Geotechnical Conditions Memorandum

Kennedy Bridge Planning Study
US Trunk Highway 2 over Red River
State Project No. 6018-02
East Grand Forks, Minnesota
Grand Forks, North Dakota

Prepared for

CH2M Hill

Professional Certification:
I hereby certify that this plan, specification, or report was prepared by me or under my direct supervision and that I am a duly Licensed Professional Engineer under the laws of the State of Minnesota.

Megan J. L. Hoppe, PE
Project Engineer
License Number: 49905
January 24, 2014

Project BL-12-04987

Braun Intertec Corporation

BRAUN
INTERTEC

Braun Intertec Corporation
11001 Hampshire Avenue S
Minneapolis, MN 55438

Phone: 952.995.2000
Fax: 952.995.2020
Web: braunintertec.com

January 24, 2014

Project BL-12-04987

Mr. Dale Thomas
CH2M Hill
1295 Northland Drive, Suite 200
Mendota Heights, MN 55120

Re: Geotechnical Conditions Memorandum
Kennedy Bridge Planning Study
US Trunk Highway 2 over Red River
State Project No. 6018-02
East Grand Forks, Minnesota
Grand Forks, North Dakota

Dear Mr. Thomas:

We are pleased to present this Geotechnical Conditions Memorandum for the Kennedy Bridge Planning Study. This memorandum provides a summary of the available subsurface data and existing bridge foundation information. It also presents our geotechnical recommendations for rehabilitation and/or replacement of the existing bridge foundations as well as our general geotechnical opinions regarding foundation options for a new replacement bridge.

Thank you for making Braun Intertec your geotechnical consultant for this project. If you have questions about this report, or if there are other services that we can provide in support of our work to date, please call Megan Hoppe at 612.916.8723 or Jeff Gebhard at 612.282.3992.

Sincerely,

BRAUN INTERTEC CORPORATION

Megan J. L. Hoppe, PE
Project Engineer

Debra Mickelson
for

Jeffrey A. Gebhard, PE
Vice President - Engineering

Geo Report

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A. Introduction

A.1. Project Description

This Geotechnical Conditions Memorandum provides geotechnical and foundation information for the Kennedy Bridge Planning Study. The Minnesota Department of Transportation (MnDOT) and the North Dakota Department of Transportation (NDDOT) will use the study to improve the Kennedy Bridge, carrying US Trunk Highway 2 (USTH2) across the Red River of the North between Grand Forks, North Dakota and East Grand Forks, Minnesota. The study area extends from North 3rd Street on the west end to 4th Street NW on the east end, and it is limited to 1,000 feet upstream and downstream from the existing Kennedy Bridge.

A.2. Purpose

This memorandum provides a summary of the pertinent subsurface data and existing bridge foundation information. It also presents our geotechnical opinions for rehabilitation and/or replacement of the existing Pier 6 foundations as well as our general geotechnical opinions regarding foundation options for a new replacement bridge.

A.3. Reference Documents

To facilitate our evaluation, we were provided with or reviewed the following information or documents:

- Aerial imagery.
- United States Geologic Survey (USGS) topographic maps.
- Lithostratigraphic Map of the Greater Red River Valley, Minnesota and North Dakota in the Regional Hydrogeologic Assessment, Quaternary Geology – Southern Red River Valley, Minnesota (RHA-3, Part A, Plate 2) by Harris, West, Lusardi, and Tipping published by University of Minnesota, Minnesota Geologic Survey (1995).
- Skidmore Avenue Bridge Plan by NDDOT (July 25, 1962).
- Pile Load Bearing Test Report by NDDOT (April 11, 1963).
- Riverside Drive Neighborhood portion of Report by United States Army Corp of Engineers (USACE) (August 29, 2005).
- Kennedy Bridge, NDDOT Bridge No. 0002-358.090, Bent 5 and Pier 6 Tilt Report by NDDOT Geotechnical Section (data thru May 2, 2012).

- MnDOT Bridge Inspection Report, Bridge 9090 by MnDOT (June 21, 2012).
- MnDOT Structure Inventory Report, Bridge 9090 by MnDOT (June 21, 2012).
- NDDOT Bridge Inventory – Structure Inventory and Appraisal Sheet, Structure No. 0002-358.090 by NDDOT (June 21, 2012).

For brevity, these documents are not attached to this report.

B. Review of Existing Data

B.1. Surface Conditions

USTH2 is an east/west orientated highway traversing the northern portion of the United States. At the North Dakota and Minnesota border, the Kennedy Bridge takes USTH2 over the Red River of the North. This river is present within a broad valley called the Red River Valley, which is actually a bedrock valley that has been filled with Quaternary age glacial lake and river sediments. Surface elevations within this area of the Red River Valley are at or below elevation 850 over a width of about 20 to 25 miles centered about the Red River, which drains to the north, ultimately to Lake Winnipeg in Canada.

USTH2 is a four-lane highway, often divided and with paved shoulders where space allows. Within the towns of Grand Forks and East Grand Forks, USTH2 is also called Gateway Drive. Over the Kennedy Bridge, the roadway width is reduced with a low and narrow concrete median, curb and gutter, and narrow concrete walk.

Within the past decade, many of the residential areas adjacent to the Red River in Grand Forks and East Grand Forks have been converted to park land due to flooding. This is generally the case in the areas adjacent to the Kennedy Bridge, where river front parks are present. These park areas are covered with grasses, brush, and scattered trees; they also contain paved and unpaved trails. Further from the river, flood control levees have been constructed to control flood waters from entering the mostly residential and limited commercial areas beyond. The neighborhood northwest of Kennedy Bridge is called the Riverside Historic District. The nearest structure to the bridge is St. Michael's Hospital (Riverside Manor), a four to five story brick structure located southwest of the bridge.

The bridge is located at a meander bend in the river. At this location, the west bank is the outer bank, commonly called a cutslope where bank erosion and undercutting occur, and the east bank is the inner bank, referred to as a point bar where deposition occurs. Natural site grades and surface drainage in the green spaces generally fall toward the Red River. The flood control levees create a drainage barrier between the open spaces adjacent to the river and the developed areas beyond. Along USTH2, surface elevations increase toward the bridge along the approach embankments.

B.2. General Geologic Profile

The site is underlain with a variety of geologic materials associated with glacial advances from the north, northeast, and east as well as the various stages of Glacial Lake Agassiz. The materials generally found in the Grand Forks area are described below, beginning from the surface and extending down (youngest to oldest):

- Sherack Formation - Offshore to nearshore Lake Agassiz sediment consisting of laminated clay, silty clay, and silt that is light brown, yellowish gray, or olive brown in color.
- Brenna Formation - Offshore Lake Agassiz sediment consisting of unbedded clay that is dark gray to black and has very high water content and very low shear strength.
- Falconer Formation - Glacial sediment of the Red River lobe consisting of silty pebble-clay and clayey pebble-loam which is calcareous and light gray in color.
- Argusville Formation - Offshore Lake Agassiz sediment consisting of unbedded clay gray to dark gray in color.
- Wiley Formation - Offshore Lake Agassiz sediment consisting of thinly laminated olive gray to dark gray clay and light brownish gray to olive brown silt.
- Red Lake Falls Formation - Glacial sediment of the Wadena/Rainy lobe consisting of pebble-loam which ranges from brownish gray to olive brown; the upper portion often exhibits columnar (vertical) jointing, which is sometimes described as Fragipan.
- Goose River Group - Glacial sediment of the Red River lobe generally consisting of unbedded, calcareous, clayey pebble-loam. The upper portion can be silty and ranges from gray to very dark gray. The lower portion often contains shale pebbles, may contain lignite fragments, and ranges from light yellowish brown to dark gray.
- Crow Wing River Group- Glacial sediment of the Rainy/Superior lobe generally consisting of unbedded, calcareous, clayey pebble-loam to sandy pebble-loam that is dark grayish brown to light gray.

Bedrock is generally found at depths of 250 feet or more below the surface.

B.3. Existing Bridge

B.3.a. Superstructure

This existing bridge was constructed in 1963 and is considered fracture critical. The plans for the existing Kennedy Bridge indicate it has a total of thirteen spans, including the five western approach spans, two main spans over the river, and six eastern approach spans. The shorter approach spans range from 56 to 64 feet long while the main river spans are 279 feet in length. The main river spans consists of Parker style, steel high trusses.

B.3.b. Substructures

A total of 14 substructures support the spans. The substructures are supported on steel H-pile consisting of 14BP73 sections driven both vertically and battered. The maximum required bearing for all piles is shown on the plans as 90 tons per pile. Information from the bridge plans regarding the bridge substructures is summarized in the following table; the substructures are listed from west to east and elevations are rounded to the nearest half foot.

Table 1. Existing Bridge Substructure Summary

Substructure	Approximate Substructure Bottom Elevation (feet)	Pile Length (feet)	Approximate Pile Toe (Bottom) Elevation (feet)	Pile Design Load (tons)
Abutment 1 (west)	831	145	686	59
Bent 2	817-821	140	677-681	72
Bent 3	817-821	140	677-681	72
Bent 4	820½	140	680½	72
Bent 5	820½	140	680½	72
Pier 6 (west)	802	125	677	73
Pier 7 (center)	760	85	675	64
Pier 8 (east)	800½	120	680½	73
Bent 9	805	125	680	72
Bent 10	804½	125	679½	72
Bent 11	814	135	679	72
Bent 12	811	130	681	72

Substructure	Approximate Substructure Bottom Elevation (feet)	Pile Length (feet)	Approximate Pile Toe (Bottom) Elevation (feet)	Pile Design Load (tons)
Bent 13	812	130	682	72
Abutment 14 (east)	826	150	676	59

B.3.c. River Channel

The existing bridge plan indicates a high water elevation of 830.1 feet in April of 1897 and 826.1 feet in May of 1950. The low water elevation is shown as 786.2 feet in March of 1960. Original ground surface contours within the bridge plans show a high elevation of 828 feet west of the channel, a low elevation of 772 feet within the channel, and a high elevation of about 823 feet east of the channel. Based on this plan sheet, it appears the Red River typically occupies the channel below an elevation of about 795 feet.

B.3.d. Soil Borings

Sixteen borings logs (Boring No. 1 to 16) are shown in profile versus elevation within the existing bridge plans. These logs show ground surface elevations, soil types, N-values in blows per foot, and encountered water levels. The locations of twelve of these borings are shown in relation to the existing bridge substructures, including Borings No. 4 through 8 located on the west river bank, Boring No. 9 at Pier 7 within the river channel, and Borings No. 10 through 15 located on the east river bank. The borings extended to depths of about 120 to 235 feet below the surface. A summary of pertinent information from these borings is provided in the table below, with elevations rounded to the nearest half foot.

Table 2. Existing Bridge Soil Boring Summary

Boring No.	Nearest Substructure	Ground Surface Elevation (feet)	Measured Water Elevation (feet)	Bearing Layer Elevation (feet)	End of Boring Elevation (feet)
1	Unknown	828½	819½	690	678½
2	Unknown	828½	--	690½	591
3	Unknown	828	807	690	599
4	Bent 2	828	795½	690½	619
5	Bent 2	827	--	690	607
6	Bent 3	827	--	690	591½
7	Bent 4	827	--	690	672

Boring No.	Nearest Substructure	Ground Surface Elevation (feet)	Measured Water Elevation (feet)	Bearing Layer Elevation (feet)	End of Boring Elevation (feet)
8	Bent 5	814	--	690	644½
9	Pier 7	776*	793½	690½	649
10	Pier 8	812	794½	690	682
11	Bent 9	809	--	689½	688
12	Bent 10	809	797	689	634
13	Bent 11	816½	--	688½	676½
14	Bent 12	814½	799½	686½	670
15	Abutment 14	815	801½	686	681½
16	Unknown	817½	804½	687½	677½

***Note:** Boring No. 9 was located in the channel. Therefore, water was present at the surface, and the "Ground Surface Elevation" indicated is actually the mudline elevation in the channel bottom.

In the most general terms, the soil profile at these borings consists of fine grained alluvial and lake (lacustrine) sediments with N-values of less than 15 and often less than 10 above the "Bearing Layer" referred to in the table above. This "Bearing Layer" consists of coarser glacial sediments with N-values greater than 35 and typically in excess of 50. From the river channel west, the bearing layer tends to contain more sand while east of the river it contains more clay. Within these borings, the top of the bearing layer dips slightly from about elevation 690½ feet in the west down to elevation 686 feet to the east. The ground water elevations measured tend to mirror the river level within the borings located at and near the river channel; however, the measured ground water levels increase in elevation in the borings located further away from the river.

B.4. Pile Load Bearing Tests

In the winter of 1963, two of the H-piles supporting the existing bridge where subject to static compression load testing. Test Pile #1 (TP-1) was located at Pier 8 (east), and Test Pile #2 (TP-2) was located at Pier 6 (west). The intent was to install the piles to 70 tons via formula; statically test the pile to failure to determine skin friction resistance; redrive the pile deeper to 90 tons via formula; and statically test the pile to a maximum load of 300 tons. The test piles were driven with a Link-Belt Diesel Hammer Model 520. Practical refusal was defined as 0.02 inches or less per blow for the last 10 blows (0.2 inches in 10 blows). A summary of the test pile information presented in the test pile report is provided in the table below.

Table 3. Existing Bridge Pile Load Test Summary

Test Pile, Test	Depth (feet)	Pile Toe Elevation (feet)	Final Penetration (inch/blow)	Calculated Bearing by Formula (tons)	Maximum Test Load (tons)	48-hour Sustained Load (tons)	Maximum Pile Head Movement (inches)	Permanent Pile Head Movement (inches)
TP-1, 1	113.9	686.6	0.375"/10	79.9	300	240	3.04	2.06
TP-1, 2	114.1	686.4	0.1875"/20	93.5	300	270	2.14	0.94
TP-2, 1	115.7	686.3	1.0"/15	69.6	210	180	5.54	4.88
TP-2, 2	116.2	685.8	0.125"/10	100.5	240	220	1.50	0.72

B.5. USACE Instrumentation

B.5.a. General

The Riverside Drive Neighborhood, located northwest of the Kennedy Bridge, encompassed a known slope failure, as evidenced by an escarpment in the slope and distress in a brick wall parallel to the slope. In November of 1998, the USACE instrumented a section of slope just north of the Kenney Bridge on the west bank. The instrumentation included: six vibrating wire piezometers (P), three slope inclinometers (SI), and one tiltmeter (T). The piezometers and tiltmeter were automated instruments read remotely on at least a daily basis. The slope inclinometers were read manually on two to four occasions. The monitoring occurred between the Winter of 1998 and the Fall of 1999, encompassing the flood of 1999. Surface observations, slope monitoring results, slickensides within boring samples, and out of sequence stratigraphy support a progressive slope failure mode, which is typical in the Red River Valley.

B.5.b. Water Levels

Generally, the water levels within all the piezometers were influenced by the river level, with those located at the toe of the slope showing the most influence and those at the crest of the slope showing the least influence. The piezometers indicate ground water elevations increase with distance away from the river. Water levels within the shallow piezometer at the crest of the slope (P10) mirror precipitation events, while water levels within the shallow piezometer at the toe of the slope (P14) closely follow the river level. A summary of the piezometer information is provided in the table below.

Table 4. USACE Piezometer Water Level Summary

Instrument	Location on Slope	Elevation Zone of Readings* (feet)	Range of Measured Water Elevations (feet)
P10	Crest	807.8-815.8	815-822
P11	Crest	765.1-774.4	808-813
P12	Midslope	786.0-793.9	802-820
P13	Midslope	760.7-771.3	802-815
P14	Toe	784.6-791.7	795-823
P15	Toe	764.3-773.2	795-817
Red River	--	--	795-823

***Note**: Defined as bottom and top elevations of piezometer sand pack.

B.5.c. Slope Movement

The crest slope inclinometer (SI-6) indicates possible slight tilting beginning at great depth, while the midslope (SI-7) and toe slope (SI-8) inclinometers indicate well-defined shear zones. The slope movement is predominantly downslope (east) and slightly upstream (south). The 5-foot long tiltmeter (T-7) was located about 18 feet below the shear zone indicated by the slope inclinometer. The tiltmeter indicated very small changes (within the accuracy limits of the instrument) in the tilt of the inclinometer casing from vertical. A summary of the slope inclinometer and tiltmeter information is provided in the following table.

Table 5. USACE Slope Movement Summary

Instrument	Location on Slope	Elevation Zone of Readings* (feet)	Tilting Elevation (feet)	Shear Zone Elevation (feet)	Maximum Downslope Displacement (inches)
SI-6	Crest	728.7-828.7	755½	--	⅓
SI-7	Midslope	732.7-809.8	--	781½-785½	1⅔
T-7	Midslope	764.3-769.3	--	--	--
SI-8	Toe	735.0-798.7	--	769½-773½	2⅔

***Note:** Defined as bottom and top elevations of inclinometer casing or tiltmeter.

B.6. NDDOT Instrumentation

The slope movement that has occurred along the west bank of the Red River at the Kennedy Bridge is causing movement of Bent 5 and Pier 6. The NDDOT Geotechnical Section has monitored the west bank with an inclinometer installed south of Pier 6. Inclinometer Tube #1 (KENN 1) sheared off at an unspecified elevation in 2010, and Tube #3 (KENN 3) was installed to replace it. Initially Tube #2 was also installed further to the southeast of Tube #1; however, buoyant forces distressed the casing and valid readings could not be obtained at this location. The slope movement recorded at the inclinometers KENN1 and subsequently installed KENN3 is predominantly downslope (east) and downstream (north). A summary of this slope inclinometer information is provided in the following table.

Table 6. NDDOT Slope Movement Summary

Instrument	Reading Dates	Elevation Zone of Readings* (feet)	Lower Shear Zone Elevation Range (feet)	Lower Zone Maximum Downslope Displacement (inches)	Upper Shear Zone Elevation Range (feet)	Upper Zone Maximum Downslope Displacement (inches)
KENN 1	02/25/04 to 11/23/09	744-814	783-787	5½"	805-807	5½"
KENN 3	06/14/10 to 05/02/12	764-814	782-786	2"	806-798	3½"

***Note**: Defined as bottom and top elevations of inclinometer casing; top assumed to be elevation 814 ft.

B.7. Bridge Inspection/Inventory

B.7.a. MnDOT

The MnDOT bridge inspection report indicates:

- Pier 6 is moving toward the river and twisting, causing cracks measuring 1/16-inch wide, which are more severe than those observed in the east Pier 8. Sting line measurements indicate the pier wall is bowing 5½ inches. The south end of the pier appears to be moving more than the north end, with a total movement of 25 inches to the east.
- Erosion was noted and soil is scouring at Bent 5, undermining this pile cap.
- Major transverse cracks are present in the west abutment slope paving.

B.7.b. NDDOT

The ND DOT bridge inventory and appraisal indicates the following:

- The foundation is in fair condition and the channel bank is slumping.
- There are minor diagonal cracks with seepage at the corners of the backwall of the west abutment.
- Bent 5 leans in (toward river) 2¾ inches in 4 feet, with the lean increasing over time.
- Pier 6 leans away (from the river) 1⅞ inches in 4 feet, with the lean increasing over time. This pier wall contains diagonal and vertical cracks open to 5 mm (0.2 inches). An unbalanced fill height of about 4 feet is present on the west side of this pier wall.

B.8. Discussion

B.8.a. General

Bridges up and down the Red River Valley have a history of showing distress due to the geotechnical conditions present throughout this region. Due to the soft, normally consolidated clays deposited by Glacial Lake Agassiz, slope instability is a consistent concern for new and existing structures that bear within these soils. Based on work done at TH171, TH175 and TH1, slope instability appears to be most prevalent on the cutslope of a bend in the Red River. Slope movement often occurs when water levels in the Red River are low. Low river levels can produce a rapid drawdown effect, due to the slow drainage characteristics of the Lake Agassiz clays, which in turn creates a different force equilibrium to that established thousands of years ago when Lake Agassiz drained and the Red River was formed. Within Kennedy Bridge, Pier 6, along with Bents 2 through 5, appear to be showing movement and distress due to slope instability, consistent with the bridges to the north mentioned previously.

B.8.b. Driven Piles

Foundations for bridge support in the Red River Valley typically consist of driven pile foundations. Because bedrock is hundreds of feet deep in the basin, driven piles typically extend through the lacustrine clay and penetrate down into the glacial till, where they develop substantial skin resistance and end bearing. Based on the available boring information, the glacial till bearing layer is generally found at about elevation 690 to 685 feet in Kennedy Bridge area. Generally, driven pile foundations installed in this area consist of the following:

- H-piles consisting of 10x42, 12x53, and 14x73 sections, which are used more often if cobbles or gravelly layers are encountered below the lacustrine clay at the top of the glacial till
- 9⅝-inch, 12-inch, or 16-inch outside-diameter (OD) closed-end pipe piles filled with concrete (termed CIP piles by MnDOT and NDDOT)

- Larger diameter driven piles (42-inch OD) have also been used successfully in cases of very high lateral and/or axial loads or where pile cap size needs to be minimized.

In this area, the installation of driven pile foundations generally involves less risk, time, and expense than the other deep foundation systems discussed below.

B.8.c. Drilled Shafts

Unless very large lateral loads are expected, drilled shafts have not proven to be a practical deep foundation system for bridge support in the Red River Valley. Drilled shafts will need to be either fully cased or slurry drilled. The shafts would need to extend several shaft diameters into the glacial till bearing layer, which would result in shaft lengths of about 120 to 160 feet. Drilled shafts of this length will be very expensive. In addition, a single shaft could take 3 to 5 days to construct. Finally, constructing drilled shafts in these conditions carries a much higher risk of construction delays than installing driven pile.

B.8.d. Micropile

Micropile foundations constructed using 12-inch or less diameter pipe casing, drilled with a sacrificial bit, reinforced internally with a large diameter steel bar, and filled with high strength grout under pressure are an option for bridge support. However, micropiles are more expensive than driven piles under typical installation conditions. Therefore, the use of micropiles is usually limited to low head room conditions. Further details on potential micropile foundations for rehabilitation of the existing substructures are provided in Section C.2.

B.8.e. Constructability

From a construction perspective, the project team should also be aware that:

- Excavations will penetrate the groundwater surface at variable depths depending on location. The presence of groundwater coupled with the soft clays at this site may cause drilled shafts or micropile to cave in; therefore, provisions should be made to case drilled shafts and micropile.
- The clays present at the site will need to be dried to facilitate compaction. The thickness of clay placed will also have to be restricted to limit the amount of post-construction settlement that occurs from the clay fill compressing under its own weight.
- Because there are no such resources on the site, sands or gravels will have to be imported to backfill the balance of fills that can only be partially backfilled with clay as well as to facilitate drainage behind below-grade walls and below pavements.

- Haul roads and staging areas will be particularly sensitive to disturbance and strength loss. Subexcavation and recompaction or replacement of subgrade soils can be limited if these traffic areas are protected with crushed rock.

B.8.f. Cost Comparison

On the basis of cost per ton of Factored Design Load (FDL) supported in compression, it is our opinion that a driven HP 14x73 H-pile is likely the most cost effective deep foundation for this site. Cost comparisons for various driven pile sections and two different micropile installation scenarios, one with normal headroom and one with limited headroom, are provided in the table below. The comparison is provided as the cost per ton of FDL in compression shown as a percentage relative to the existing BP14x73 driven pile. These estimates assume the pile sections are driven to their assumed structural capacities, to support FDL ranging from 115 to 240 tons, and the micropiles are installed to support a FDL of 85 tons. These estimates also assume a top-of-pile elevation of 810 feet and the top of the bearing layer at elevation 690 feet. The estimated deep foundation bottom elevations range from 677 to 664 feet, resulting in pile lengths of 133 to 146 feet, which would typically require three splices per driven pile. Drilled shafts of this length, although possible, are not typically installed in our region. Therefore, we were unable to obtain reliable costs comparisons for this deep foundation alternative.

Table 7. Deep Foundation Cost per ton of Compressive FDL Relative to Existing BP14x73

Foundation Type	Size or Section	Relative Cost/Ton
Driven H-Pile	HP 14x73	100%
Driven H-Pile	HP 12x53	110%
Driven CIP Pile	9⅝-inch OD, 0.352-inch wall CEP	130%
Driven CIP Pile	12.0-inch OD, 0.3125-inch wall CEP	140%
Driven CIP Pile	16.0-inch OD, 0.3125-inch wall CEP	160%
Micropile-normal headroom installation	9⅝-inch casing to elevation 690, 8-inch diameter below with bond length of 15 feet	440%
Micropile- low headroom installation	9⅝-inch casing to elevation 690, 8-inch diameter below with bond length of 15 feet	550%

C. Pier 6 Rehabilitation (Task 5.3)

C.1. Existing Piles

If the existing H-piles are structurally adequate to support the factored design loads, they could be reused. However, the design loads must consider additional loads caused by the slope movement and foundation tilting that has occurred, which may reduce the existing piles' load carrying capacity. An evaluation of this magnitude was beyond the scope of our work.

Based on Article 6.5.4.2 of AASHTO's LRFD Bridge Design Specifications, 2012, assuming 50 ksi steel and hard driving conditions, the factored design load for the HP 14x73 section should be less than 268 tons. MnDOT typically limits this section to 240 tons. However, based on the age of the bridge, the H-pile steel is likely 36 ksi steel; therefore, the existing piles likely have a lower structural capacity.

We used the computer program, DRIVEN, v1.2, to estimate the nominal (ultimate) static geotechnical vertical compressive resistance for the 14x73 H-pile section. DRIVEN is a static pile analysis software program developed by the Federal Highway Administration. Within Driven, we used adhesion values obtained from instrumented pile load tests performed in the Fargo Area in the same geologic formations present at this site. The results of the DRIVEN analysis are plotted with a heavy blue solid line in the graphs titled Nominal Geotechnical Resistance for Driven 14x73 H-pile located in the Appendix. These graphs show nominal capacity in tons versus elevation in feet. The second graph is simply a zoomed in version of the first graph.

On these graphs, we have also plotted two points each from the TP-2 static load tests performed at Pier 6. Test #1 results are indicated by a purple **X**, and Test #2 results are indicated by a green **O**. The first (leftmost) point in each set corresponds to the applied load and pile toe elevation during the last load increment prior to plunging during the load test, which is assumed herein to be a "safe" load. The second point in each set corresponds to the subsequent applied load and pile toe elevation when plunging occurred during the load test; assumed herein to be the "plunge" load.

We have used this data to create linear projections of plunge and safe load versus elevation. The plunge load projection is shown with a red dotted line, and the safe load projection is shown with a thin green solid line. Finally, the average pile toe elevation of the production pile at Pier 6 is plotted with a thick black solid line.

Factored design loads for the pile should be less than the nominal pile resistance (R_n) multiplied by the appropriate resistance factor (Φ). The American Association of State Highway and

Transportation Officials (AASHTO) recommend relating the resistance factor to the degree of construction control. We recommend using a resistance factor of 0.75, based on the assumptions that the production pile driving criteria for Pier 6 was established by utilizing the successful static load test performed at Pier 6 and that no dynamic testing was performed.

In our opinion, the load corresponding to the intersection of the thick black solid line (average pile toe elevation of the production pile at Pier 6) and the thin green solid line (linear projection of safe load) on the attached graphs (275 tons) is a nominal geotechnical pile resistance that should result in pile settlement of less than 1-inch for the existing pile. Applying a resistance factor of 0.75 to the 275 ton nominal resistance, the factored resistance for the pile at Pier 6 should be less than 206 tons (412 kips).

The following assumptions were made in our analyses of geotechnical compression resistance for the existing 14x73 H-pile at Pier 6:

- Soil profile information based on Boring No. 8 in the 1962 bridge plans and the Pile Load Bearing Test Report.
- Bottom of pile cap elevation of 801.89.
- Vertical pile orientation.
- Pile skin friction based on a box perimeter for the H-pile.
- Pile end bearing based on the H-pile steel area.
- Pier 6 Test Pile TP-2 bottom elevation of 686.50 for Load Test #1.
- Pier 6 Test Pile TP-2 bottom elevation of 685.23 for Load Test #2.
- Average production pile length of 119.37 feet (average pile toe elevation of 682.52).

C.2. Micropile Foundations

New deep foundations could consist of the types discussed in Section B.8; however, based on the limited headroom of about 10 feet available during rehabilitation of the substructures, we presume micropile will be the most cost effective foundation type for rehabilitating the existing substructures.

We recommend that the micropile contractor construct the micropile using overburden drilling techniques that involve a casing to reach the top of the glacial till bearing layer at about elevation 690. At this elevation, the micropile can be advanced with temporary casing or open-hole drilling techniques. Due to the great depth of rather soft soils, we recommend the casing for the micropiles be permanent. We evaluated micropile bond lengths within the till below elevation 690 based on Type B or E micropile with grout to ground bond nominal resistances based on the presumptive values in Table C10.9.3.5.2-1 of the AASHTO LRFD Bridge Design Specifications, 2012 and our experience.

For Factored Design Loads (FDL) of 85 tons (170 kips) and 9⅝-inch casing to elevation 690 with an 8-inch micropile diameter below 690, we estimate bond lengths of 10 to 20 feet will be required. For FDL of 200 kips and a 12.0-inch casing to elevation 690 with 11-inch micropile diameter below 690, we estimate bond lengths of 9 to 17 feet will be required. These estimates ignore skin friction along the cased section and end bearing. These estimates are based on a resistance factor of 0.55.

C.3. Slope Stability Remediation

Whether the existing piles are reused or new foundations are installed, the slope instability needs to be remediated or considered in the deep foundation design. There are generally two methods to stabilize slope movement: (1) reduce the driving force and (2) increase the resistance along the failure surface.

Flattening the slope from Abutment 1 to Pier 6 may be an option in order to reduce the driving force. However, based on the available information, it appears that reducing the existing slope will be a complex issue. Major concerns include the following:

- Maintaining the existing TH2 approach embankment.
- Maintaining flood protection embankments and existing structures.

Lightweight fill could be utilized in the approach embankments in order to reduce the driving force. A variety of materials are available including: Geofoam blocks, lightweight cellular concrete, lightweight aggregate, tire derived aggregate (shredded or chips), and wood chips. Each has unique engineering properties, advantages, disadvantages, and cost implications.

Options available to increase the resistance along the failure surface include the following:

- Vertical pinning with structural elements such as driven piles or drilled shafts.
- Diagonal pinning with soil nails or similar elements.
- Drainage of the failure area to decrease pore water pressure and increase effective strength of the soil.
- Chemical strengthening the soil near the failure surface.
- Managing the water levels in the Red River.

C.4. Recommendations for Additional Investigation and Testing

Additional subsurface investigation, to better define the soil properties; pore water pressures; and the specific slope failure mode, depth, and location, is critical in order to formulate a comprehensive plan for remediation. Our recommendations for additional investigation include Standard Penetration Test (SPT) borings, Cone Penetration Test (CPT) soundings, laboratory testing, piezometers, and slope inclinometers.

C.4.a. Subsurface Investigation

In general, we recommend foundation borings and soundings be performed per MnDOT's most recent Consultant Specifications for Subsurface Investigation & Geotechnical Analysis and Design recommendations, which can be found at
http://www.mrr.dot.state.mn.us/geotechnical/foundations/tcontract.asp

In general, the laboratory soil testing should also follow these specifications. However, we recommend laboratory soil testing also include a minimum of 4 triaxial compression tests, consolidated undrained method with pore water pressure measurement, to determine soil strength parameters within the shear zones.

C.4.b. Geotechnical Instrumentation

A line of instrumentation including vibrating wire piezometers and slope inclinometer casings should be installed on each side of (north and south of) the Kennedy Bridge on the west bank. Each section should include a minimum of three installations located at the crest, midslope, and near the toe of the slope. At each location, a shallow and deep vibrating wire piezometer and an inclinometer casing should be installed. We recommend the inclinometer casings extend at least 100 feet below the surface. We recommend the vibrating wire piezometers be installed at appropriate depths based on the results of drilling for the inclinometer casings.

C.4.c. Micropile Test Program

We recommend performing an instrumented static load test to verify the presumptive grout to ground bond nominal resistances assumed for micropile design and the geotechnical resistance of the micropile. It is our opinion one static load test should be sufficient for rehabilitation of the substructures located on the west bank.

D. New Bridge Support (Task 6.1)

D.1. General Foundation Concepts

The foundation options discussed in Section B.8 provide a background for consideration of foundation types for a new bridge. Further details regarding micropiles were discussed in Section C.2. More information regarding driven pile and drilled shafts are provided in the following sections. In our opinion, a driven pile is likely the most cost effective foundation for support of the bridge at this site.

D.1.a. Driven Piles

For pipe piles, we recommend ASTM A252 Grade 3 steel having a yield strength of at least 45 kips per square inch (ksi). For H-piles, we recommend ASTM A572 Grade 50 steel, which has a yield strength of at least 50 ksi.

In our opinion, the working capacities of piles spaced at least 3 pile diameters apart need not be reduced due to group effects. If a closer spacing is ultimately selected, we recommend having a geotechnical engineer evaluate the magnitude of the group effect and the extent to which the working capacities should be reduced.

Using an under- or over-sized pile-driving hammer can be detrimental to the successful installation of piling. Therefore, prior to system acceptance, we recommend performing a wave equation analysis modeling prospective contractors' pile installation systems. The wave equation analysis is used to estimate probable driving stresses and pile penetration resistance based on the type of hammer proposed, the specified pile type/size and the site-specific material conditions which, when combined, help evaluate system suitability. Our firm can discuss the requirements and limitations of wave equation analyses and, if needed, perform them.

D.1.b. Drilled Shafts

As the shafts are drilled, obstructions may be encountered that cannot be removed with conventional drilling equipment. In our opinion, an obstruction can be considered to consist of a dense concentration of cobbles, boulders, detached rock slabs, or other material, either natural or man-made, that impedes drilling with conventional augers and requires special equipment, including but not necessarily limited to core barrels, air compressors, or hand excavation tools to penetrate. The obstruction can be considered to have been penetrated once conventional augering can resume.

The presence of in-place rock which must be removed with special equipment in order to attain the required embedment does not, in our opinion, qualify as an obstruction, but should instead be considered to fall under the category of rock removal.

D.2. Outline of Potential Pre-Design Investigation and Testing

For a new bridge, we recommend subsurface investigation include Standard Penetration Test (SPT) borings, Cone Penetration Test (CPT) soundings, laboratory testing, piezometers, and slope inclinometers.

D.2.a. Subsurface Investigation

We recommend foundation borings, soundings, and laboratory testing be performed per MnDOT's most recent Consultant Specifications for Subsurface Investigation & Geotechnical Analysis and Design recommendations. In general, the laboratory soil testing should also follow these specifications. However, we also recommend laboratory soil testing which includes a minimum of 6 triaxial compression tests, consolidated undrained method with pore water pressure measurements, to determine soil strength parameters within potential shear zones at the approach embankments.

D.2.b. Geotechnical Instrumentation

A line of instrumentation, including vibrating wire piezometers and slope inclinometer casings, should be installed along each bank of the river at the proposed bridge location. Each line should include a minimum of three installations located at the crest, midslope, and near the toe of the slope. At each installation, a shallow and deep vibrating wire piezometer and an inclinometer casing should be installed. We recommend the inclinometer casings extend to about elevation 680 feet. We recommend the vibrating wire piezometers be installed at appropriate depths based on the results of drilling for the inclinometer casings.

D.2.c. Deep Foundation Test Program

For driven pile, we recommend performing a test pile program consisting of dynamically monitoring at least one test pile per substructure, in general accordance with ASTM International D 4945. Data from dynamic testing should be used to formulate a driving/length criterion by which the remainder of the pile should be driven.

If static load testing will be used for deep foundation quality control, which we recommend for micropile and/or drilled shaft foundations, it is our opinion two static load tests should be performed for the new bridge, one on each side of the channel.

E. Qualifications

E.1. Variations in Subsurface Conditions

E.1.a. Material Strata

We did not perform a subsurface investigation or provide monitoring of geotechnical instrumentation for preparation of this report. Our evaluation, analyses, and recommendations were developed from a limited amount of site and historical subsurface information. It is not standard engineering practice to retrieve material samples from exploration locations continuously with depth; therefore, strata boundaries and thicknesses must be inferred to some extent. Strata boundaries may also be gradual transitions, and subsurface strata can vary in depth, elevation, and thickness away from the exploration locations.

Variations in subsurface conditions present between exploration locations may not be revealed until additional exploration work is completed or construction commences. If any such variations are revealed, our recommendations should be re-evaluated. Subsurface variations could increase construction costs, and a contingency should be provided to accommodate such variations.

E.1.b. Groundwater Levels

The groundwater levels discussed herein were taken by others and provided within their documentation. It should be noted that in some cases, such as the boring groundwater levels, the observation periods may have been relatively short; therefore, the measured levels may not represent static groundwater levels. Groundwater can be expected to fluctuate in response to rainfall, flooding, irrigation, seasonal freezing/thawing, surface drainage modifications, as well as other seasonal and annual factors.

E.2. Continuity of Professional Responsibility

E.2.a. Plan Review

This report is based on a limited amount of information, and a number of assumptions were necessary to help us develop our recommendations. It is recommended that our firm review the geotechnical aspects of the designs and specifications, in order to evaluate (1) whether the design is as expected, (2) if any design changes have affected the validity of our recommendations, and (3) if our recommendations have been correctly interpreted and implemented.

E.2.b. Additional Services

It is recommended that we be retained to perform additional subsurface investigation and geotechnical monitoring prior to final design, as well as construction observations and testing during construction. This will allow correlation of the subsurface conditions and provide continuity of professional responsibility.

E.3. Use of Report

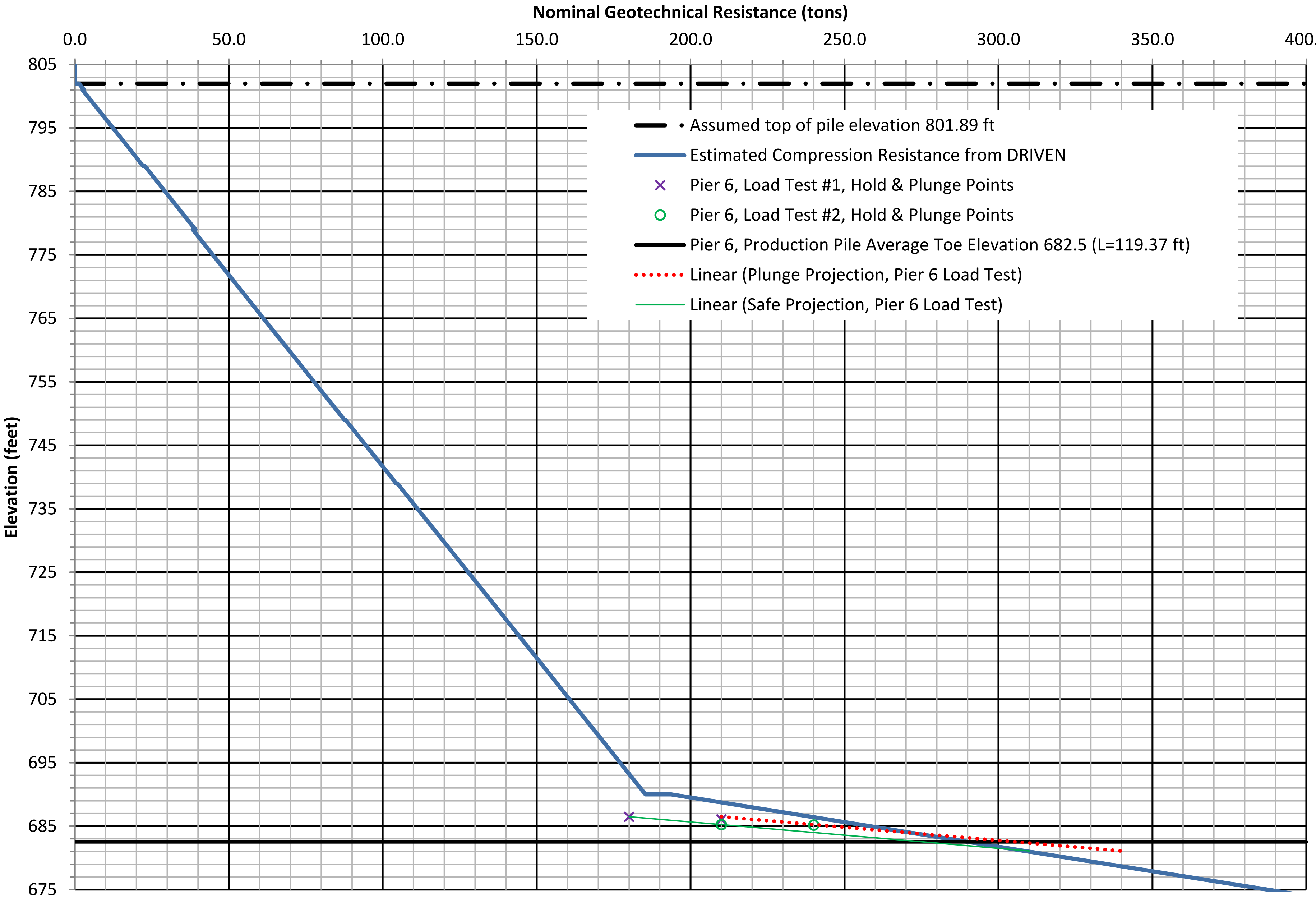
This report is for the exclusive use of the parties to which it has been addressed. Without written approval, we assume no responsibility to other parties regarding this report. Our evaluation, analyses, and recommendations may not be appropriate for other parties or projects.

E.4. Standard of Care

In performing its services, Braun Intertec used that degree of care and skill ordinarily exercised under similar circumstances by reputable members of its profession currently practicing in the same locality. No warranty, express or implied, is made.

Appendix

Nominal Geotechnical Resistance for Driven 14x73 H-Pile
Pier 6, Boring B-8



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Pier 6, Boring B-8