 1409 feet. The lake bed grade beginning just inside the east abutment drops to an elevation of about 1300 feet within 250 feet of the bridge end. Consequently, to build a causeway from the east, fill thickness of over 100 feet will be needed to shorten the bridge 250 feet. Granular fill, fill placement under water, and relatively flat side slopes (i.e., large embankment footprint) would be required to construct the causeway. Due to placement methods and fill materials, we would anticipate that monitoring of fill settlements or special ground improvement techniques may be required to control long term settlements to within tolerable limits. The construction will also necessitate mitigation by the excavation of the same amount of material within similar elevations elsewhere on the lake to avoid altering the hydraulic conditions of the lake. A causeway placed on the west side of the existing bridge would be require shallower thicknesses of fill due to shallower grades to the lake bottom but fill thicknesses exceeding 50 feet would still be required to achieve design grades.

It is our opinion that because of the reasons stated in the previous paragraph a causeway is not appropriate for the project site.

Comparison of Bridge Foundation Options

The Task Team evaluated several foundation options to aid the project team in selecting the most feasible type given the numerous variables. Based on our experience with other major river crossings, similar geology and local conditions, we have identified the following as feasible foundation options:

- Drilled Shafts – Constructed by using an auger to drill through the alluvial soils and coring or other rock drilling techniques to socket into the Niobrara formation. Drilling would include using slurry, temporary casing, permanent casing or a combination of slurry and casing to achieve anticipated design depths.
- Closed-Ended Pipe Piles – Steel pipe piles with diameters up to 48 inches driven with a plate on the bottom. Subsequent to driving, the piles would be filled with sand and/or concrete to increase axial and/or lateral resistance and provide corrosion resistance.
- Open-Ended Pipe Piles – Steel pipe piles with diameters up to 48 inches driven with a protective driving shoe into the Niobrara formation. Typically, open-ended piles are filled with concrete to some depth in order to increase axial and/or lateral resistance, in addition to providing corrosion resistance.
- H-Piles – steel piles driven to the Niobrara Chalk.

We have prepared a comparison matrix of these foundation options identifying some of the variables for them attached on the following page.

Nominal Geotechnical Resistance

Axial

We have developed an estimate of the nominal geotechnical resistance for each foundation type for use in evaluating construction costs. We utilized *Sheet 2 of 21 Subsurface Investigations for 5655'-6" Bridge over Ft. Randall Res.* for Project WAR-008-SEC.1, dated November 1961 to evaluate the subsurface profile and select analysis locations. For planning level estimates, thus far we evaluated the resistance at approximate Station

20+00 (upper to middle elevation of Niobrara Chalk) and Station 34+00 (lowest elevation of Niobrara Chalk).

The subsurface profile below the water is comprised of an approximate 10-foot-thick layer of “mud” underlain by river sands and gravels to the Niobrara Chalk. In our calculations, we treated the “mud” as a soft, lacustrine silt and clay deposit. We modeled the river sands and gravels as two layers, with the upper layer transitioning to the bottom layer where the SDDOT drive test blow counts were over 50. The upper layer was treated as a loose silty sand with some gravel and the lower layer as a medium dense silty sandy gravel. We estimated the intact rock strength of the Niobrara Chalk by reviewing published values for Pierre Shale and Niobrara Chalk. The Niobrara Chalk was assumed to have a strength value higher than the Pierre Shale and lower than the Limestone (values were taken from Farmer, 1968 and Jeager & Cook, 1969). For conservatism we took the upper limit of Pierre Shale strength as the value for the Niobrara Chalk, 2,500 pounds per square inch (psi). Using a reduction factor of 10 for the jointing, we utilized an unconfined compressive strength of 250 psi to evaluate the bearing capacity of the Niobrara Chalk.

We used the computer program UniPile, version 5, to estimate the static, nominal, geotechnical resistance of the driven deep foundation systems. UniPile software, developed by UniSoft Geotechnical Solutions Ltd., can calculate pile resistance using a variety of methods. For our analysis, we used the Beta-method, an effective stress method, to estimate the static geotechnical resistance for these piles. This method evaluates shaft resistance using Bjerrum-Burland beta coefficients (β), which are based on soil type and effective friction angle. We estimated the β values for the soft lacustrine, loose alluvium and medium dense alluvium using Table 7-9 from the Federal Highway Administration (FHWA) Publication No. NHI-16-009, Design and Construction of Driven Pile Foundations – Volume 1, September 2016, and our experience in similar soil conditions. The Beta-method determines end-bearing resistance using toe bearing capacity factors (Nt), which are also based on soil type and effective friction angle. We estimated the Nt values from Table 7-9 of the September 2016 FHWA publication and our experience. The end-bearing resistance in the Niobrara Chalk was estimate to be 9 times the unconfined compressive strength times the bearing area. Top of pile was assumed to be the bottom of pile cap identified on the existing plan sheets, elevation 1358 feet. Table 2 presents the nominal geotechnical resistance at the top of the Niobrara Chalk at the two locations. The total pile length is 101 feet at Station 20+00 and 176 feet at Station 34+00. The values presented in Table 2 do not include any resistance factors and assume infinite structural capacity of the piles. An appropriate resistance factor based on the level of construction control would need to be applied to the nominal values and the factored structural resistance of the piles also calculated to determine the appropriate factored load.

Table 2: Nominal Geotechnical Resistance for Driven Foundations

Foundation Type	Size	Station 20+00 Capacity (kips)	Station 34+00 Capacity (kips)
Closed-Ended Pipe Piles	24” Diameter, 1” wall	1150	2600
	36” Diameter, 1” wall	2400	4800
	48” Diameter, 1” wall	4200	7700
Open-Ended Pipe Piles	24” Diameter, 1” wall	500	1700
	36” Diameter, 1” wall	750	2600
	48” Diameter, 1” wall	100	3400
H-Piles	14x73	300	1000
	14x102	325	1100
	14x117	350	1150

The values in Tables 3 and 4 can be used to evaluate the resistance for drilled piers. The values in the tables do not include a resistance factor.

Table 3: Nominal Resistance Parameters for Drilled Piers at Station 20+00

Stratum	Elevation of Stratum (feet)	Side Friction/Adhesion, psf	End Bearing, psf
Soft Lacustrine	1315 – 1305	Ignore	Not Recommended
Loose Alluvium	1305 – 1280	Ignore, potential for permanent casing.	Not Recommended
Medium Dense Alluvium	1280 – 1260	1060 – 1360, average of 1210	Not Recommended
Niobrara Chalk	≤ 1260	1380	120,000

Table 4: Nominal Resistance Parameters for Drilled Piers at Station 34+00

Stratum	Elevation of Stratum (feet)	Side Friction/Adhesion, psf	End Bearing, psf
Soft Lacustrine	1290 – 1280	Ignore	Not Recommended
Loose Alluvium	1245 – 1280	Ignore, potential for permanent casing.	Not Recommended
Medium Dense Alluvium	1185 – 1245	1560 – 2480, average of 2020	Not Recommended
Niobrara Chalk	≤ 1185	2480	120,000

Table 5 presents our calculations for nominal resistance of various drilled pier sizes at each analyzed station.

Table 5: Nominal Drilled Pier Resistance

Diameter (feet)	Station 20+00 Capacity (kips)	Station 34+00 Capacity (kips)
6	3850	5650
8	6643	9000
10	10000	13000

Lateral

Lateral loads will be exerted on the foundations by ice loads and wind loads on the bridge. To resist these lateral loads for the driven foundation options, the elements will likely be battered. The following design parameters can be used to estimate lateral support of the driven piles and drilled piers.

Table 6: Soil Lateral Parameters at Station 20+00

Stratum	Elevation of Stratum, feet	Cohesion, psf	Friction Angle, degrees	Effective Unit Weight, psf	Soil Model
Soft Lacustrine	1315 – 1305	330	-	38	Soft clay below water
Loose Alluvium	1305 – 1280	-	30	54	Sand (Reese)
Medium Dense Alluvium	1260 – 1280	-	33	58	Sand (Reese)
Niobrara Chalk	≤ 1260	See Table 8			

Table 7: Soil Lateral Parameters at Station 34+00

Stratum	Elevation of Stratum, feet	Cohesion, psf	Friction Angle, degrees	Effective Unit Weight, psf	Soil Model
Soft Lacustrine	1290 – 1280	330	-	38	Soft clay below water
Loose Alluvium	1280 – 1245	-	30	54	Sand (Reese)
Medium Dense Alluvium	1245 – 1185	-	33	58	Sand (Reese)
Niobrara Chalk	≤ 1185	See Table 8			

Table 8. Lateral Parameters for Niobrara Chalk

Station	Elevation of Stratum, feet	Material Model	Effective Unit Weight, psf	Unconfined Compression Strength, psi	Initial Modulus of Rock Mass, ksi	Rock Quality Designation (RQD), %	Strain Factor (krm)
20+00	≤ 1260	Weak Rock (Reese)	125	250	3,000	70	0.0005 to 0.00005
34+00	≤ 1185						

Factored Geotechnical Resistance

Factored geotechnical pile capacities are determined by multiplying the pile driving resistance factor ($\Phi_{dynamic}$) by the nominal pile resistance (R_n). For driven piles, the appropriate resistance factor is selected based on the degree of construction control that will be utilized. Resistance factors range from about 0.4, used when construction control is by FHWA-modified Gates formula, to 0.8, used when static load tests are performed and quality control consists of high-strain dynamic testing on at least two percent of the production piles. Similarly, the resistance factor for drilled shafts depends on whether the design incorporates a static load test or not. With a static load test, the design can use a resistance factor of 0.70. Without a static load test, the design should use a resistance factor of 0.50 or 0.55, depending on the material providing resistance.

For this project, we anticipate that static load testing will not result in changing the pile lengths. However, static load testing will allow the design to utilize higher factored resistances, which should lead to fewer piles. Given the anticipated number of substructures and associated piles, we recommend including static load testing as part of the design. We also recommend performing the static load tests during the design phase of the project. This will allow the project team to evaluate the constructability issues associated with the different foundation options. Evaluating the constructability issues during design will reduce the risk of delays and additional cost during construction, thereby reducing the construction cost of the project.

Static load testing for the driven piles should follow ASTM International (ASTM) D1143 Standard Test Methods for Deep Foundations Under Static Axial Compressive Loads. For drilled shafts, static load testing should follow ASTM D1143 or ASTM WK30360 Test Methods for Deep Foundations under Bidirectional Static Axial Compressive Load. High-strain dynamic testing for driven pile should follow ASTM D4945 Standard Test Method for High-Strain Dynamic Testing of Deep Foundations.

Substructure Type	Axial Load Carrying Capacity	Lateral Load Carrying Capacity	Influence on Existing Structure	Contractor Experience	Construction Cost	Reference Projects
Drilled Shafts	+ Can use belling to increase capacity when get to bearing strata.	+ Concrete and reinforcing can transfer horizontal loads to underlying rock.	+ Can be installed with minimum vibrations in the substrata that transmit to existing foundation.	- Installation of drilled shafts will require a specialized contractor due to the depth of installation and diameters potentially greater than 6 feet.	- Use of drilled shafts will require permanent steel casing installed to rock and removal of all spoils from the shaft excavation. Cobbles may also create significant difficulty augering and advancing casing.	Discovery Bridge over Missouri River, Yankton, SD
Driven Closed Ended Pipe Piles	+ Steel walls and plug at base provide adequate bearing area.	+ Placement of concrete in pile can maximize lateral capacity.	- Due to large displacement, likely to induce some vibration that should be considered for impact on existing foundations.	+ Typical bridge contractor has experience in performing driven foundations.	- Driving of these piles may require predrilling and thick walls on piles to withstand the driving stresses through cobbles.	Four Bears Bridge over Lake Sakakawea in New Town, ND
Driven Open Ended Pipe Piles	+ Steel walls provide bearing area.	+ Placement of concrete in pile can maximize lateral capacity.	+ Due to less displacement, less likely to induce vibration that should be considered for impact on existing foundations.	+ Typical bridge contractor has experience in performing driven foundations.	+ Driving of these piles will not require predrilling, but thick walls will be needed to withstand driving stresses through cobbles.	US 34 over Missouri River, Mills County, IA
H-Piles	- Load carrying capacity limited by cross section of steel area.	- Lateral carrying capacity limited by cross section of steel area.	+ Vibrations may be induced through driving but of much smaller magnitude than anticipated from open or closed ended pipe piles.	+ Typical bridge contractor has experience in performing driven foundations.	- No predrilling needed for the piles, however risk of lateral displacement at toe when encountering cobbles/boulders.	Memorial Bridge over the Missouri River, Bismarck, ND

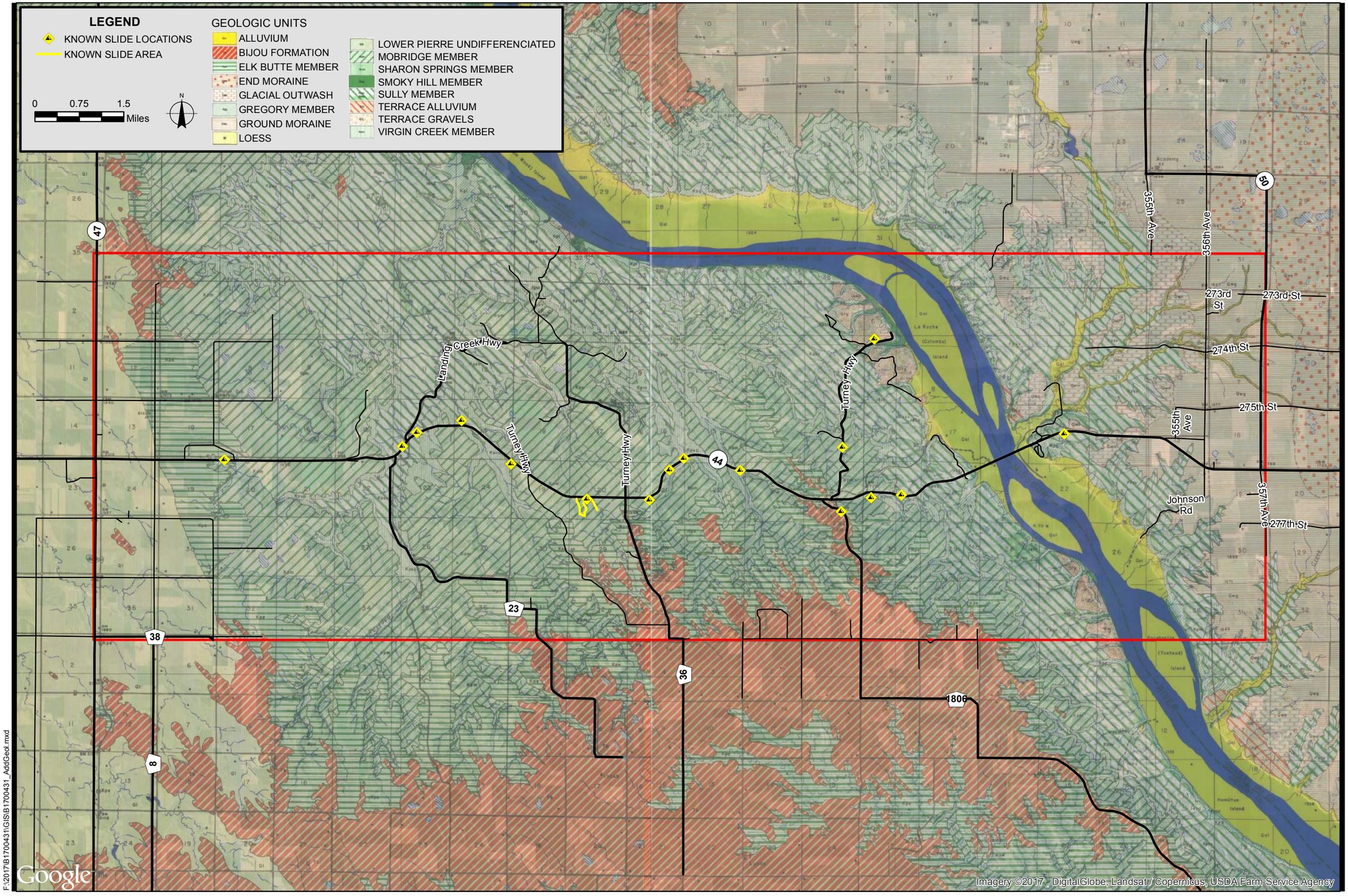

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FIGURE 1: AREAL GEOLOGY

SD-44/PLATTE-WINNER BRIDGE CORRIDOR STUDY
GREGORY COUNTY AND CHARLES MIX COUNTY
SOUTH DAKOTA

Project No:	B1700431
Drawing No.	B1700431_AddGeol
Scale:	1 in = 7,920 ft
Drawn By:	CMF
Date Drawn:	03/27/2017
Checked By:	EB
Last Modified:	6/29/17
Sheet:	1 of 3
Fig.	1

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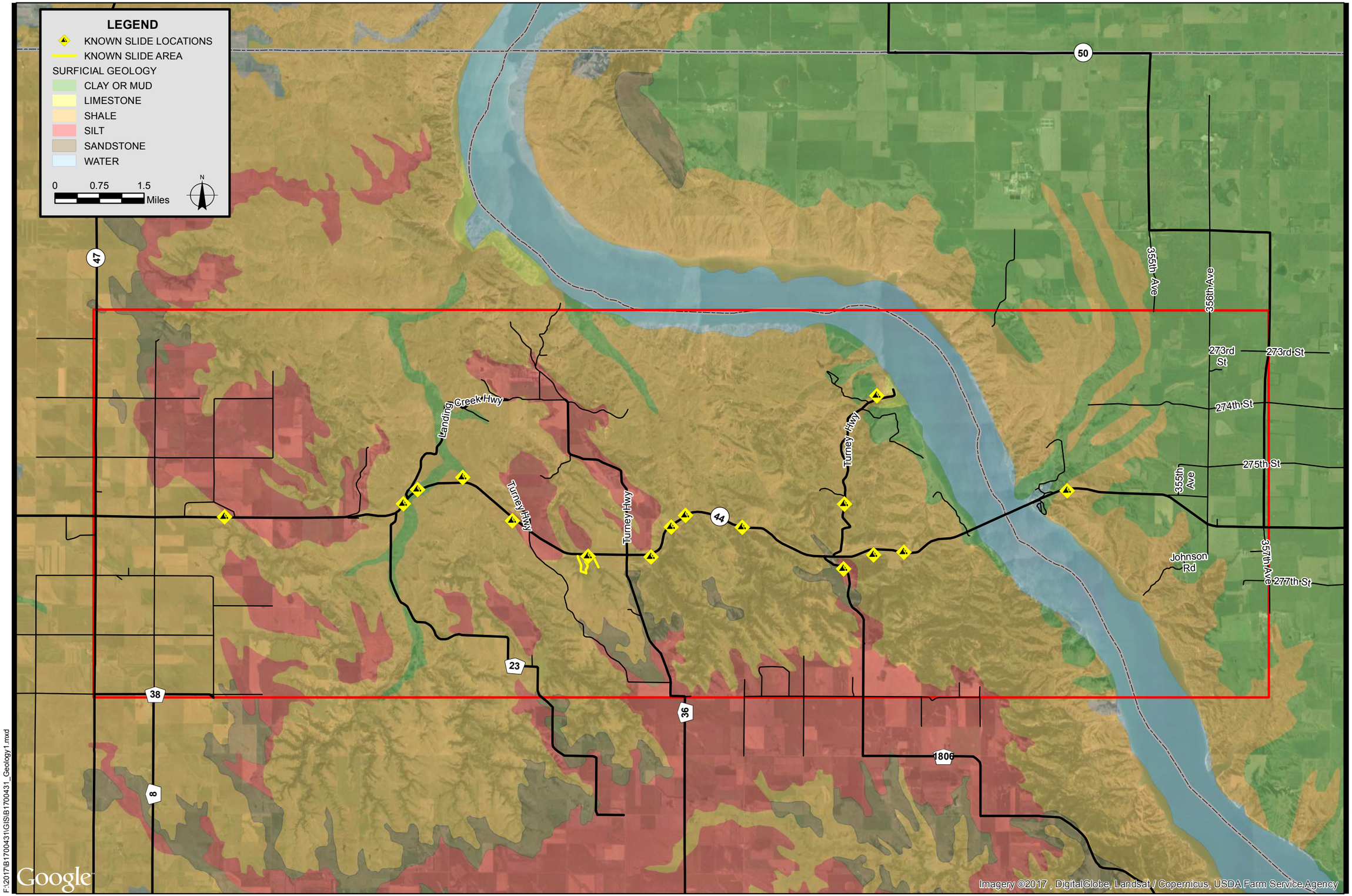

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FIGURE 2: SURFICIAL GEOLOGY SKETCH

SD-44/PLATTE-WINNER BRIDGE CORRIDOR STUDY
GREGORY COUNTY AND CHARLES MIX COUNTY
SOUTH DAKOTA

Project No:	B1700431
Drawing No.	B1700431_Geology1
Scale:	1 in = 7,920 ft
Drawn By:	CMF
Date Drawn:	03/27/2017
Checked By:	EB
Last Modified:	6/29/17
Sheet:	2 of 3
Fig.	2

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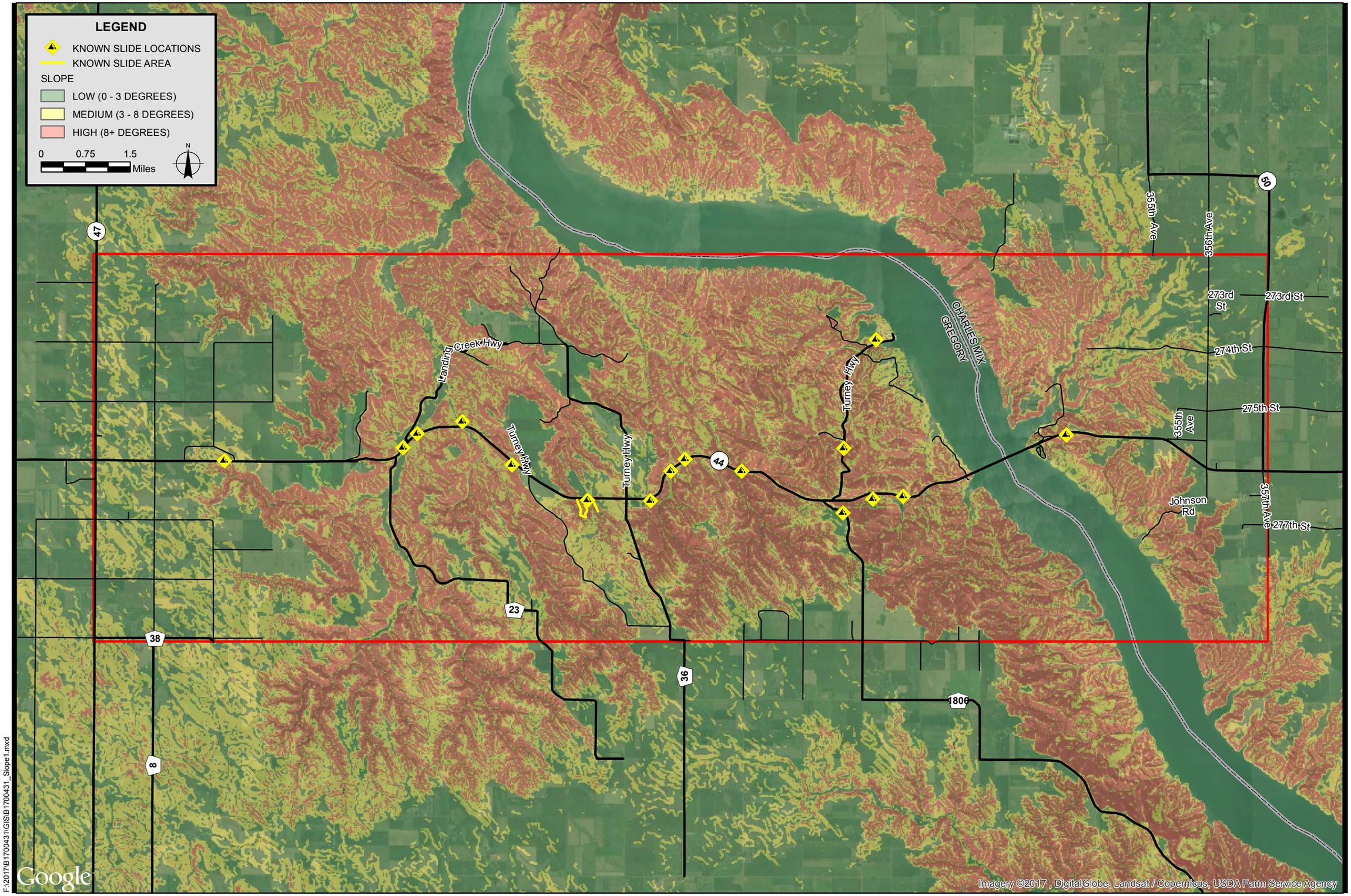

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FIGURE 3: SLOPE GRADIENT SKETCH

SD-44/PLATTE-WINNER BRIDGE CORRIDOR STUDY
GREGORY COUNTY AND CHARLES MIX COUNTY
SOUTH DAKOTA

Project No:	B1700431
Drawing No.	B1700431_Slope1
Scale:	1 in = 7,920 ft
Drawn By:	CMF
Date Drawn:	03/27/2017
Checked By:	EB
Last Modified:	6/29/17
Sheet: 3 of 3	Fig. 3