Rehabilitation Report for the

John F. Kennedy Memorial Bridge

FINAL REPORT

MnDOT Bridge No. 9090/NDDOT Bridge No. 02-350.220
over the Red River in Grand Forks, ND & East Grand Forks, MN

Prepared for

MINNESOTA DEPARTMENT OF TRANSPORTATION

October 2015

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GEMINI RESEARCH
Cultural Resource Consultants

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Rehabilitation Report

John F. Kennedy Memorial Bridge

Prepared for

Minnesota Department of Transportation

Final Report
October, 23, 2015

Matthew J. Cramer, PE
Project Manager

Jamison Beisswenger, PE
Lead Bridge Engineer

Susan Granger
Bridge Historian

I hereby certify that this 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.

Jamison Beisswenger, PE
License No. 44648

10/23/2015

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INTRODUCTION

The Minnesota Department of Transportation (MnDOT), in partnership with the North Dakota Department of Transportation (NDDOT), proposes to rehabilitate the John F. Kennedy Memorial Bridge (Kennedy Bridge) over the Red River of the North (MnDOT Bridge No. 9090; NDDOT Bridge No. 02-350.220). The bridge was built in 1963 and carries US Trunk Highway 2 (US2) between Grand Forks, North Dakota and East Grand Forks, Minnesota.

The Kennedy Bridge is eligible for listing on the National Register of Historic Places, thereby meeting the definition of a "historic" property under Section 106 of the National Historic Preservation Act of 1966 and Section 4(f) of the Transportation Act of 1966. Section 106 requires that federally-funded projects take historic properties into consideration during planning and implementation. Under Section 4(f) of the Transportation Act, a federally-funded transportation project cannot "use" a historic property unless there is no prudent and feasible alternative to the use and the undertaking includes all possible planning to minimize harm. Both laws require the project to take into consideration both the historic Kennedy Bridge and other historic properties within the area of potential effect.

The purpose of this rehabilitation study is to summarize the bridge's historic background, significance, and current condition, and explore an approach for rehabilitating the Kennedy Bridge in a manner consistent with the Secretary of the Interior's Standards for the Treatment for Historic Properties (36 CFR Part 68).

Figure 1. U.S. Trunk Highway 2/John F. Kennedy Memorial Bridge (Looking Southeast)

A proposed rehabilitation of the Kennedy Bridge is consistent with MnDOT's *Management Plan for Historic Bridges* (2006) which states that the preferred option for the treatment of an historic bridge is rehabilitation for continued vehicular use on-site, with the rehabilitation following the Secretary of the Interior's Standards for the Treatment of Historic Properties. (See Appendix D for the Rehabilitation Standards.)

The *Management Plan* arises from a Section 106 Programmatic Agreement (PA) on Historic Bridges in Minnesota that was finalized in 2008 by MnDOT, FHWA, the MnSHPO, and other signatories. Among its provisions, the PA encourages historic bridge projects to explore context-sensitive solutions during project planning, including the use of tools such as design exceptions, where practical, to help preserve a bridge's historic integrity.

MnDOT is serving as the lead state agency for the Kennedy Bridge project and, correspondingly, the MnDOT Cultural Resources Unit (MnDOT CRU) is responsible for carrying out FHWA's Section 106 review role under the terms of the above-referenced PA. Section 106 consulting parties include the MnSHPO, the North Dakota State Historic Preservation Office, the Grand Forks Historic Preservation Commission, and NDDOT's Cultural Resources Section.

In 2013, MnDOT led a study to develop an approach to maintain the US2 crossing of the Red River of the North. This study compiled background and supporting documents, laid out the setting and the context for the project, established alternatives, and conducted a comparison of these alternatives. A report documenting the findings of this study entitled *US Trunk Highway 2 Red River Crossing (Kennedy Bridge) Planning Study* (Planning Study) was completed by CH2M Hill and can be found in Appendix E.

The planning work included the formation of a Study Advisory Committee which conducted two rounds of public outreach and held meetings with, or sought input from, the following stakeholders:

- MnDOT
- NDDOT
- Federal Highway Administration (FHWA)
- City of Grand Forks, ND (Engineering)
- City of East Grand Forks, MN (Dept. of Public Works)
- Polk County, MN
- Grand Forks County, ND
- Grand Forks-East Grand Forks Metropolitan Planning Organization (GF-EGF MPO)
- Grand Forks Historic Preservation Commission
- U.S. Fish & Wildlife Service
- U.S. Army Corps of Engineers
- U.S. Environmental Protection Agency
- Minnesota Department of Natural Resources
- Minnesota Pollution Control Agency
- North Dakota Game & Fish Department
- North Dakota Department of Health
- Minnesota State Historic Preservation Office
- North Dakota State Historic Preservation Office

Input from these stakeholders was elicited during the writing of this report and will continue to be sought during the final plan development process.

1.0 PROJECT PURPOSE AND NEED

The project Purpose and Need is defined in the project's Categorical Exclusion (CATEX) document. The CATEX document gives information regarding functional, maintenance, and geometric needs; current and projected non-motorized recreation use; and how traffic demands change during flooding as nearby bridges close. The following is a summary of the Purpose and Need as defined by the CATEX document:

1.1 Purpose/Objectives

The primary purpose of this project is to provide a structurally sound U.S. Trunk Highway 2 (US2) crossing over the Red River of the North between Grand Forks, North Dakota and East Grand Forks, Minnesota. The project has additional needs due to the role of US 2 in the project area transportation system. These needs include providing for non-motorized crossings of the Red River, maintaining the continuity of US 2 between Grand Forks and East Grand Forks, and maximizing maintenance of traffic during construction.

1.2 Need/Deficiencies

The existing bridge is classified as structurally deficient as soil movement has decreased the capacity of the pier adjacent to the river's west bank. Additionally, deck deterioration is creating an unsound driving surface, steel girders and beams were designed for smaller vehicles, and failure of the paint has resulted in steel corrosion.

Grand Forks and East Grand Forks currently have 46 miles of paved pedestrian/bicycle trails that traverse both cities. The distance between two, existing trail connections over the Red River is approximately 4 miles along the Grand Forks side of the Red River, and approximately 4.5 miles along the East Grand Forks side of the Red River. The current 2.5-foot wide curb located on both sides of the bridge does not meet current standards for pedestrian or combined bicycle/pedestrian use. The Grand Forks/East Grand Forks Metropolitan Planning Organization has identified a desire for improved pedestrian access, mobility, and connectivity to existing Red River crossings.

2.0 BRIDGE HISTORICAL BACKGROUND AND SIGNIFICANCE

2.1 Background

The Kennedy Bridge, first known as the Skidmore Avenue Bridge, was built in 1963 as a cooperative project of the North Dakota and Minnesota state highway departments, with North Dakota taking the lead. It was built with a mix of state and federal funds. The bridge was designed by the NDDOT's Bridge Division under the direction of state bridge engineer Joseph R. Kirby. The contractor was Walter D. Giertsen Company of Minneapolis and steel components were fabricated by the St. Paul Foundry and Manufacturing Company.

Construction of the Kennedy Bridge was part of a major undertaking in the Grand Forks area that involved establishing a new river crossing north of the two downtowns, significant highway realignment and reconstruction, and relocation of the Grand Forks airport. US 2 was a major

corridor linking Grand Forks and East Grand Forks with cities such as Minot and Williston to the west and Duluth to the east. The 1963-1964 project shifted US 2 from the central business districts of the two cities, where it crossed the Red River on the Sorlie Bridge at Demers Avenue, to a new river crossing at Skidmore Avenue (now Gateway Drive) about eight blocks to the north. The Sorlie Bridge route became the US 2 business route while the US 2 main line was rerouted through the northern part of the two cities. In East Grand Forks the US 2 realignment required road reconstruction from the east edge of the city to the new bridge. In Grand Forks, Skidmore Avenue was widened to four lanes from the Red River west to a point about five miles west of the city limits. Here the highway would serve three new facilities: a new Grand Forks International Airport under construction, the six-year-old Grand Forks Air Force Base, and a future Interstate 29 scheduled to be built on a north-south alignment past the new airport.

The bridge, highway, and airport improvements occurred during a post-World War II expansionary period in the area's history when employment opportunities, housing, hospitals, schools and colleges, and shopping centers were built or expanded. In 1960, Grand Forks was the second-largest city in North Dakota with 34,400 people. In addition, several thousand lived at the Grand Forks Air Force Base west of the city and nearly 7,000 people lived in East Grand Forks. (Today Grand Forks is North Dakota's third-largest city.)

The Skidmore Avenue Bridge opened on November 15, 1963. A week later President John F. Kennedy, who had visited Grand Forks in September 1963 to deliver a speech at the University of North Dakota (UND), was assassinated. Several months later the bridge was renamed the John Fitzgerald Kennedy Memorial Bridge in his honor.

The Kennedy Bridge was the metropolitan area's third and northernmost Red River crossing. In the center of the cities, the first permanent bridge at Demers Avenue was built in 1888; the current Sorlie Bridge was built in 1929. The southernmost crossing is at Minnesota Point (now Minnesota Avenue) where the current bridge dates from 1967.

Designer and Builders

Joseph R. Kirby (1904-1991), lead designer for the Kennedy Bridge, worked for NDDOT from 1926-1969 and was state bridge engineer from 1955-1969. He was a 1959 recipient of the Elwyn F. Chandler Award for engineering achievement and in 1999 was inducted into the North Dakota Highway Hall of Honor. Kirby was responsible for the design of many of North Dakota's largest and most notable bridges of the postwar period.

The Walter D. Giertsen Company, contractor for the Kennedy Bridge, was established in Minneapolis in 1918 and entered the field of bridge and heavy road construction in the 1950s. The Giertsen Company built bridges and highways throughout the Midwest, including several segments of the interstate highway system. When the Kennedy Bridge was built in the early 1960s, the company was headed by Richard W. Giertsen, son of the founder.

2.2 National Register Eligibility

The Kennedy Bridge, a Parker through truss bridge built in 1963, has been determined eligible for the National Register by the FHWA and both the Minnesota State Historic Preservation Office and the North Dakota State Historic Preservation Office have agreed with this determination.

The Kennedy Bridge is eligible for the National Register of Historic Places under Criterion C (design and construction) in the area of Engineering, and under Criterion A (broad patterns of history) in the area of Transportation.

In Minnesota, the bridge was determined eligible for the National Register as part of a statewide evaluation of post-1955 highway bridges directed by MnDOT in 2010-2011 (Mead and Hunt 2011). The Kennedy Bridge is one of 8 Minnesota bridges from the period 1955-1970 that were recommended by the study as being eligible for the National Register under Criterion A, and one of 29 bridges recommended as being eligible under Criterion C.

The Kennedy Bridge's engineering significance within the context of Minnesota bridges lies in its exceptional main span length for this type of truss. While most steel Parker through truss bridges are 40 to 250 feet long, the Kennedy Bridge's trusses are 279 feet long representing the upper limits of span length for this bridge type.

The bridge's transportation significance in Minnesota is based on the role it played in the expansion of a regional transportation network that helped facilitate economic development in northwestern Minnesota. The bridge was built as part of a major modernization of the state trunk highway system in 1956-1970 that increased capacity and introduced controlled-access highway design principles to non-interstate routes (Mead and Hunt 2011:41).

In North Dakota, the bridge was determined eligible for the National Register within the context of a 2004 update to North Dakota's 1991 statewide historic bridge study (Hufstetler and Goff 2004). The Kennedy Bridge meets the 2004 bridge study's registration requirements for National Register Criterion A by being an important crossing of a major waterway, and for Criterion C as an increasingly rare example of a Parker through truss structure. The Kennedy Bridge has the second-longest Parker trusses in North Dakota. (The Sorlie Bridge has the longest Parker trusses at 283 feet.) The Kennedy Bridge possesses additional engineering significance for its unique design in response to the challenging soil conditions of the Red River Valley. (See Description below.) It was the last steel through truss bridge built in North Dakota.

In Minnesota, the Kennedy Bridge was determined to have a state level of significance on the National Register's local, state, national scale (Mead and Hunt 2011). Gemini Research recommends that the period of significance is the year of completion, 1963. (The level and period of significance have not been determined in North Dakota.) Gemini Research recommends that the boundary of the National Register eligible property is a rectangle that measures approximately 100 feet north-south by 1,300 feet east west (see Figure 2).

2.3 Other Historic Properties

Under Section 106 and similar statues, work on the Kennedy Bridge must take into consideration potential effects to adjacent historic properties. "Historic" properties are generally defined by applicable laws as sites or districts listed on, or eligible for, the National Register of Historic Places.

There are three historic properties in close proximity to the Kennedy Bridge, all on the west bank of the river in Grand Forks (Figure 2). Archaeological resources are also addressed below.

Figure 2. Historic Property Boundaries

St. Michael's Hospital and Nurses' Residence

St. Michael's Hospital and Nurses' Residence (Figure 3) is located about 100 feet south of the west end of the bridge. The bridge is a substantial element in its setting and is highly visible from the property. St Michael's Hospital and Nurses' Residence was listed on the National Register under Criterion A (broad patterns of history) and Criterion C (architecture). The property is Grand Forks' most intact early hospital facility and includes St. Michael's Hospital, built in 1907, and a dormitory for nurses, built in 1913 in association with the hospital's school of nursing. Both buildings are locally significant examples of the Classical Revival style. The facility served as a hospital until 1952, was a nursing home until 1981, and in 1995 was rehabilitated for apartments. A flood wall was built along the east side of the property circa 2007.

Figure 3. St. Michael's Hospital and Nurses' Residence with a segment of Granitoid Pavement in the foreground (Looking Northwest – Bridge 9090 is at center right)

R. S. Blome Granitoid Pavement, Lewis Boulevard Segment

The Lewis Boulevard Segment of the R. S. Blome Granitoid Pavement consists of two areas of Granitoid pavement located on the former alignment of Lewis Boulevard, a city street on the west bank of the river that ran under span 2 near the west end of the Kennedy Bridge. The first section of Granitoid Pavement is roughly three blocks long and located about 250 feet north of the west end of the bridge. The second section is a roughly one block long and located about 120 feet south of the west end of the bridge (both sections are labeled on Figure 2).

The Lewis Boulevard Segment of Granitoid pavement is one of three areas of Granitoid pavement in Grand Forks, totaling more than 30 linear blocks, which together were listed on the National Register in 1991. The boundaries were revised in 2009 because of loss of integrity in some areas (Figure 2 shows the segments near the bridge after the boundary revision). The Granitoid pavement was listed on the National Register under Criterion A for its role in Grand Forks' transportation history and under Criterion C as a distinctive and nationally rare paving type. Granitoid is a patented mixture of Portland cement and crushed granite that was scored on top to resemble stone blocks and to provide traction. Installed in Grand Forks in 1910-1911, the attractive and durable material was designed to accommodate both horses and autos at a time when society was making the transition between these two modes of transportation. It was installed when most of Grand Forks' streets were still unpaved. The portion of pavement north of the bridge is located within the Riverside Neighborhood Historic District (as shown in Figure 4).

Riverside Neighborhood Historic District

Riverside Neighborhood Historic District is located about 100 feet north of the west end of the Kennedy Bridge. The bridge is a substantial element in the setting of the district and the Gateway Drive sidewalk west of the bridge may be within the district boundaries. The bridge is visible from the southeast portion of the district, although oblique views are somewhat obscured by post-1997 flood protection levees. The 112-acre historic district is significant for its concentration of well-preserved examples of late 19th and early 20th century residential architecture. When the district was listed on the National Register in 2007, it contained 119 contributing resources and 54 noncontributing resources. A segment of the Granitoid Pavement (Figure 4) is located in the southeast corner of the historic district.

Figure 4. Riverside Neighborhood Historic District with a segment of Granitoid Pavement in the foreground (Bridge 9090 is at center right)

Archaeological Properties

There are no archaeological concerns within the Kennedy Bridge's existing footprint. A review of archaeological survey work will be needed to rule out potential impacts to archaeology if ground disturbing work is proposed outside of the existing footprint.

3.0 BRIDGE NOMENCLATURE

Throughout this report, elements of the structure are referred to using the technical definitions provided by FHWA's *Bridge Inspector's Reference Manual* found on FHWA's website located at http://www.fhwa.dot.gov/bridge/nbis/pubs/nhi12049.pdf.

3.1 Approach Span Nomenclature

Figure 5 and Figure 6 indicate the location and nomenclature used to identify key elements of the approach spans (spans 1-5 and spans 8-13).

Figure 5. Approach Span Nomenclature

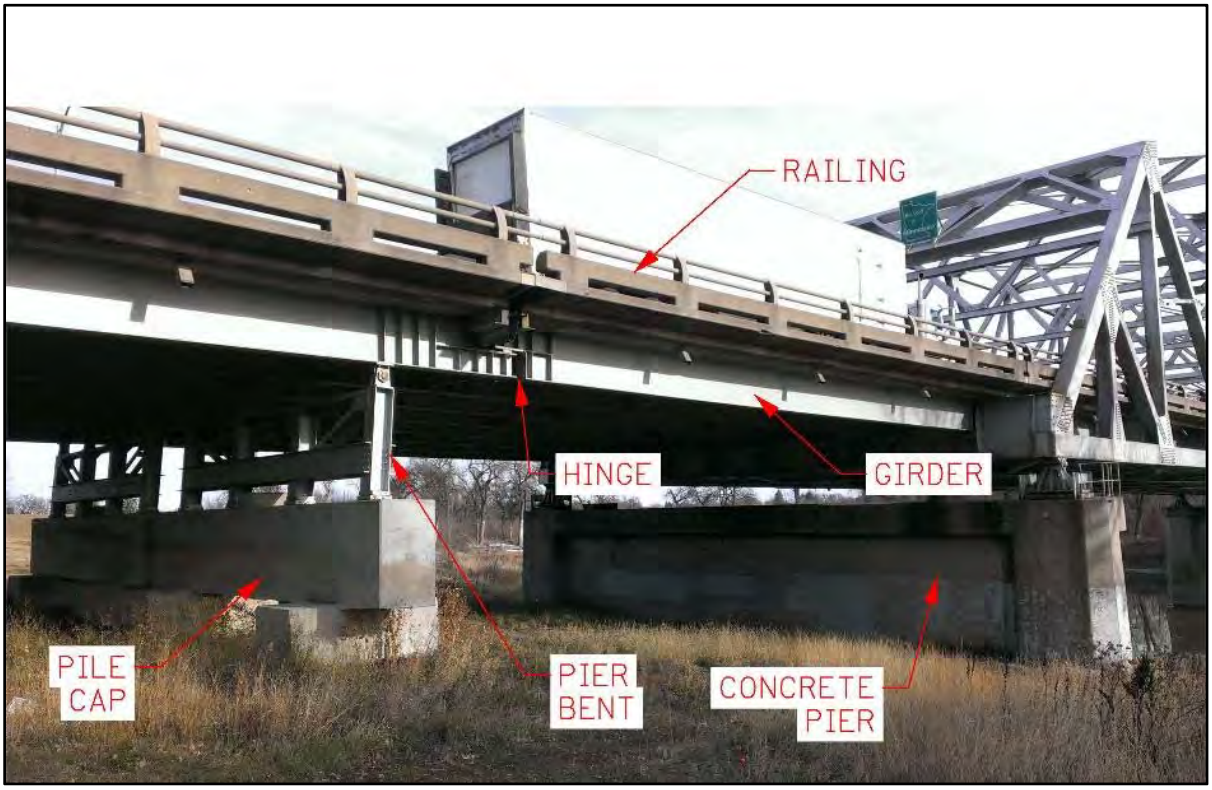
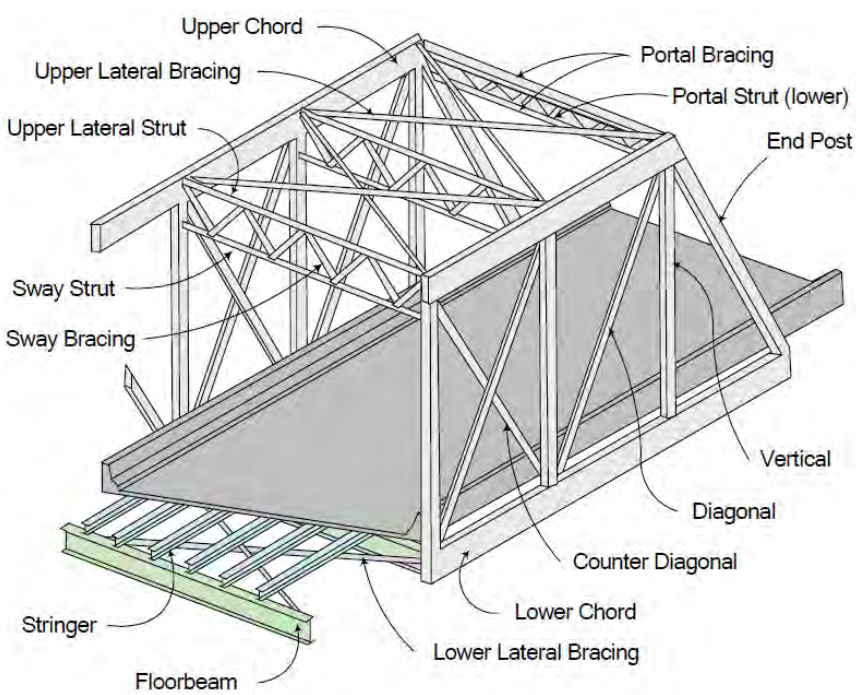


Figure 6. Approach Span Nomenclature

3.2 Truss Span Nomenclature

Figure 7 indicates the general nomenclature used for common elements of the truss spans (spans 6 and 7).

Figure 7. General Truss Nomenclature
(Source: FHWA Bridge Inspector's Reference Manual)



4.0 BRIDGE SETTING, DESCRIPTION, HISTORIC INTEGRITY AND CHARACTER-DEFINING FEATURES

The Kennedy Bridge (MN Bridge 9090/ND Bridge 2-350), built in 1963, carries U.S. Highway 2 (Gateway Drive) over the Red River of the North between East Grand Forks, Minnesota, and Grand Forks, North Dakota. The bridge is jointly owned by the two states.

US 2 is on the National Highway System (NHS) and classified as an urban principal arterial route. It is a four-lane roadway rated as a ten-ton route. The average daily traffic (AADT) is 20,800, with a heavy commercial average daily traffic (HCADT) of 1,770. The posted speed limit is 35 mph. The bridge is closed to pedestrians.

Bridge 9090 is a fracture critical bridge with non-redundant structural design (main spans only). The bridge is not currently posted and is carrying normal traffic. However, substructure movement, deck deterioration, and under capacity pin and hanger assemblies threaten the ability to maintain adequate load capacity. The bridge is currently the only bridge in Grand Forks-East Grand Forks that carries permitted, overweight loads. The closest river crossing without clearance or weight restrictions is located approximately 50 miles south at Halstad. Posting the bridge to prohibit heavy loads would create a lengthy detour for heavy commercial traffic or overweight loads.

During annual spring flooding, the Kennedy Bridge is one of four bridges over the Red River that typically remains open between Moorhead and the Canadian border. If flood waters became high enough, the bridge would likely be the last of the four to close

4.1 Setting

The Red River of the North is a shallow river that is subject to frequent flooding and subsoil movement. Like several of the Red River bridges, the Kennedy Bridge was originally designed to accommodate these conditions, as described below. Flooding and soil movement have contributed to issues of erosion and substructure movement addressed in the proposed rehabilitation.

The bridge is located in an urban setting along Gateway Drive, a major commercial corridor. Nearby are residential properties and parkland, with a commercial/warehouse area a short distance to the southwest. West of the bridge are three historic properties, one of which is a large historic district (see Other Historic Properties in Section 2.3).

The bridge was historically surrounded by a number of residential properties that were removed after the catastrophic 1997 flood. A system of levees and floodwalls was built after the flood, and Lewis Boulevard was removed from under span 2. Land along the river was developed into a linear park known as the Red River Greenway. The park land on the Minnesota side of the river is designated the Red River State Recreation Area and includes a campground southeast of the bridge. As part of the Greenway trail system, a 13-foot wide bituminous recreational trail runs under span 2 and a 10-foot wide trail now runs under span 12 (see Appendix A photos).

Vegetation surrounding the bridge consists of long grasses, mowed turf, and deciduous trees in a natural distribution.

4.2 Bridge Description

The bridge is 1,261 feet long and consists of two 279-foot-long Parker through-truss spans flanked by 11 steel beam approach spans. Spans and substructure elements (piers and abutments) are numbered west to east as shown in Figure 8. There are five approach spans west of the trusses, together measuring 318 feet, and six spans east of the trusses, together measuring 385 feet. The bridge has a gentle horizontal curve in both the west and east approach spans.

Figure 8. Bridge Elevation (Looking North)

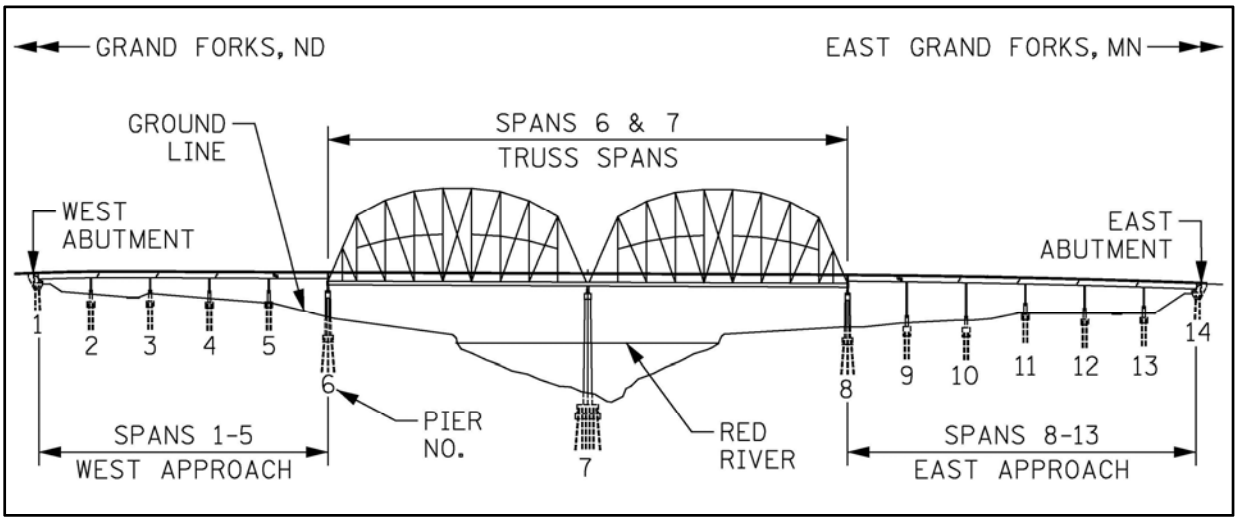


Figure 9. Bridge Truss Spans (Looking Southeast)

4.2.1 Superstructure

The two truss spans are essentially identical structures comprised of nine, 31-foot panels each. The north and south trusses are 69 feet apart, measured center to center and provide approximately 19.8 feet of vertical clearance over the top of the deck. At each panel, a 68-inch floor beam supports eight, 30 inch deep I-beam stringers.

The truss chords consist of box sections that are made from four plates welded together while the vertical and diagonal members are I-beam shapes. Overhead sway bracing consists of a combination of built up box sections and I-beam shapes. The box sections of the truss chords and overhead sway bracing are perforated with 8 inch wide and 16 inch long holes spaced at approximately 3 feet to provide welders access during fabrication (Figure 65 and Figure 66). The truss members are connected with bolted gusset plates of various thicknesses.

Each approach span is constructed of eight, 36 inch deep I-shaped girders separated by 18 inch deep, bolted, channel diaphragms. Adjacent to piers 2-4 and piers 10-13, each girder was constructed with a pin and hanger assembly that allows vertical girder rotations and the transferring of loads between the spans (Figure 42). Adjacent to pier 5 and pier 9, each girder was constructed with a hinge assembly that functions like a pin and hanger assembly, but also accommodates longitudinal movements due to thermal expansion and contraction.

Horizontal swivel assemblies (Figure 44) were constructed below the center of both the eastbound and westbound roadways, adjacent to piers 2-4, 6, 8, and 10-13. These swivel assemblies accommodate transverse substructure movements by allowing the bridge to rotate horizontally as described in Section 9.4 of this report.

At the abutments and at piers 6-8, the bridge is supported on cast steel, rocker style bearings (Figure 51). There are a total of sixteen bearings at the abutments (eight at each abutment) that support each girder. The abutment bearings are 18-inch tall pedestals anchored to the concrete and pinned to each girder. The two bearings that support the north trusses at pier 7 consist of 36-inch tall pedestals anchored to the concrete and pinned to the truss bottom chord. The other six corners of the truss are supported by 32-inch tall pedestals with a curved surface that accommodate bridge movement under thermal expansion/contraction or soil movements (Figure 30).

In 1981 and 1996, both the truss and approach spans were repainted. The structural steel elements of the superstructure retain their historic aluminum paint color.

4.2.2 Deck

The deck was constructed with uncoated concrete with some form lines visible. The deck is supported by the truss stringers and floor beams and approach span girders. The deck is a 7-inch thick, cast-in-place structure reinforced with uncoated steel. It is separated into halves by a longitudinal joint giving it the ability to accommodate transverse substructure movements. In 1984 the upper deck surface was covered with a 2-inch concrete overlay and 12 of the 14 existing finger joints were replaced with strip seal joints (Figure 33).

The deck overhangs consist of 44 inch wide raised concrete curbs that function as safety walkways and support the bridge railing (Figure 10 and Figure 11). The curbs are 11 inches above the deck (reduced to an effective 9 inches by the 2 inches concrete overlay) and the safety walkways are 28 inches wide. The walkways (labeled "sidewalk" on the original plans) were historically used by pedestrians but have been closed for several years because of their narrow width.

Figure 10. Underside of Bridge Deck (East Approach Spans – Looking Southwest)

Figure 11. Bridge Deck (Looking West)

The roadway measures 60 feet between the outside curbs and consists of an original 4-foot concrete median in the center, four 12-foot lanes, two 1-foot inside shoulders, and two 3-foot outside shoulders. The deck drains via scuppers and steel tubes extending from the bottom of the deck. The deck originally had 12 transverse finger joints; 10 of which have been replaced with strip seal joints (Figure 33). The center median, which is bisected by the longitudinal joint (Figure 32), has 9- inch curbs (reduced to 7 inches by the concrete overlay).

4.2.3 Railing

The railing, which is continuous on both approach and truss spans, consists of an aluminum post and pipe system bolted to the top of a concrete parapet. Its late Streamline Moderne design is not unique to the Kennedy Bridge, but was used on bridges of similar age in Minnesota and elsewhere. From many locations the railing is one of the most visually dominant elements of the bridge.

The railing is 39 inches tall measured from the top of the safety walkway. It has 6-3/4 inch openings, both between the lower pipe and the concrete parapet, and between the two pipes.

Figure 12. Bridge Railing (West End of Bridge – Looking Northeast)

The aluminum portion of the rail is 21 inches tall and consists of more than 300 cast aluminum posts joined by pairs of extruded aluminum pipes. The posts are 10 inches wide and 6-1/2 inches long at the base. They curve inward and are aligned over the concrete parapet's posts. The pipes pass through the posts. The upper pipe is 4 inches in outside diameter and the lower is 3-1/2 inches. The aluminum railing is unpainted and has aged to a dull light gray finish.

The concrete parapet is 18 inches tall and consists of a 12-inch by 12-inch beam supported by 2-foot-long posts. The posts are spaced about 8 feet apart, creating a visually bold repeating pattern of 6-foot openings alternating with 2-foot posts. The final 6-foot segment at each end of the bridge has smaller posts and openings.

The railings end with distinctive concrete endposts that are 6 feet long and 12 inches wide. They curve up toward the railing, rising from 27 inches tall at the ends to 41 inches where they meet the aluminum rail. The bridge has no guardrails.

The concrete on the railing and endposts has a smooth, unpainted surface with no distinct form lines.

The existing railing has not been crash-tested in accordance with FHWA standards (NCHRP 350) and does not meet the strength criteria required for bridges on the National Highway System. The raised curb/safety walk is considered a hazard that can cause vehicles to roll over the existing railing (FHWA, 2014)

4.2.4 Substructure

Bridge 9090's substructure consists of reinforced concrete abutments, three concrete piers, and nine steel bents above concrete footings. All substructures are supported by driven steel piles.

The three concrete piers (piers 6 through 8) are located at the ends of the trusses. During ordinary non-flood conditions, only the central pier (pier 7) is in the water. Each pier has a pair of hexagonal columns spaced about 69 feet (center to center). The upstream column of pier 7 (in the river) is slightly wider than the downstream column for extra strength and is protected by a steel ice nose. The columns on piers 6 and 8 have small platforms with safety railings to facilitate bearing adjustment. Extending between each pair of columns is a 42-inch, buried beam element and an 18-inch thick smooth wall. The top of each pier has a 3-foot tall cap that overhangs the adjacent vertical faces by about 21 inches. The piers are similar in appearance to those on other bridges designed by the NDDOT.

Figure 13. Pier 7 (Looking Southeast)

Piers 2-5 and piers 9-13 consist of pairs of steel 'bents' that independently support the eastbound and westbound portions of the bridge deck. Each steel bent consists of four 12-inch I-shaped columns braced by four inch angles supported by a 3 -foot wide, 30-foot long, concrete pile cap. Each column is supported by a curved rocker plate and is attached to each girder using a pinned connection plate that allows the bents to rotate as described in Section 9.6 of this report.

The piers, concrete bent footings, and abutments have generally smooth, unpainted concrete with some board form lines.

4.2.5 Provisions for Substructure Movement

The Kennedy Bridge, like other bridges crossing the Red River, was designed to accommodate the shifting clay soils of the Red River Valley which tend to slowly move the bridge substructures

toward the main river channel. The bridge's provisions for movement are common to bridges crossing the Red River. They comprise a character-defining feature of the Kennedy Bridge and include the following:

Hinged Pier Bents: It was anticipated that, over time, the pile caps supporting the pier bents would move due to the shifting soils. To accommodate this movement, each column in the pier bents was supported on a curved bearing plate and was connected to each girder with a pinned connection plate (Figure 24). This allowed the pier bents to tilt or 'hinge' as the pile caps moved. The pinned connection plate was designed to allow the columns to be disconnected from the girders and readjusted if the tilt became excessive.

Adjustable Truss Bearings: Like the pier bents, it was anticipated that pier 6 and pier 8 would also move towards the river. To accommodate this movement, engineers designed the truss bearings to be adjustable along the bottom chord of the truss. When movements become excessive, the bearings could be disconnected and readjusted. Figure 30 shows the bolts placed along the bottom chord to facilitate this adjustment.

Swivels Assemblies, Pin and Hanger Assemblies, and Deck Joints: To accommodate differential transverse pier movements (movements perpendicular to the bridge centerline), the bridge was constructed with a system of swivels (Figure 44), pin and hanger assemblies (Figure 42), transverse deck joints (Figure 33 and Figure 36) and a longitudinal deck joint (Figure 32). As one pier moves relative to the adjacent piers, the swivels allow each deck segment to rotate horizontally. The pin and hanger assemblies allow each girder to translate relative to each other by opening and closing at the transverse deck joints. Additionally, the longitudinal joint placed along the center of the bridge allows one half of the bridge to translate longitudinally relative to the other half.

Pier 7 Bearings: As with the pier bents, the bearings of pier 7 were designed to accommodate differential substructure movements. The northern bearings are rigidly affixed to pier 7 while the southern bearings were designed to move. This allows the trusses to rotate slightly about pier 7 if the trusses move up- or downstream at piers 6 and 8.

4.2.6 Lighting

There are seven lights on the bridge (Figure 61), all believed to be original lights in original locations. There is a light mounted on the sway bracing of each truss, four pole-mounted lights on the approaches (two on the west approach and two on the east approach), and one light above the trail under the west approach. The poles are mounted on the outside of the deck curb and bridge railing. There are also pole-mounted lights just off the ends of the bridge. The steel poles are a standard design for highway bridges of the era. The cobra-style fixtures may be replacement fixtures that resemble the originals.

4.2.7 Name Plates

Each of the four concrete railing endposts has a rectangular brass bridge plate indicating the associated state bridge number (Figure 40). In addition, a rectangular aluminum plate reading "John Fitzgerald Kennedy Memorial Bridge" is mounted on northeast and southwest truss endposts (Figure 72).

4.3 Historic Integrity

The Kennedy Bridge has experienced only minor alterations or repairs. The following is a summary of the bridge's maintenance history:

1981: The bridge steel was repainted.

1984: Deck was scarified and a 2 inch overlay was added. At the same time, ten of the bridge's twelve finger joints were replaced with strip seal joints and the remaining two finger joints were raised.

1996: Repainted steel elements of the truss spans, truss bearings, 5 feet of approach span girders adjacent to trusses, and 6 feet of approach span girders at the abutments.

2002: The abutment received minor concrete repairs and the approach panels were replaced.

2007: Steel cracks adjacent to tack welds were discoverer on the truss and ground smooth.

After the 1997 flood, Lewis Boulevard was removed from under span 2, recreational trails were built under spans 2 and 12, and the setting was altered as described above. In general the bridge retains good integrity of location, design, setting, materials, workmanship, feeling, and association.

4.4 Character-Defining Features

Character-defining features are prominent or distinctive qualities or elements of an historic property that contribute significantly to its physical character, historic integrity, and significance. A list of character-defining features does not identify all important aspects of an historic property, however. Each historic property contains additional elements of location, design, setting, materials, workmanship, feeling, and association that together comprise its historic integrity or authenticity. Character-defining features of the Kennedy Bridge include:

- A location north of downtown Grand Forks and East Grand Forks that represented a major new river crossing.
- Two 279-foot-long steel Parker through trusses that include bolted connections, gusset plates, box-shaped top chords, inclined end posts at the same angle as diagonals, light-weight verticals and diagonals, lateral struts and bracing, portal struts and bracing, and floor beams and stringers. The trusses' broad smooth surfaces represent an aesthetic characteristic of the 1950s and 1960s.
- Elements that comprise the engineering response to the challenges of the Red River setting and its unstable soils including the provisions for substructure movement described in section 4.2.5.
- A late Streamline Moderne railing system consisting of a concrete parapet, aluminum pipe and post assembly, and curving concrete endposts.

Important elements of historic fabric also include the reinforced concrete piers with a restrained classically-influenced design, hexagonal columns, and 3-foot thick pier caps.

5.0 CONDITION ANALYSIS OF THE BRIDGE

5.1 Condition Terminology

The terminology used in this report to describe the condition of the bridge components is consistent with the National Bridge Inventory (NBI) guidelines and FHWA's *Recording and Coding Guide for Bridge Inspections*. The condition ratings used throughout this report are defined below:

Condition Rating	Definition	NBI Rating
Excellent	Element is in new condition	9
Very Good	Element has superficial deterioration	8
Good	Element has minor/isolated deterioration	7
Satisfactory	Element has minor to moderate deterioration	6
Fair	Element has moderate deterioration	5
Poor	Element has advanced deterioration	4
Serious	Element has severe deterioration	3
Critical	Element has critical deterioration (Immediate repairs may be required)	2
"Imminent" Failure	Element is no longer stable	1
Failed	Element has failed beyond the point of corrective action	0

5.2 Condition Summary

Deck

The 2014 MnDOT inspection report indicates that the deck is in **FAIR** condition with an NBI rating of 5. Numerous delaminated areas have been located and approximately 10 percent of the deck surface is unsound. Extensive areas of unsealed cracking have been identified as sources of leaching and deck saturation. The 2 inch thick, low slump concrete overlay is over 30 years old and is at the end of its useful life as defined by MnDOT's 2016-2020 *Bridge Preservation and Improvement Guidelines*. The deck overhangs are suffering from extensive delamination and rusting reinforcement (Figure 14).

Figure 14. Deck Overhang Delamination

The primary source of corrosion in the deck reinforcement is the presence of chlorides due to the use of deicing salts on the bridge deck. Chloride levels above 1,500 parts per million (ppm) are considered to be beyond the threshold at which steel reinforcement will start to corrode (MnDOT, 2015). Due to the observed corrosion of the bridge deck reinforcement, MnDOT conducted tests in 2013 to determine the chloride content of the deck concrete. Results of the testing are shown in Appendix C.

In general, the tests show that the chloride content greatly exceeds the 1,500 ppm threshold in the top two inches of the deck. Also, at the level of the reinforcing steel (2-3 inches), the chloride content exceeds the 1,500 ppm threshold at 50 percent of the tested locations and was greater than 730 ppm at the remaining tested locations.

Railings

The 2014 MnDOT inspection report indicates that the railing has minor spalls, scattered areas of cracking, and localized areas of impact damage. To assess the condition of the aluminum portion of the railing, a railing condition survey was conducted on May 19, 2015. The condition of each rail post was documented and given an associated condition category as shown in Table 1.

Table 1. Rail Post Condition

Condition Category	Reference Figure	Number of Posts	
		North Railing	South Railing
No or Minimal Damage	NA	133	138
Scraped/Scratched	Figure 15	8	6
Bent	Figure 16	6	2
Bent/Torn Minor	Figure 16	4	4
Bent/Torn Severe	Figure 17	0	1
Replaced	Figure 18	1	1

Figure 15. Scraped/Scratched

Figure 16. Bent or Bent/Torn (Minor)

Figure 17. Bent/Torn (Severe)

Figure 18. Replaced

Superstructure

The 2014 MnDOT inspection report indicates that the superstructure is in **SATISFACTORY** condition with an NBI rating of 6. There are numerous areas of corrosion and paint failure on the truss members, gusset plates, floor beams, stringers, pin and hanger assemblies, and approach span girders. Pack rust is causing distortion of some lower chord gussets and lower lateral bracing connection plates. Areas of isolated section loss were noted on the floor beams and lower chords of the truss spans. However, this section loss was noted as not significant.

Figure 19. Typical Superstructure Corrosion
Source: MnDOT 2013 Fracture Critical Inspection Report

Substructure

The 2014 MnDOT inspection reports that the substructure is in **POOR** condition with an NBI rating of 4. The primary factor affecting the substructure rating is the condition of pier 6.

Pier 6

There is severe cracking and a bow of approximately 5.5 inches in the concrete wall between pier columns as shown in Figure 20 and Figure 21. The pier has moved significantly towards the east and is tilting.

Figure 20. Pier 6 Cracking (East Face – Looking West)
Source: MnDOT 2013 Fracture Critical Inspection Report

Figure 21. Pier 6 Bow (West Face – Looking South)

In May, 2013, a technical memorandum entitled *Kennedy Bridge – Summary of Pier 6 Movement Records* was prepared by CH2MHill and summarized pier 6 movement records using data provided by MnDOT. This memorandum is included in Appendix F.

Figure 22 shows the displacement of the southern column from the constructed position over time based on an evaluation of the data provided in the 2013 memorandum. Prior to 1999, the column footing appeared to be moving at a rate of approximately 0.4 inches per year. Since 1999, the southern pier column foundation has moved at an average rate of 1.8 inches per year. As of 2012, the north and south column foundations have moved a total of 13.5 inches and 38.5 inches from

their constructed locations respectively. This movement is causing the northern pier column to tilt approximately 0.6 degrees and the southern pier column to tilt approximately 2.8 degrees. A comparison of the as-constructed position to the position as of December 2012 is shown in Figure 23.

Figure 22. Pier 6 Movement History (South Column)

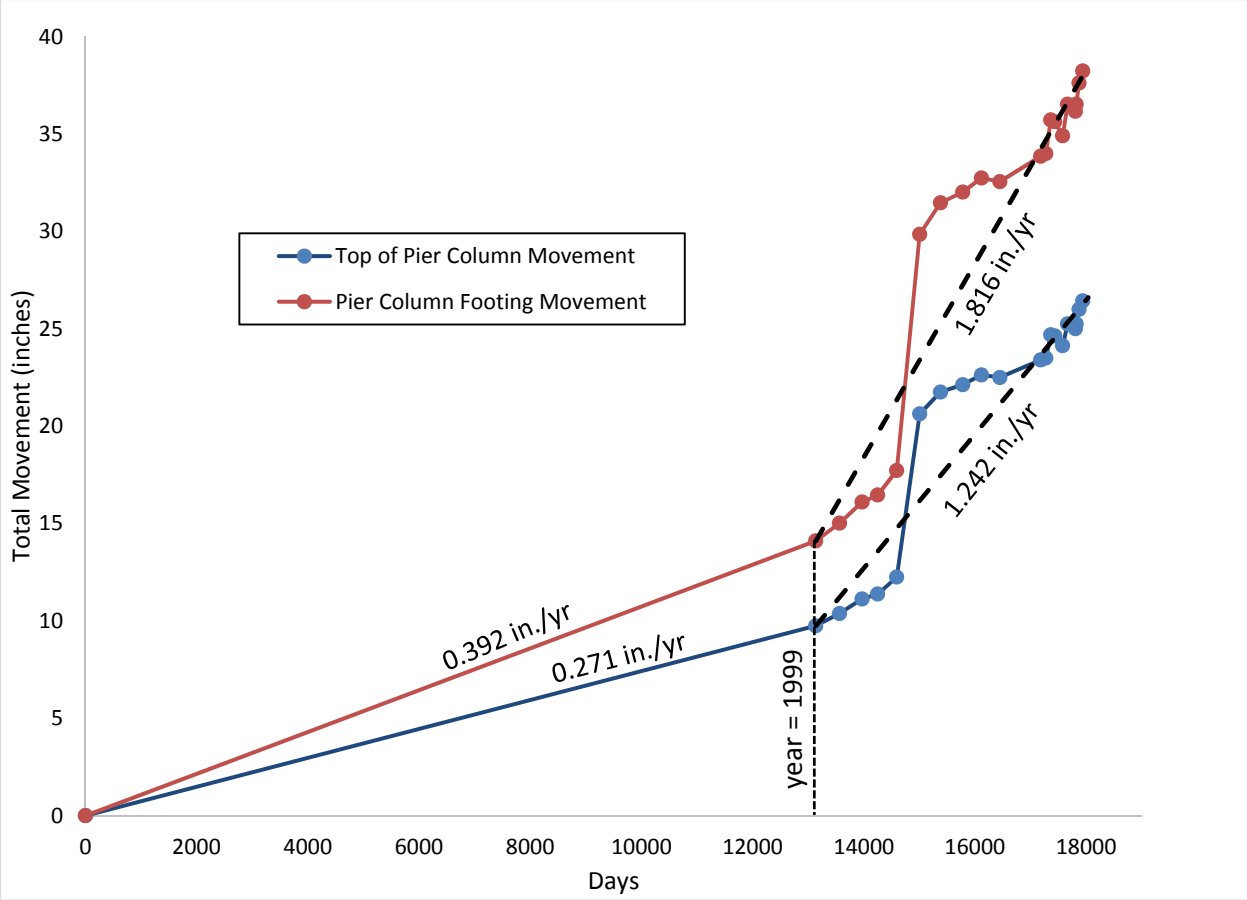
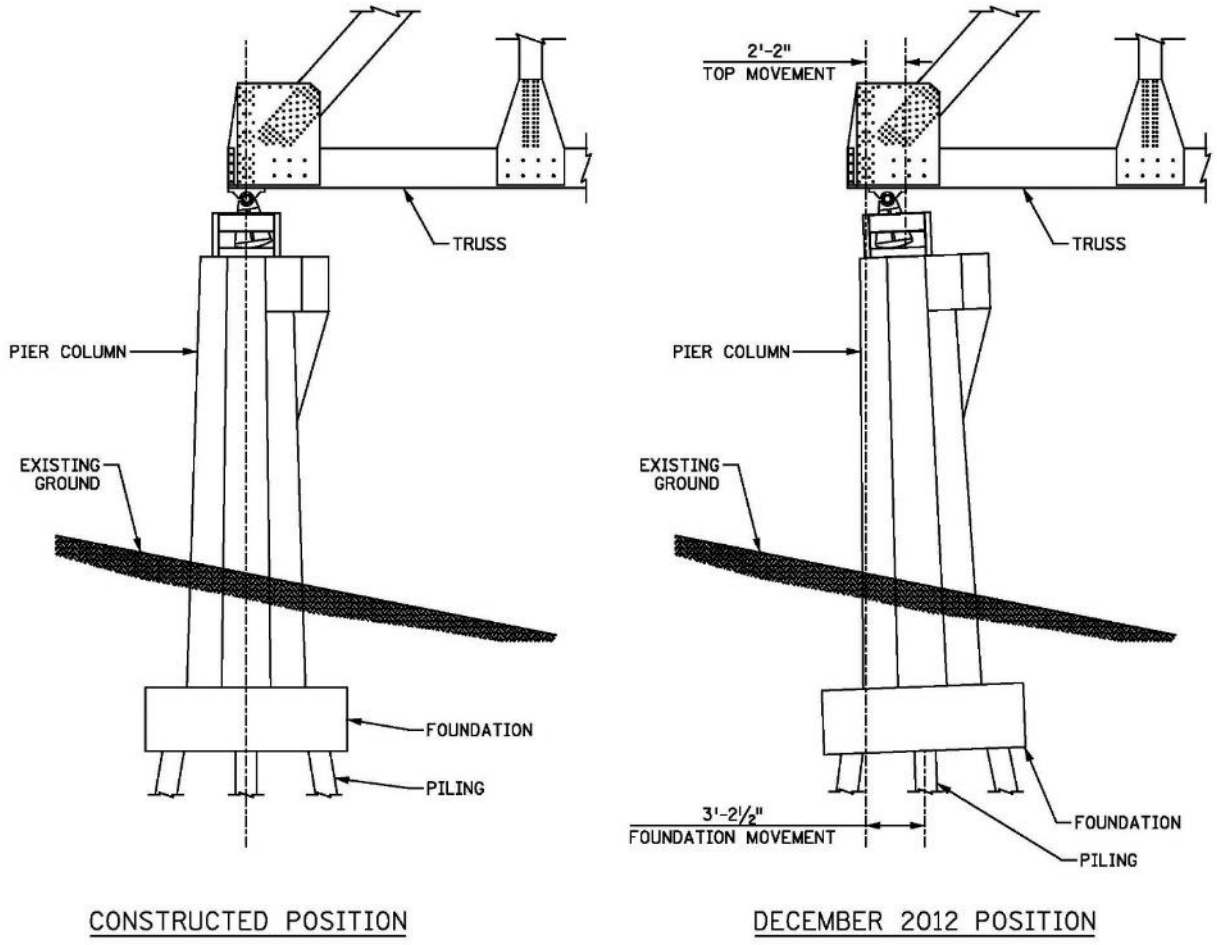


Figure 23. Pier 6 Constructed vs. Deflected Position (South Column)



Approach Span Pier Bents

MnDOT's bridge inspection report notes that the pier bents on both the North Dakota and Minnesota side of the river are all tilting towards the river as shown in Figure 24. Tilt measurements were taken on May 19, 2015. As the bridge expands and contracts due to temperature changes, the bearing tilt also changes. Table 2 lists the calculated tilt ranges for extreme cold and hot temperatures based on the temperature at the time that the tilt measurements were recorded.

To date, no actions have been undertaken to adjust the pier bent out-of-plumbness.

Abutments

MnDOT's inspection report notes that the abutments have undergone very little deterioration. There are isolated locations of minor hairline cracks and spalls. There are spalls behind all bearing assemblies due to the anchor pullout shown in Figure 51.

Figure 24. Approach Span Pier Bent Tilt (Bent No. 2 – Looking North)

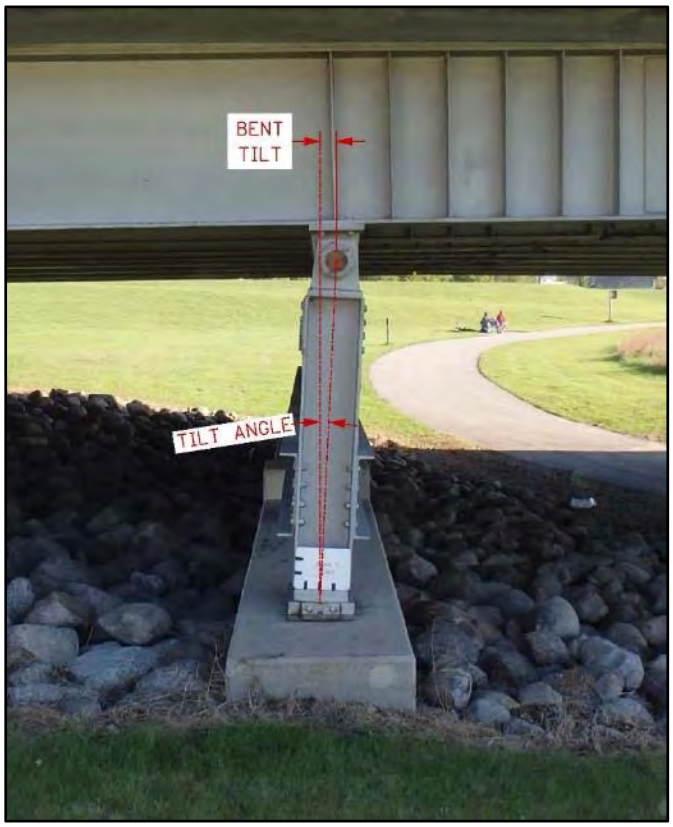


Table 2. Pier Bent Tilt Ranges
+ indicates tilt to the east; - indicates tilt to the west

	Westbound Spans		Eastbound Spans	
Bent No.	Tilt Range -30°to 120°F (inches)	Tilt Angle Range -30° to 120°F (degrees)	Tilt Range -30°to 120°F (inches)	Tilt Angle Range -30° to 120°F (degrees)
2	2.7 to 3.3	1.3 to 1.6	3.1 to 3.8	2.6 to 3.1
3	3.2 to 4.6	2.5 to 3.7	1.9 to 3.3	0.9 to 1.6
4	-0.2 to 2.0	-0.2 to 1.6	0.2 to 2.3	0.1 to 1.9
5	3.1 to 6.0	2.5 to 4.8	2.2 to 5.1	1.8 to 4.2
9	0.2 to -3.4	0.1 to -0.9	-1.2 to -4.8	-0.3 to -1.3
10	-0.5 to -3.4	-0.1 to -0.9	-1.3 to -4.2	-0.4 to -1.1
11	0.1 to -2.1	0.0 to -1.0	-1.3 to -3.5	-0.6 to -1.7
12	-1.8 to -3.2	-0.7 to -1.3	-2.1 to -3.5	-0.8 to -1.4
13	-2.7 to -3.3	-1.3 to -1.6	-2.7 to -3.3	-1.3 to -1.6

6.0 REHABILITATION ALTERNATIVES CRITERIA

6.1 Alternative Criteria

This 2013 *US Trunk Highway 2 Red River Crossing (Kennedy Bridge) Planning Study* by CH2MHill established alternatives to maintain the US 2 crossing of the Red River. Alternatives were developed and evaluated based on the following criteria:

Bridge Capital Cost: Considers construction costs to implement various bridge rehabilitation, repair, or replacement concepts.

Structural Performance and Maintenance: Considers the ability of the structure to perform over the long term (several decades into the future). This criteria also considers the ease at which the bridge can be maintained.

Construction Period and Traffic Impacts: Considers the ability to maintain reasonable movement of vehicular traffic during periods of construction and maintenance.

Historic Preservation (Section 106) Review: Considers the potential of the given alternative to cause an adverse effect to the historic bridge or to other historic resources.

Long-Term Traffic and Bike/Pedestrian Function: Considers the ability of the given alternative to safely and effectively serve vehicular traffic and accommodate bicycles and pedestrians.

Project Development Risks and Approval Process: Considers the risks associated with required design and environmental review as well as risks to the future performance of each alternative.

6.2 Rehabilitation Alternatives

The following alternatives were evaluated as part of the 2013 Planning Study and are summarized below. A more in depth discussion of the alternatives can be found in Appendix E.

No Action: The No Action or No Build alternative involves no significant repairs and would only consist of routine maintenance activities for a bridge of this type. This scenario would also include routine and fracture critical bridge inspections and continued monitoring of pier 6.

Bridge Rehabilitation A (bicycle/pedestrian accommodation *internal* to truss): This alternative consists of rehabilitating, repairing, or replacing of significant bridge components as described in Section 9 of this report. This alternative modifies the current deck configuration to accommodate a pedestrian/bicycle shared use path within the limits of the existing deck width.

Bridge Rehabilitation B (bicycle/pedestrian accommodation *external* to truss): This alternative modifies the Bridge Rehabilitation A alternative by attaching a new pedestrian/bicycle shared use path on the outside of the existing bridge structure.

Bridge Replacement: This alternative consists of a new, wider bridge adjacent to the existing structure. This alternative also explored possible improvements to the 4th Street ramps as well as possible replacement of the bridges over 4th Street.

6.3 Evaluation of Alternatives

Each alternative was evaluated on its performance relative to other alternatives for the six criteria listed in Section 6.1 of this report. A comparison of each alterative for the six criteria is given in Table 3.

Based on a review of the evaluation criteria, MnDOT recommends the project be developed as a bridge rehabilitation of the type shown in Alternative A. All six of the evaluation criteria were considered and weighed together in leading to a decision. MnDOT also considered the input from the Grand Forks and East Grand Forks community. Community input was gathered through public open houses, meetings with staff from the two city public works/engineering departments, the local and state offices of the North Dakota DOT, the Metropolitan Planning Organization, and each of the City Councils. Community input was gained, and continues to be sought, in the structural, construction, historic and bike/pedestrian function evaluation criteria.

Structural Performance and Maintenance: Rehab Alternative A addresses the primary need of a structurally sound bridge crossing. Alternative B also addresses the primary need but it introduces maintenance and inspection concerns with additional structure outside the truss. The community expressed the desire of a reliable crossing not subject to frequent closures or a reduction in capacity. It is recognized that the Replacement Alternative meets this criteria best.

Construction Period and Traffic Impacts: All rehab alternatives and the replacement option impacted traffic about equally. The community desired that at least one lane in each direction be open to traffic during construction.

Historic Preservation (Section 106) Review: Alternative A has the lowest potential for effects that diminish the historic characteristics of the bridge.

Long-Term Traffic and Bike/Pedestrian Function: Alternative A provides for a good improvement over the existing condition of no separate bike/pedestrian facility on the existing bridge. It is recognized that Rehabilitation Alternative B and the Replacement Alternative would each provide for a better bike/pedestrian accommodation for crossing the bridge. The community expressed desire to improve the existing condition that has no accommodations. They communicated that the north side would be most desirable as it matches the in-place trail system on the Grand Forks side, and allows for the trail to be eventually connected to the trails on the East Grand Forks side.

Project Development Risks and Approval Process: The Rehabilitation Alternative A has the lowest potential for project development risks and the shortest review time frame when compared with Rehabilitation Alternative B or the Replacement Alternative.

Table 3. Comparison of Alternatives

Evaluation Criteria	Alternative			
	No Action	Bridge Rehabilitation A	Bridge Rehabilitation B	Bridge Replacement
Bridge Capital Cost	None	Lowest	Medium	Highest
Structural Performance and Maintenance	Does not address condition of pier 6 or condition of deck, low service life (even with routine maintenance)	Addresses primary need; good long-term service life with regular maintenance; inspection issues similar to the existing bridge	Addresses primary need, with conflicts; good long-term service with regular maintenance; however, inspection of the original truss members around the external structure and maintenance access would be blocked or limited	Addresses primary need and adds structural redundancy; longest service life; superior inspection and maintenance access provided through design of any replacement bridge type
Construction Period and Traffic Impacts	No construction delays	About 1 year of temporary traffic impacts and capacity restrictions	About 1 year of temporary traffic impacts and capacity restrictions	About 2 years of temporary traffic impacts with periodic capacity restrictions
Historic Preservation (Section 106) – Risk of Adverse Effect	No adverse effect	Low to moderate potential for adverse effects	Much higher potential for adverse effects	Removal of historic bridge is an adverse effect; potential additional impacts to the Riverside Neighborhood Historic District
Long-Term Traffic & Bike/Pedestrian Function	4 roadway lanes with future load restrictions possible; no ped/bike accommodations	4 roadway lanes; ped/bike on bridge deck; good performance for all modes	4 roadway lanes; separate ped/bike path on external structure; superior performance for all modes	4 roadway lanes; separate ped/bike path, superior performance for all modes
Project Development Risks and Approval Process	No approval process; risk of future load restrictions or complete bridge closure due to continued movement of pier 6 or further deck deterioration	Lowest-risk and least potential for adverse impacts, including Section 106 adverse effects; smallest footprint; prompt reviews and approvals possible through a CE or an EA prepared in parallel with detailed design	Substantially higher project development risks (Section 106, hydraulic, and other impacts); increased footprint; review and approvals more complex, through an EA; potential to add months or years to the design/approval process	Highest project development risks (many adverse impacts, including Section 106, with largest footprint); review and approvals most complex, through an EA or EIS; potential to add years to the design/approval process

7.0 PROPOSED CROSS-SECTION

Selection of the proposed cross-section geometry (Figure 26) was based on balancing the project's purpose and need with the desire to preserve the historic integrity of the bridge. The recommended deck width of 67-foot, 4-inches was selected to match the width of the existing bridge deck (Figure 25) and to fit within the confines of the existing truss members. However, in order to meet the project's need of providing for non-motorized river crossings, it is recommended that the curb and median elements not be included in the proposed cross-section.

Figure 25. Existing Cross Section

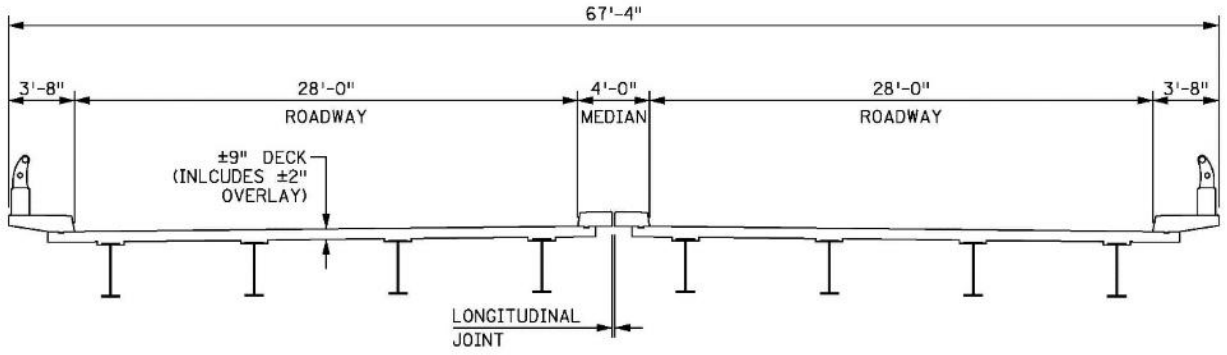
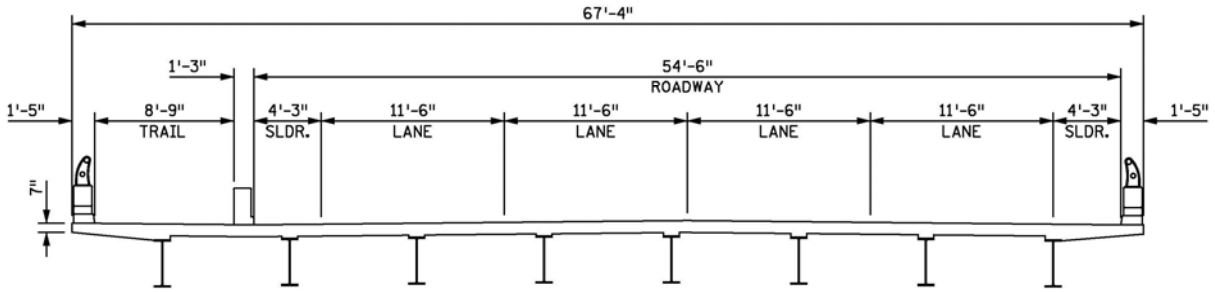


Figure 26. Proposed Cross Section



Early in the project development, a memorandum (Appendix G) was developed that identified the geometric elements of the proposed deck section that are desirable from a transportation function standpoint. This memorandum compared trail, lane, shoulder, and roadway widths with those required by various MnDOT specifications. When added together, the sum of the widths of all geometric elements exceeded the proposed deck width. Therefore, proposed element widths were developed and an overall consensus from all stakeholders was sought.

Table 4 compares the element widths of the recommended, proposed cross-section with those of that are required or desirable from a transportation function standpoint.

Table 4. Desirable#, Required, and Proposed Cross-Section Element Geometry

Cross-Section Element	Desirable# Dimension	Minimum Required Dimension	Proposed Dimension	Applicable Code Reference
Trail Width	10'-0"	8'-0"	8'-9"	MnDOT LRFD Bridge Design Manual – Section 2.1.2
Lane Width	12'-0"	11'-0"	11'-6"	MnDOT Bridge Preservation and Improvement Guidelines – Table 1
Shoulder Width	6'-0"	4'-0"	4'-3"	MnDOT Tech Memo No. 12-14-B-03 – Section 2.1.1.2.e
Roadway Width	53'-0"	54'-6"	54'-6"	MnDOT Bridge Preservation and Improvement Guidelines – Chapter 6*

#Desirable from a transportation function standpoint

*Width required for NBI Deck Geometry Rating of 5

8.0 EVALUATION OF LOAD CAPACITY

8.1 Proposed Loading Criteria

Permanent Loads

Permanent Loads (also referred to as dead loads) are loads which do not vary with time upon completion of construction. Permanent loads for Bridge No. 9090 include the weight of the deck, girders, stringer, truss system, railings, substructures or any other items permanently affixed to the bridge.

Live Loads

Live Loads are loads due to pedestrian and vehicle traffic which vary over the life of the structure. For Bridge No. 9090, these loads include the following:

Pedestrian Loads

Section 3.6.1.6 of AASHTO's *LRFD Bridge Design Specifications,* requires that a pedestrian load of 75 pounds per square foot shall be applied to sidewalks and shall be considered to act simultaneously with vehicular design loads.

Design Vehicle Loads

Section 6A.1.5.1 of AASHTO's *Manual for Bridge Evaluation,* requires that ratings at both the inventory and operating level shall be based on the HL-93 loading as defined by Section 3.6.1.2 of AASHTO's *LRFD Bridge Design Specifications*. This load consists of a truck with an 8,000-pound front axle and two, 32,000 trailing axles or a tandem consisting of a pair of 25,000-pound axles. In addition to the truck or tandem loads, an additional lane load of 640 pounds per foot shall be applied in each lane.

Legal and Permit Vehicle Loads

In addition to the AASHTO specified vehicle loads, the bridge will also be required to accommodate the Minnesota legal (posting) loads and the Minnesota standard permit trucks defined in Section 15 of MnDOT's *LRFD Bridge Design Manual*.

Other Loads

Additional loads which will be accounted for in final design include loads due to inspection vehicles, earth, wind, ice, and thermal expansion/contraction.

8.2 Superstructure Load Rating

For the proposed cross-section, a preliminary load rating using AASHTO design vehicles, was performed for each superstructure element using the Load and Resistance Factor Rating (LRFR) method defined in AASHTO's *Manual for Bridge Evaluation*. The Load Rating calculates a rating factor for each element in order to assess the performance of the existing bridge at two levels:

Inventory Level: Rating equivalent to the level of stress a bridge can expected to safely carry traffic for life of 75 years. A rating factor greater than or equal to 1.0 indicates the element meets the strength requirements for a new bridge. However, in accordance with MnDOT's *Bridge Preservation and Improvement Guidelines*, an inventory rating factor of 0.9 or greater is considered acceptable for bridge improvement projects recognizing that some of the service life of the bridge has transpired.

Operating Level: Rating equivalent to the maximum permissible stress level to which a structure may be subjected. The operating rating determines the safe, structural capacity of the bridge for state legal loads and for permit loading. Allowing an excessive number of vehicles to use a bridge at the operating level may shorten the life of the bridge. An operating rating indicating that the bridge does not require posting for state legal loads and provides no restrictions for standard overweight permit vehicles is considered satisfactory in accordance with MnDOT's *Bridge Preservation and Improvement Guidelines*.

The rating factor for each element was calculated using the general form of the rating equation given below:

$$Rating\ Factor = \frac{Structural\ Capacity - Permanent\ Loads - Pedestrian\ Loads}{Traffic\ Loads}$$

In accordance with MnDOT's 2013 *Design Exceptions and Variances on Historic Bridges: Effective Applications and Utilization Guidelines* document, refined analysis methods were used to evaluate all elements of the bridge. A two-dimensional, finite element analysis was used to determine the loads in all truss members and a three-dimensional, finite element analysis was used to determine the loads in the approach spans and the truss floor system.

Ratings were calculated using the AASHTO HL-93 design loads. Minnesota legal and permit vehicles were not evaluated as part of the preliminary load rating. However, HL-93 operating rating factors greater than 1.35 are generally considered to be an indication of the element's adequacy for legal and permit loadings.

The results of the preliminary load rating are presented in Table 5. Results of the preliminary load rating indicate that the inventory rating factors for the pin and hanger assemblies and truss span stringers fall below the acceptable value of 0.90. Additionally, the inventory rating factors for the truss span floor beams are close to the acceptable 0.90 value. Any future deterioration of the truss span floor beams could cause the ratings to fall below acceptable levels.

It should be noted that these results are considered preliminary and will be refined during the final design process.

Table 5. Preliminary HL-93 Rating Factors

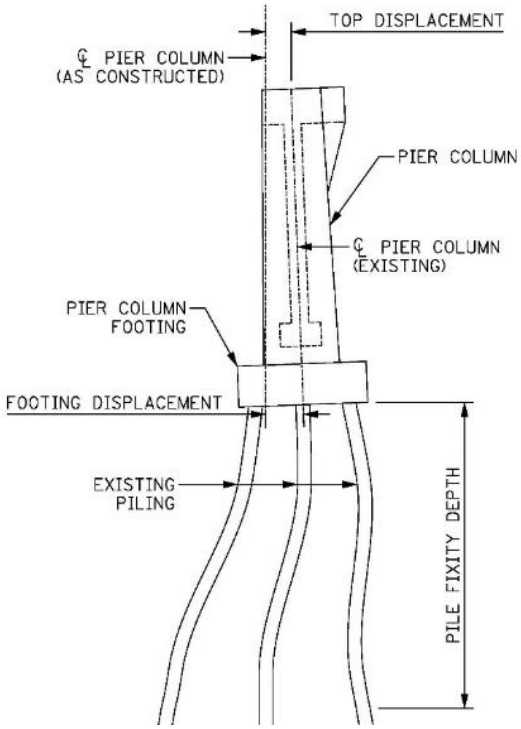
Element	Spans	Inventory Rating Factor	Operating Rating Factor
Approach Span Girders	1-5 & 8-13	1.20	1.58
Pin and Hangers	1-4 & 9-13	0.60	0.78
Truss Members	6 & 7	1.10	1.42
Floor Beams	6 & 7	0.92	1.19
Stringers	6 & 7	0.74	0.95

8.3 Substructure Evaluation

Pier 6 Movement

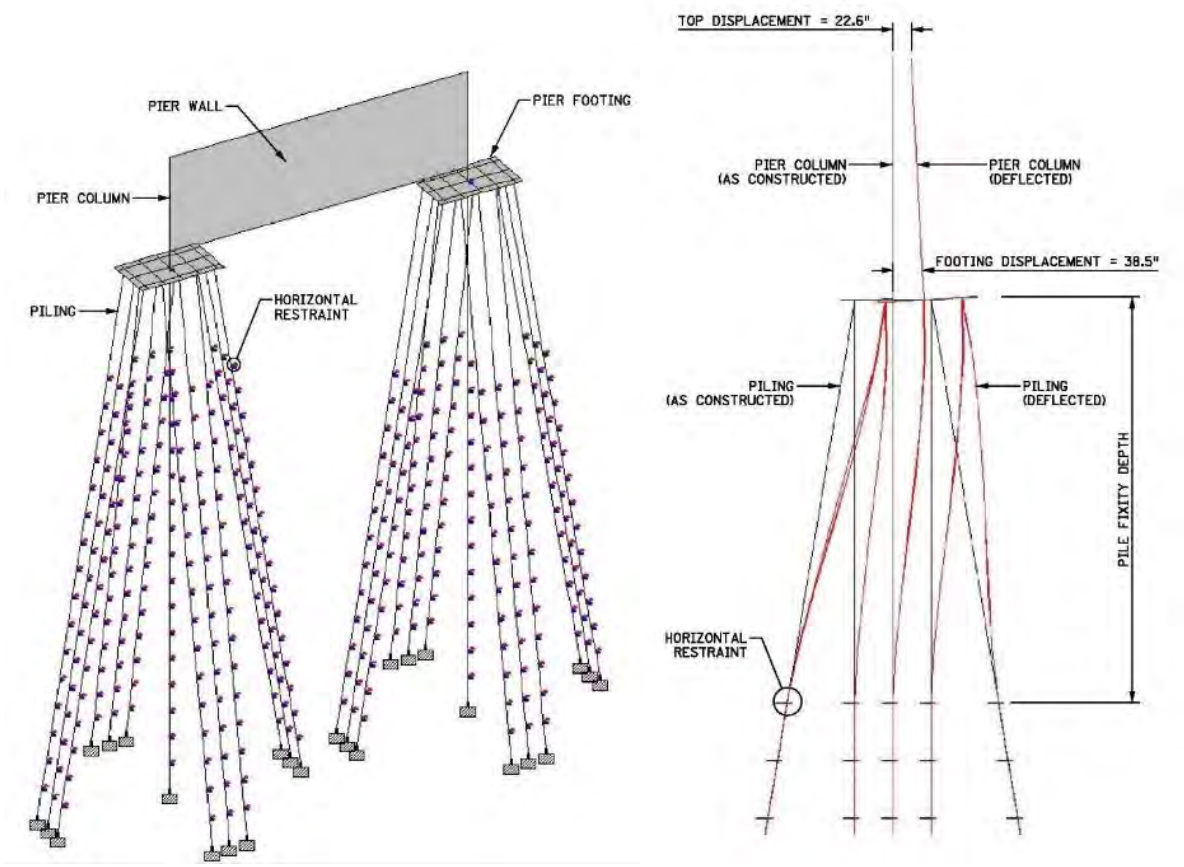
In order to assess the structural capacity of the existing piling, an exercise was conducted to determine the magnitude of the current forces induced in the piling due to soil movement. Although the existing piles are not able to be observed, an approximate position or deflected shape of the piling below pier 6 can be inferred based on the magnitude of horizontal movement and tilting as shown in Figure 27.

Figure 27. Pier 6 Pile Deflected Shape



To determine the deflected shape of the piling, a three-dimensional finite element model was created to model the effects of the known substructure movement (Figure 28). Horizontal restraints were placed along the length of the piling to model the soil support. The restraints were removed to mimic locations where the moving soil has deflected the piles. These restraints were removed until both the top displacement and the footing displacement matched what has been observed. Using the results of the finite element model, forces in the piling could be estimated. Evaluation of the forces found that the stresses in the piling significantly exceed the limits of AASHTO's *LRFD Bridge Design Specifications*. This indicates that the piles have undergone permanent deformation and are no longer providing a reliable resistance to lateral loads.

Figure 28. Pier 6 Finite Element Model

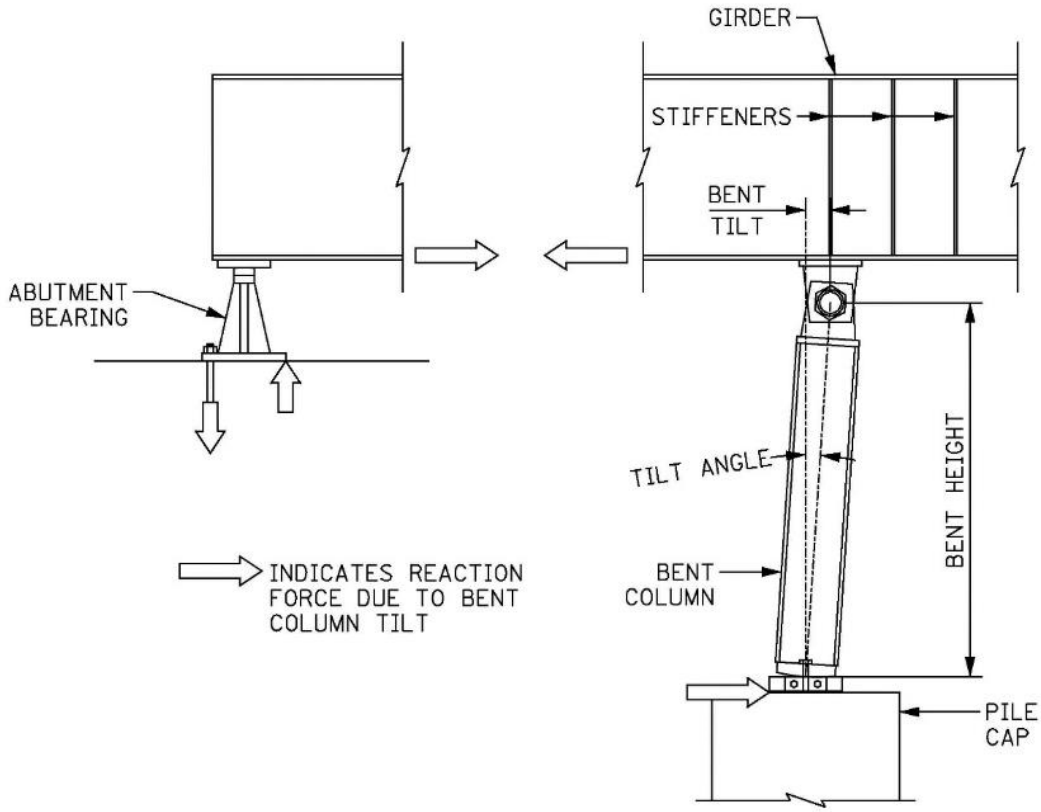


ISOMETRIC VIEW (MODEL) SIDE VIEW (RESULTS)

Approach Span Pier Bent Tilt

Excessive tilting of the pier bents induces a horizontal force at the tops of the pile caps and a corresponding reaction at the abutment bearings as shown in Figure 29. The reaction at the abutment bearings causes the bearing to rotate and induces tension in the anchor rods.

Figure 29. Pier Bent Horizontal Reaction Forces



An analysis of the approach span pier bents was undertaken to determine the effects of the observed tilting on the pier's structural capacity. Large tilt angles were found to induce lateral forces in the pile caps and reduce the capacity of the existing piling. From this analysis it was determined that any pier bent with a tilt angle greater than 3.7 degrees could cause pile stresses that exceed the limits of AASHTO's *LRFD Bridge Design Specifications*.

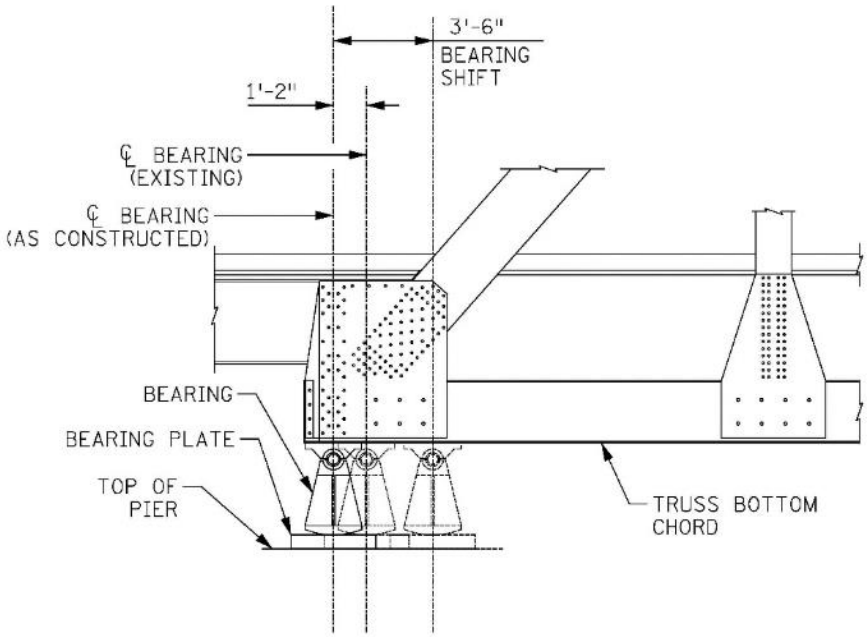
8.4 Truss Bearing Adjustment Evaluation

A unique feature of Bridge No. 9090 is the ability to accommodate movements of the substructure elements. To accommodate movements of piers 6 and 8, engineers designed the truss bearings to be adjustable along the bottom chord of the truss. Figure 30 shows the bolts placed along the bottom chord to facilitate this adjustment.

Figure 30. Truss Bearing Adjustment Accommodation

To evaluate the potential structural effects of this rehabilitation, a preliminary analysis was conducted to determine the maximum adjustment that could support modern traffic loads. The analysis found that the bearing can be adjusted along the bottom chord by 3.5 feet and still retain an inventory rating factor greater than 1.0. Figure 31 shows the as constructed position, the existing position, and the position of the bearing which achieves a rating factor greater than 1.0.

Figure 31. Truss Bearing Adjustment



9.0 PROPOSED REHABILIATION

9.1 Deck and Deck Joints

Bridge Deck

The deck, which is original, is a 7-inch thick, cast-in-place concrete deck constructed with uncoated, or "black", reinforcing steel. To enable soil movements and thermal expansion/contraction, finger style joints (Figure 36) were placed at every pier (12 total joints). In 1984, a new 2-inch overlay was placed on top of the existing driving surface. To facilitate this 2-inch overlay, the original finger joints at piers 2-4, 6-8, and 10-13 were replaced with 2-inch wide strip seal style joints (Figure 33) and the finger joints at piers 5 and 9 were salvaged and raised. In addition to the geometrical considerations described in Section 7.0 of this report, the decision to repair or replace the existing deck must weigh the desire to maintain historic fabric with the condition of the existing deck.

For a deck with a NBI condition rating of 5, MnDOT's *Bridge Improvement and Preservation Guidelines* specify that the deck should be repaired with a new, 2-inch overlay. However, an additional requirement specified in the Guidelines also states that "The bridge should be programmed for overlay replacement before the [chloride content of the] concrete at the level of the top rebar reaches half of the corrosion threshold."

The chloride content at and below the level of the top rebar (3-4 inches) exceeds the corrosion threshold. Repair of the existing overlay would remove the chlorides in the top 2 inches of concrete, but the chlorides at and below the level of the top rebar would remain. This would indicate that deterioration of the existing rebar would continue and the life of any new overlay would be significantly decreased. At a certain level of corrosion, the deck would no longer provide the structural capacity required to support modern traffic loads. Therefore, replacement of the deck is recommended under this rehabilitation.

Replacement of the deck would necessitate removal of the railing, sidewalks, and center median which are integrally connected to the deck. The new deck would be 9 inches thick (rather than the original 7 inches) and would taper to 7 inches at the edge of the overhangs to help visually mitigate the fact that the cantilevered raised sidewalk edge treatment will not be reproduced (see Figure 37). The new deck drainage system would be designed to resemble the original system with slight modifications as described in Section 9.9 below.

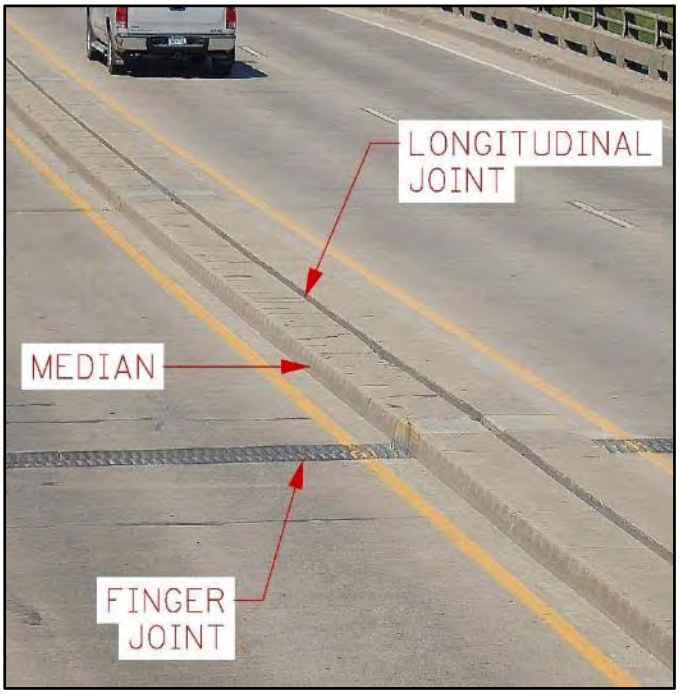
Selection of the appropriate functional geometry (deck, lane, and trail widths) of the new bridge deck is based on accommodation of the various modes of transportation while minimizing changes to the historic bridge. The geometric elements of the deck, as viewed from the exterior of the bridge, were developed in concert with the railing and are described in Section 9.2 below. To achieve these elements, the new deck would not have a symmetrical configuration, a center median, or raised safety walkways, as it did historically. It would have a new 8-foot 6-inch pedestrian/bike trail, on the north side only, separated from the four travel lanes by a new inner barrier as described in Section 7.0 above.

Longitudinal Median Joint

The longitudinal joint (Figure 32) runs the entire length of the bridge at the centerline and allows the eastbound half of the bridge to translate longitudinally relative to the westbound half. The joint is part of the bridge's character-defining provisions for soil movement. To function as intended, the longitudinal joint must be placed between the two center girders as shown in Figure 25

In order to accommodate the proposed trail and roadway widths shown in Figure 26, the inside westbound lane must be located over the center girders. To function properly, a longitudinal joint would be required in a traffic lane and at the surface of the roadway. Installing a longitudinal joint in the traffic lane or at the surface of the roadway is not prudent from a safety standpoint. A longitudinal joint is considered a hazard to vehicles (especially motorcycles) as the joint can catch a wheel and cause a vehicle to become unstable.

Figure 32. Longitudinal Deck Joint



An evaluation of the structural effects of removing the longitudinal deck joint is described in the technical memorandum found in Appendix I. The evaluation found that, from a structural standpoint, removal of the joint is feasible.

Since the addition of a longitudinal joint to the newly constructed deck is not prudent from a safety standpoint and since removal of the joint is feasible from a structural standpoint it is recommended that, under this rehabilitation, the deck be constructed without a longitudinal joint.

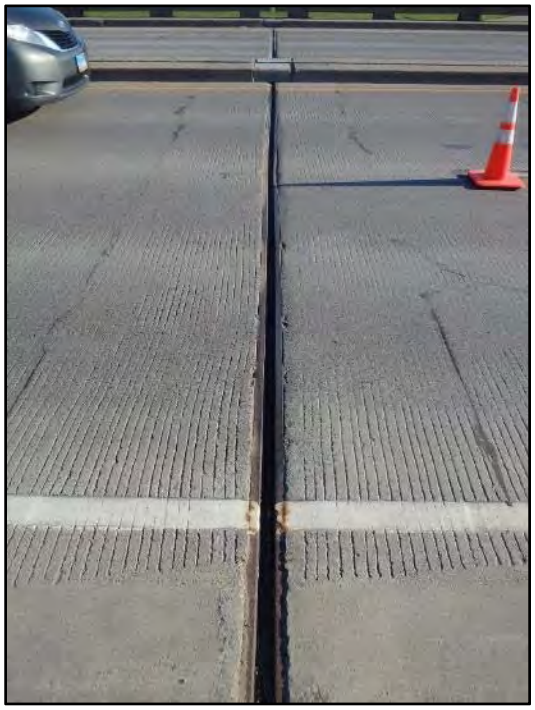
Transverse Roadway Deck Joints

The transverse deck joints consist of 12 strip seal and 2 finger joints (one located at each pier). The strip seal joints (installed in 1984) replaced 12 of the 14 original finger joints. The remaining finger joints are original, but were raised in 1984 to match the top of the newly installed wearing surface. The existing deck joints were fabricated to conform to the shape of the top of the existing deck, curbs, and median. The existing joints would not conform to the shape of the proposed deck section and, therefore, their reuse is not practical from an engineering standpoint. Since the proposed deck cross section would preclude reuse, it is recommended that the existing transverse joints be replaced as described below.

Strip Seal Joints

The purpose of the strip seal joints (Figure 33) is to permit girder rotations at the pin and hanger joints and to accommodate substructure movements at piers 6, 7, and 8. Replacement joints would be of a similar style, with an adjusted profile to match the new top of roadway. Joints at piers 2-4, pier 7 and piers 10-13 would be 2 inches wide to match the in place joints. Due to the history of movements at pier 6 and the potential for movement at pier 8, 3-inch wide joints are recommended at these piers. The need for these larger joints was evaluated in the memo found in Appendix I.

Figure 33. Typical Strip Seal Joint



To prevent tripping, the Section 302.3 of the American's with Disabilities Act specifies that "Openings in floor or ground surfaces shall not allow passage of a sphere more than 1/2-inch (13 mm) diameter." Additionally, in order to provide a smooth walking surface, Section 14.2.2 of MnDOT's LRFD Bridge Design Manual requires engineers to "Detail cover plates on sidewalks,

medians, and pedestrian bridges to cover the [expansion joint] opening." Additionally, the openings in transverse joints can cause steering difficulties for bicyclists and are considered a hazard (MnDOT, 2007).

An example of a curb cover plate on the existing curb of Bridge 9090 is shown in Figure 34. For this rehabilitation, it is recommended that a similar style plate be used at all strip seal joint locations and extended to cover the full width of the shared use path as shown in Figure 35.

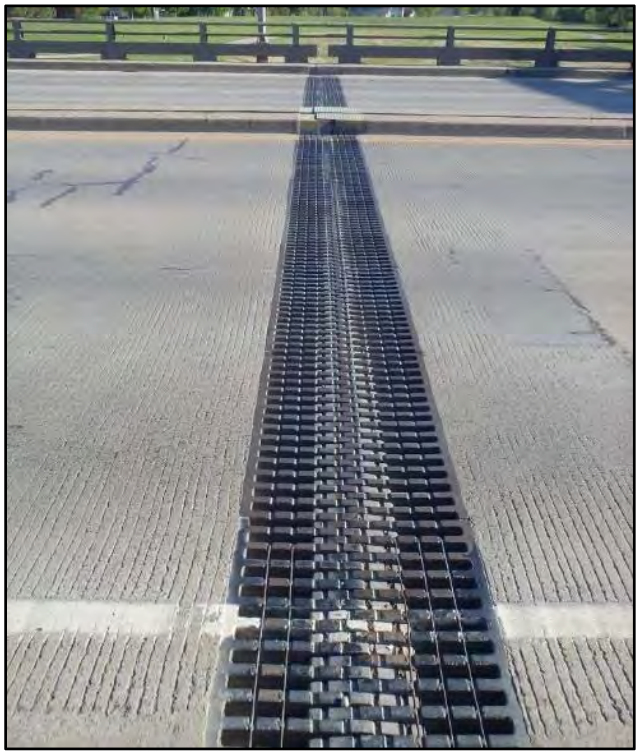
Figure 34. Existing Curb Cover Plate

Figure 35. Sidewalk Transverse Joint Cover Plate Example

Finger Joints

The primary purpose of the finger joints (Figure 36) at pier 5 and pier 9 is to accommodate the expansion or contraction of the bridge due to changes in temperature that occurs at the steel hinges. A secondary function of these joints is to permit girder rotations at the steel hinges and to accommodate transverse substructure movements of piers 5, 6, 9 and 10. Replacement joints would be of a style fabricated to replicate the existing finger joints, with an adjusted profile to match the new top of roadway. Sidewalk cover plates, similar to those required at the strip seal joints, will also be used where the finger joints cross the path.

Figure 36. Typical Finger Joint



9.2 Railings

On May 30, 1997, the FHWA published an action letter regarding the crash testing of Bridge Railings. This document states FHWA's position that, on all NHS projects, "The minimum acceptable bridge railing will be a TL-3 (MSL-2 until August 1998) unless supported by a rational selection procedure" (FHWA, 1997). The TL-3 test level is established by conducting crash testing of railings to determine if minimum strength and safety criteria are met. Included in FHWA's action letter was a list of railings that have been tested and approved for use on the NHS. The FHWA permits modifications to these railings provided that an analytic comparison is performed on the modified railing.

In addition to the requirements for vehicular railing, the *AASHTO LRFD Bridge Design Specifications* has strength and geometry requirements for exterior pedestrian/bicycle railings.

Exterior Railings

Replacement of the bridge deck will require replacement of the existing railing, a character-defining feature of the bridge. Replacement railing options were developed along with the deck cross section to balance pedestrian, bicycle, and traffic requirements with the bridge's historic integrity. A summary of the railing selection process is detailed further in Appendix H.

In order to meet the requirements for both pedestrian, bicycle, and vehicle railings and reduce the visual impact to the bridge, use of a modified Nebraska Open Concrete Bridge Rail is recommended for this rehabilitation. The original, crash tested, Nebraska railing would be modified to more closely match the existing railing geometry while still meeting FHWA and AASHTO requirements. An analytic comparison of the modified railing was performed and submitted to the FHWA for approval. A summary of this evaluation is given in the memorandum in Appendix J.

During development of railing concepts, MnDOT bridge maintenance engineers requested that the openings in the railings be filled with concrete to reduce the exposure of the truss bottom chord to deicing chemicals. Creating recesses in the barrier to replicate the size of the existing openings was explored as a way to mitigate the visual effects of a solid barrier. However, the open railing is one of the most visually dominant elements of the bridge and this dramatic visual change was determined to significantly detract from the historic integrity of the bridge. Therefore, the option was not explored further.

Salvaged Aluminum Railing

The aluminum portion of the bridge railing is not part of the crash tested concrete barrier. Most of the railing is in good condition and it is recommended that it be salvaged and reinstalled on top of the crash tested Nebraska railing where the condition of the railing and the condition of the components allow.

The railing's pipes were determined to be in good condition and their reuse is recommended. To determine where reuse or replacement of the rail posts was warranted, criteria based on the railing condition was developed for both the north railing, which would be adjacent to the pedestrian/bicycle trail, and the south railing, which would be adjacent to the roadway, as follows:

> **Railing Adjacent to Roadway:** Minor scrapes, bends and tears, as shown in Figure 15 and Figure 16, do not substantially impact the safety of the railing. Rail posts classified as damaged in this way are recommended for reuse adjacent to the roadway. Posts classified as having severe damage, as shown in Figure 17, could be a hazard to vehicles and are recommended to be replaced.
>
> **Railing Adjacent to Ped/Bicycle Trail:** Minor scrapes, bends and tears, as shown in Figure 15 and Figure 16, could snag the clothing of, or cut, a pedestrian and could be a hazard. Therefore, it is recommended that these posts be replaced, in addition to the more severely damaged posts, where adjacent to the pedestrian/bicycle trail.

In addition, there are a few replacement posts that do not match the originals in material and geometry (Figure 18), and are also recommended to be replaced.

As shown in Table 1, there are approximately 20 posts which meet the specified replacement criteria. Additionally, posts could be damaged in the future and purchasing an additional 10-20 posts is recommended. Replacement posts matching the material and geometry of the original posts are available and are recommended to be used for this rehabilitation.

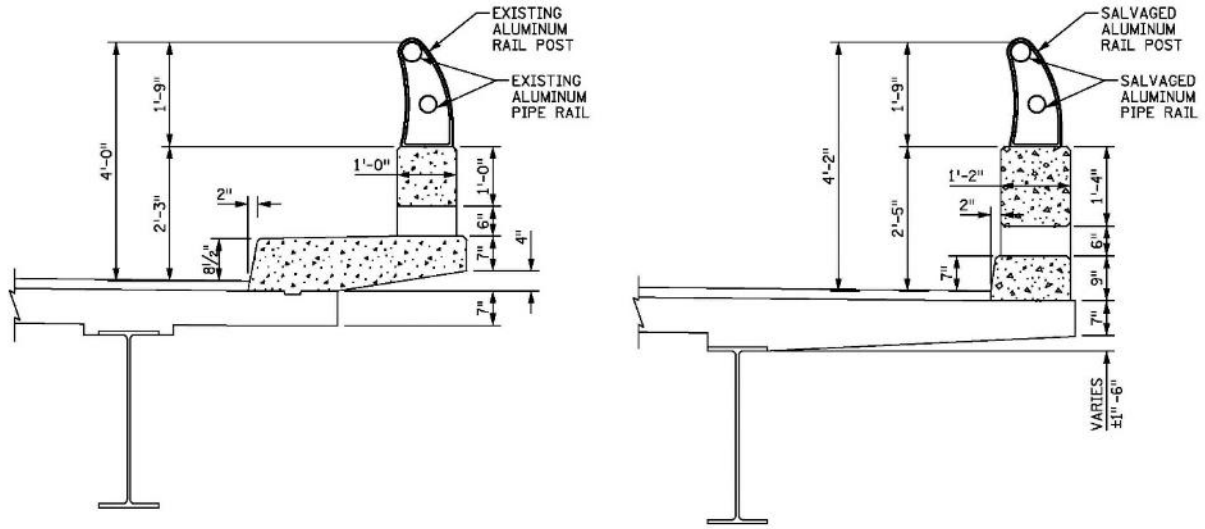
Typical Railing

Figure 37 and Figure 38 show a comparison of the existing and proposed cross-section and elevation of railing at typical locations along the bridge. To meet strength and safety requirements, the proposed, concrete rail thickness would be increased from existing by 2 inches and the height of the continuous beam element would be increased by 4 inches (see Figure 37). Also to meet crash testing requirements, the width of the post element adjacent to expansion joints would be increased by 1-foot from existing (see Figure 38).

The proposed railing will be 2 inches taller than existing when measured from the inside. The overall height of the existing railing is 48 inches, measured from the roadway surface, and 39 inches measured from the top of the safety walkway. The proposed railing would be 50 inches tall as measured from the surface of the new roadway and pedestrian/bicycle trail.

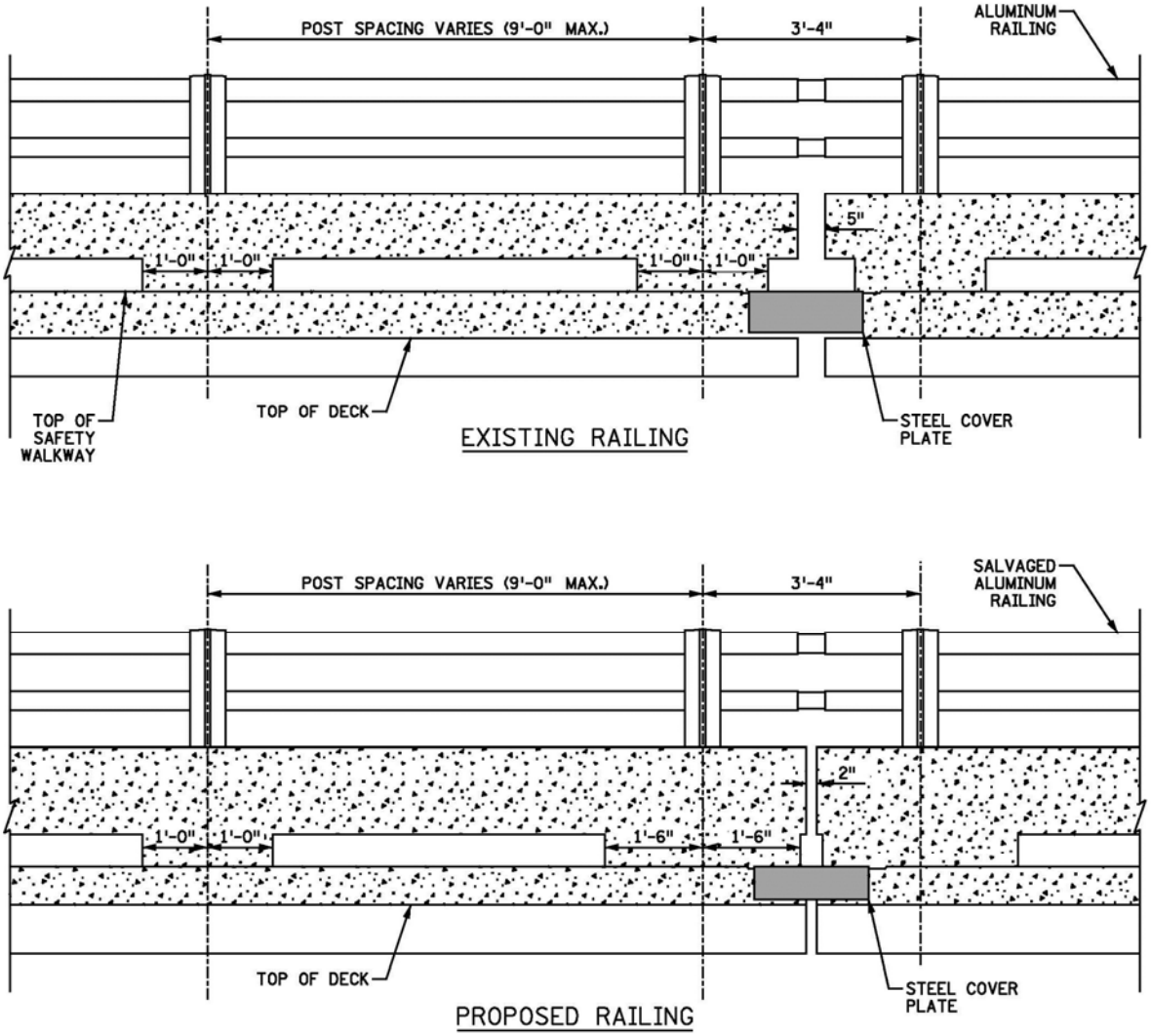
When viewed from the outside, the proposed railing will be 13 inches taller. The existing rail and deck edge measure 46 inches (Figure 37and Figure 38). The proposed railing and deck edge would measure 59 inches. The pattern of openings to posts at the bottom of the proposed concrete rail will be similar to existing as shown in Figure 38.

Figure 37. Existing vs. Proposed Railing Cross-Section



EXISTING RAILING & DECK EDGE **PROPOSED RAILING & DECK EDGE**

Figure 38. Existing vs. Proposed Railing Typical Elevation



Railing at Ends of the Bridge

A comparison of the existing and proposed railing elevation at the ends of the bridge is shown in Figure 39. In order to fit enough steel reinforcement in the final 5 feet of railing to meet FHWA strength requirements it is recommended that the smaller openings in the railing be reduced by 3 inches. The width and height of the replacement 6-foot end post would be increased to match the replacement railing, but it is recommended that the geometry of the 'wavy' top be retained.

It is also recommended that the four original brass name plates in the existing end posts (Figure 40) be salvaged and reinstalled in the same position. These name plates should not be cleaned or painted to retain their patina of age. It is recommended that one new plate, indicating the bridge rehabilitation date, be installed on the northwest end post.

New, Type 3, object markers are recommended to be installed in front of the two, southern end posts adjacent to the roadway as described in Section 9.12.

Figure 39. Existing vs. Proposed Railing End of Bridge Elevation

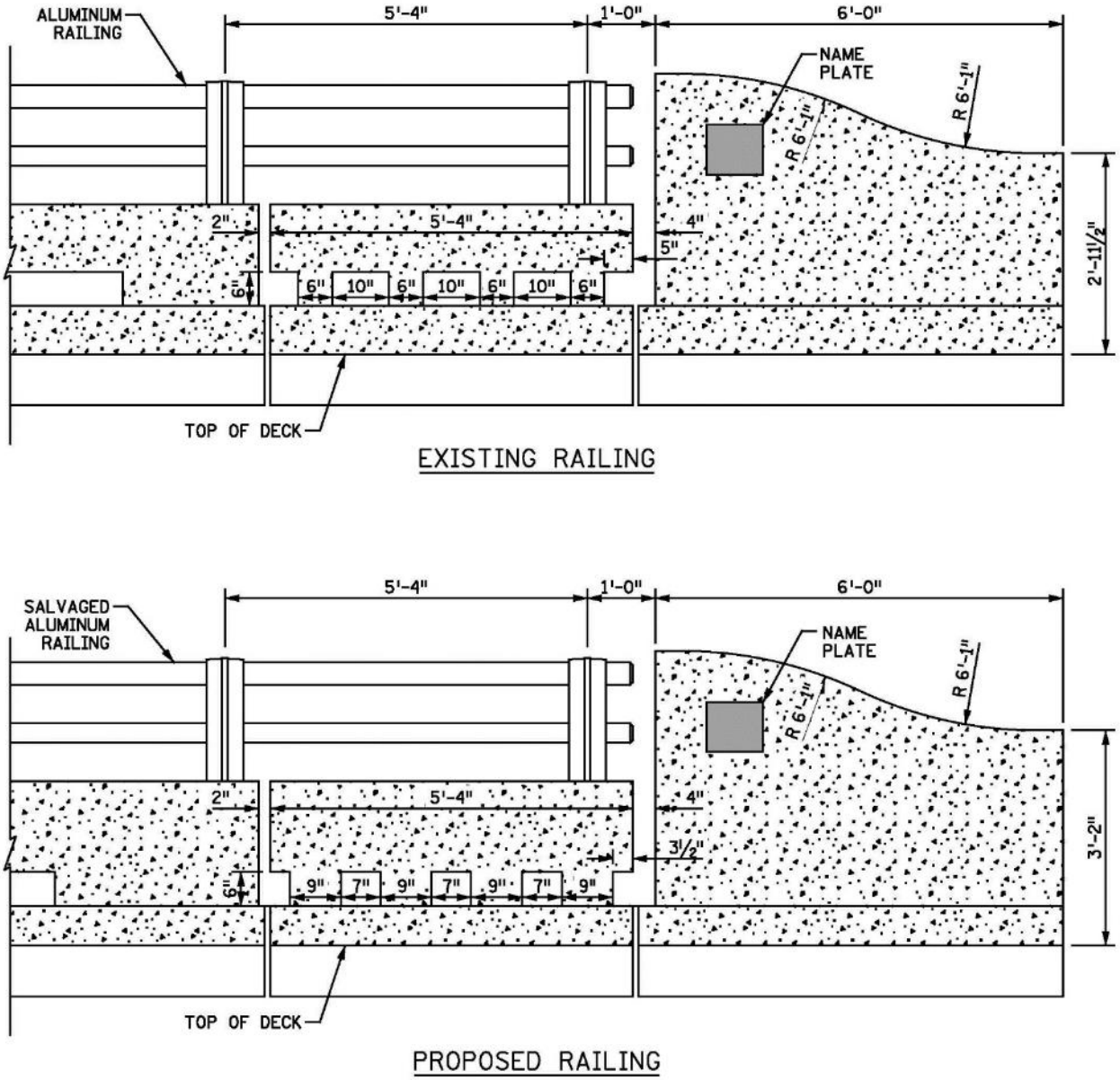
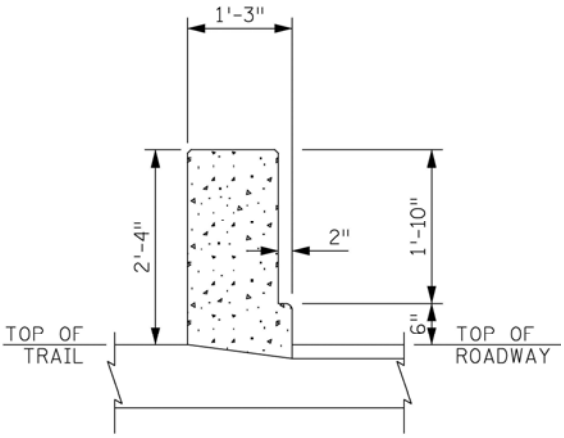


Figure 40. Bridge Name Plate

Traffic/Trail Separation Railing

An inner barrier would be installed to separate the new 8 foot-9 inch pedestrian/bicycle path on the north side of the deck from the adjacent roadway. Several options were explored in an effort to meet project needs while minimizing the visual impact of the new structure. It is recommended that a standard MnDOT P1 concrete parapet be used (Figure 41). It has a modest scale and simple design that is fairly neutral visually.

Figure 41. Traffic/Trail Separation Railing



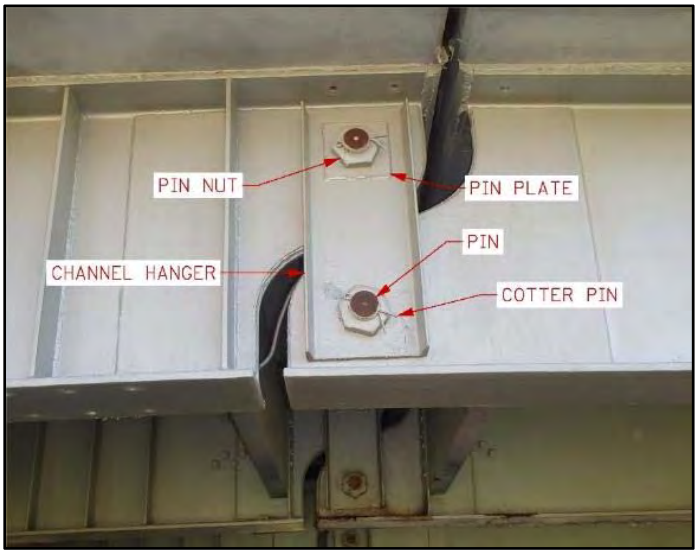
9.3 Pin and Hanger Assemblies

A pin and hanger assembly is located at each girder adjacent to piers 2-4 and piers 10-13 (56 total locations). The primary function of these assemblies is to transfer vertical forces between each approach span while allowing rotation and horizontal translation along the length of the bridge. The pins and hangers are among the set of elements designed to accommodate soil movement which collectively are a character-defining feature of the bridge. The components of the existing pin and hanger assemblies are shown in Figure 42. While the existing assemblies are in fair condition, preliminary analysis indicates that all of the channel hangers have an inventory rating that falls below 0.9 and do not have sufficient capacity to carry modern traffic loads. If one hanger assembly fails,

adjacent hanger assemblies would not have additional capacity to resist the increased loads and a progressive failure could result.

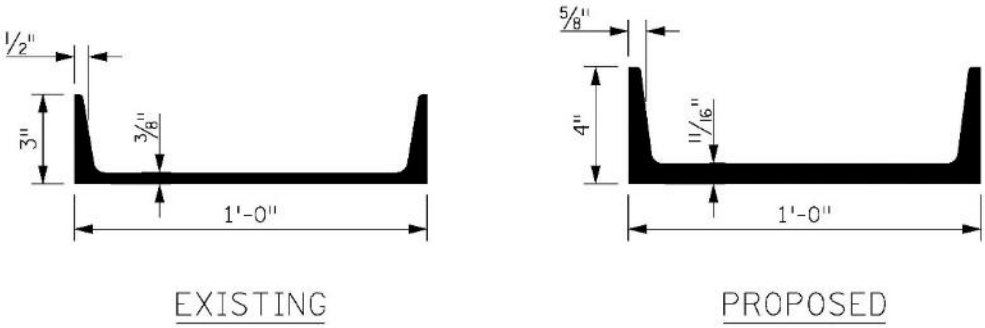
Since the hangers do not have sufficient capacity to carry modern traffic, and since failure of one hanger could result in the loss of an entire span, replacement of all hanger assemblies is recommended.

Figure 42. Typical Pin and Hanger Assembly



In order to preserve the historic integrity of the bridge, it is recommended that the new pin and hangar assemblies resemble existing as closely as possible. Currently, the channels used are classified as C12x30 steel shapes. In order to provide the required capacity with a shape that resembles the originals, a MC12x45 channel is recommended to be used as a replacement. Figure 43 compares the existing hanger channel dimensions and the proposed hanger dimensions.

Figure 43. Existing and Proposed Channel Hanger Sizes

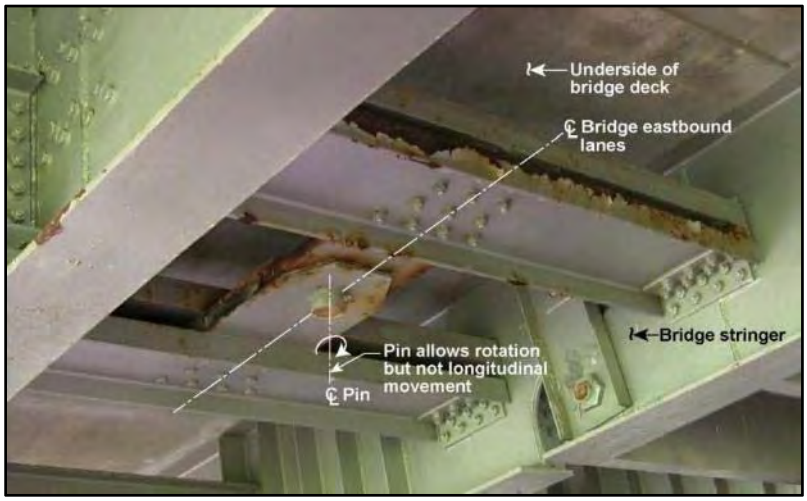


Due to the increased thickness of the proposed channel, longer pins will be required. In order to increase the durability and performance of the assemblies, stainless steel replacement pins and cotter pins are recommended. Replacement pin plates and pin nuts would have dimensions matching existing.

9.4 Swivel Assemblies

Swivel assemblies are located adjacent to all pin and hanger joints and at the connection of the approach spans to the truss spans (piers 6 and 8). Like the pins and hangers, they are part of the bridge's character-defining provision for soil movement. The primary purpose of the longitudinal joint and swivels is to accommodate differential transverse movements of the pier bents as soil moves perpendicular to the bridge centerline. As one pier moves relative to the adjacent piers, the swivels allow each deck segment to rotate horizontally (Figure 44). To achieve this function, a longitudinal joint (Figure 32) is required to allow one half of the bridge to translate longitudinally relative to the other half.

Figure 44. Typical Approach Span Swivel
(Source: Kennedy Bridge Planning Study Technical Memorandum dated 4/28/2014 prepared by CH2MHill)



Since removal of the longitudinal joint is recommended, the swivel assemblies will no longer be able to function as intended. An evaluation of the structural effects of removing the longitudinal deck joint is described in the technical memorandum found in Appendix I.

Swivels at Pier Bents

The evaluation described in Appendix I provides evidence that the typical pier bents (bents 2-5 and bents 9-13) have undergone little, if any, relative transverse movements. Additional analysis indicates that the structural effects of any future transverse soil movements resulting from removal of the existing longitudinal joint is not likely to be significant. Therefore, it is recommended that no modifications to the existing swivels at the pier bents be implemented due to removal of the longitudinal deck joint.

Swivels at Piers 6 and 8

The evaluation described in Appendix I provides evidence that the cast-in-place concrete piers (piers 6 and 8) may have undergone significant transverse movements in the past. If future movements occur at these piers and are not accommodated, significant stresses could be induced in pier bents 5 and 9. Transverse movements of less than 2 inches could impact the function of the finger joints or lock up hinge expansion bearings. Therefore, it is recommended that the swivel connections at piers 6 and 8 be modified to accommodate future pier movements.

In order to accommodate future transverse movements, the addition of a new swivel at the center of the bridge, at piers 6 and 8, is recommended (a total of two new assemblies). To permit rotation about the center of the bridge, removal of the pins in the four existing swivel assemblies would be required, but they would otherwise be unaltered. To avoid confusion during future inspection and to prevent future maintenance crews from reinstalling the pins, it is recommended that the existing pin holes be filled with a plug or weld.

Figure 45 shows the location of the existing swivel assemblies and the location of the proposed new swivel assembly. Figure 46 shows the proposed modifications to the typical swivel assembly located at piers 6 and 8 required to permit rotation about the center of the bridge.

Figure 45. Existing and Proposed Channel Swivel Locations
Pier 6 Shown, Pier 8 Similar

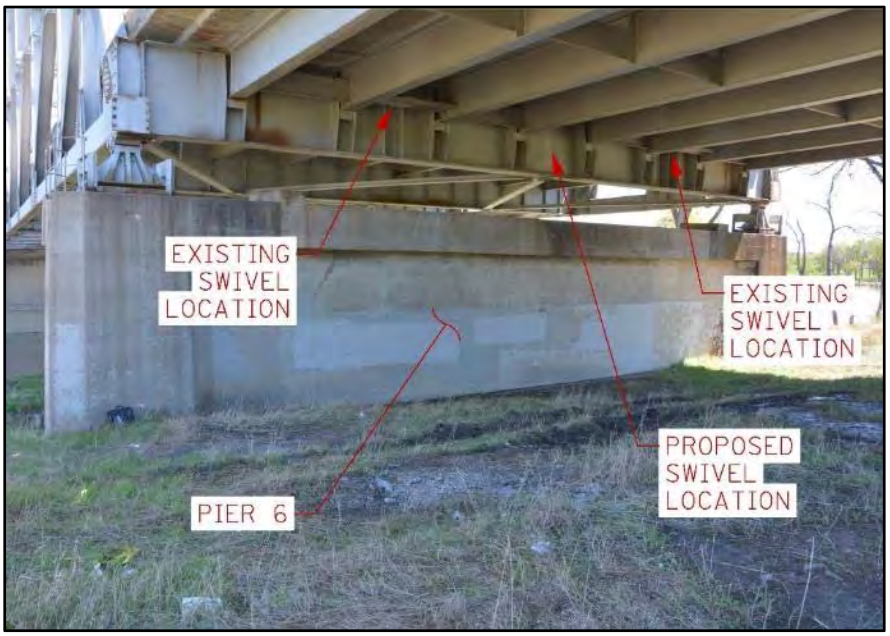


Figure 46. Existing Swivel Modification

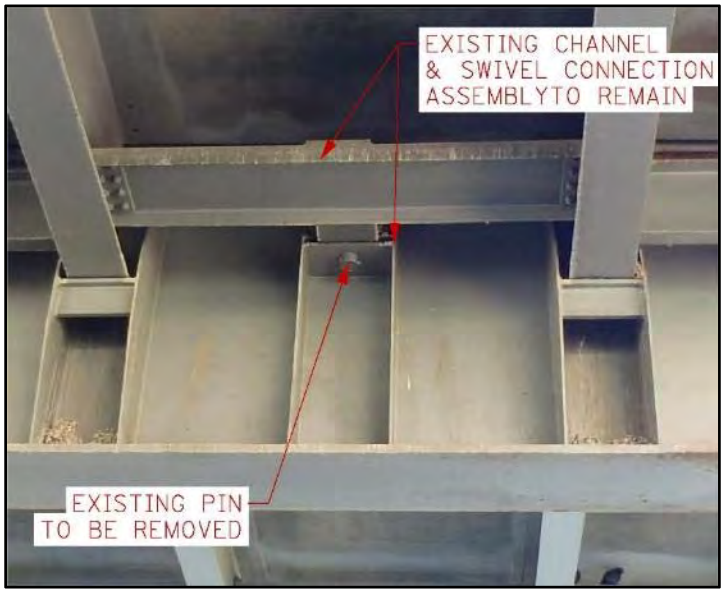
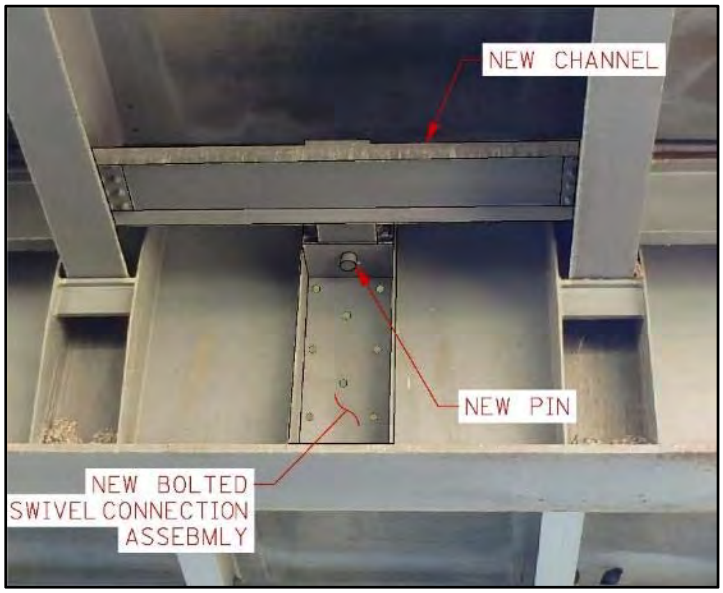


Figure 47 shows the new swivel proposed at the center of the bridge. The general form of the swivel assembly would be similar to the existing. However, the size of the pin, channel, and connection plate may be increased. The size of these components would be determined during the final design process.

While the existing assemblies were originally welded to the floor beam a new bolted assembly is recommended. The heat required to weld a new assembly could temporarily reduce the capacity of the floor beam. In order to maintain the need for maximum maintenance of traffic, the floor beam must retain sufficient capacity during repair operations.

Figure 47. Proposed Swivel



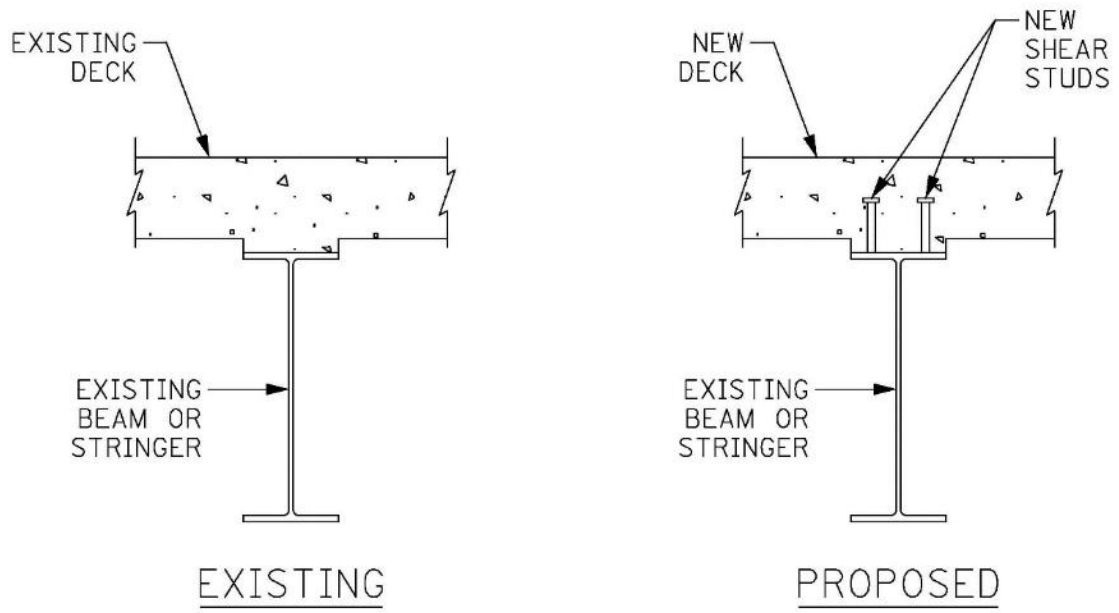
9.5 Truss Floor System Members

The truss floor system consists of stringers and floor beams as shown in Figure 7. There are 10 floor beams in each truss span (20 total floor beams) with eight stringers joining each floor beam (144 total stringers). The purpose of the stringers and floor beams is to support the deck and transfer loads to the truss. Preliminary analysis indicates that the existing floor system members do not have sufficient capacity to support modern traffic loads.

The existing floor system members were constructed to act independently from the bridge deck. Evaluation indicates that, if the bridge deck and the floor system members were to act together, rating factors would be increased to a level sufficient to support the required loads.

Figure 48 shows the recommended repair to allow the deck and concrete to work together. New shear studs would be welded to the top of the stringers and floor beams and encased in the new concrete deck. The new shear studs would not be visible and the original floor system members would be retained. The exact number and locations of the new studs would be determined in final design. The number of studs would be designed to minimize impacts to the existing structure while still providing the strength to support the required loads.

Figure 48. Stringer & Floor Beam Repair



9.6 Approach Span Pier Bents

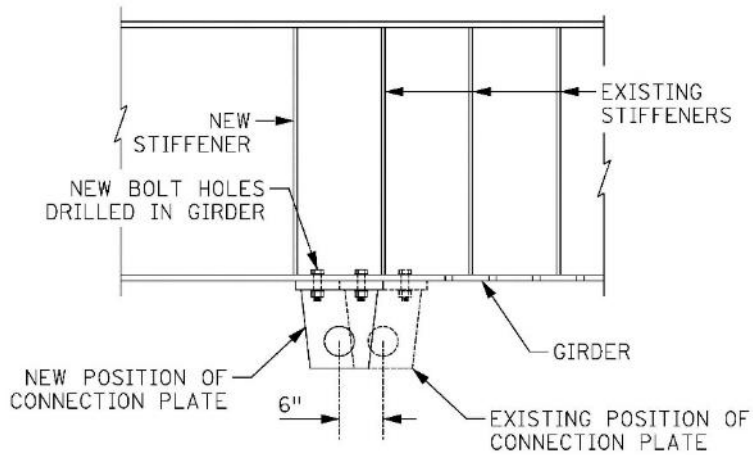
There are four bents (piers 2-5) located on the west approach and five bents (piers 9-13) located on the east approach. Each bent consists of a pair of assemblies (one supporting each direction of travel). Each pair consists of four columns attached to superstructure with a connection plate. The purpose of the approach span pier bents is to provide vertical support to the bridge superstructure while still permitting relative foundation movements or thermal expansion/contraction of the bridge deck.

Analysis indicates that approach span pier bents with a tilt angle greater than 3.7 degrees, will induce stresses into the piles in excess of the limits prescribed by AASHTO's *LRFD Bridge Design Specifications*. Table 2 indicates that both the north and south bents at pier 5 will have tilts in excess of 3.7 degrees during warmer temperatures and readjustment would be recommended.

Figure 49. Pier Bent No. 5 (looking north)

The bridge was originally designed to accommodate pier bent columns that tilt away from the river. However, the pier 5 columns tilt towards the river and require slight structural modifications to readjust. Figure 50 shows the recommended method for resetting the existing column connection plate 6 inches towards the abutment at pier 5. Two new holes would be drilled in the existing girders to connect the existing connection plate. To ensure the girder retains its existing structural capacity, a new stiffener would be welded to the existing girder west of the existing stiffeners.

Figure 50. Pier Connection Plate Repair



9.7 Abutments and Abutment Bearings

At both the east and west abutments each girder is supported by a pedestal style bearing (eight bearings at each abutment, 16 total). The function of the abutment bearings is to transfer vertical loads from the superstructure to the abutments while providing horizontal restraint for the approach span superstructure. Analysis shows that a horizontal reaction force on the bearings is caused by the tilting of the pier bents as shown in Figure 29. This reaction force is causing the bearing to tilt and the anchor bolts to pull out of the abutment concrete (Figure 51). Therefore, the ability of the bearings to provide horizontal restraint has been compromised. In order to ensure reliable horizontal restraint of the approach span superstructure, repair of the abutment bearings is recommended under this rehabilitation.

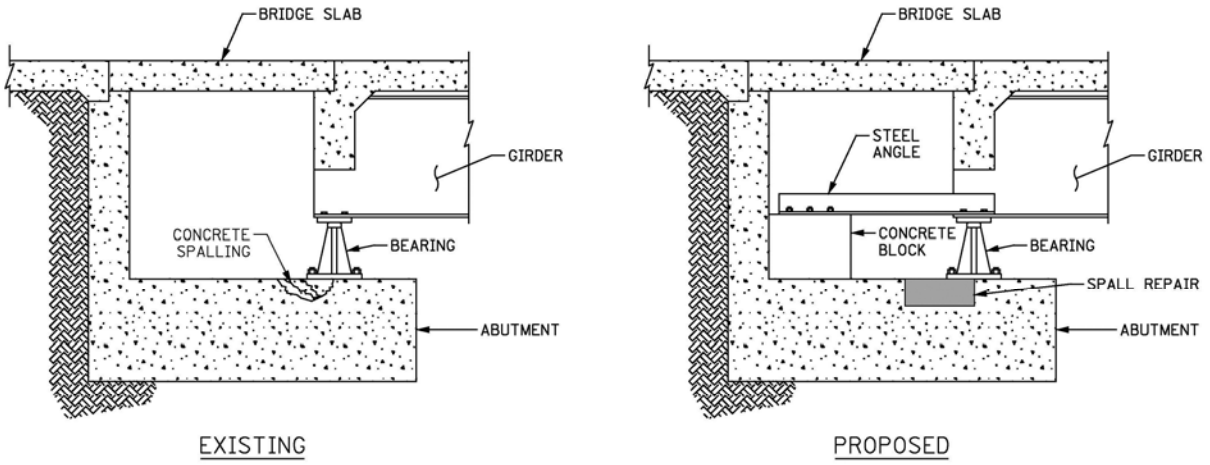
Figure 51. Abutment Bearings

The repair proposed under this rehabilitation involves providing longitudinal restraints behind each abutment bearing. The restraint would consist of steel angles connected to each girder that would transfer horizontal reactions to a new concrete block. The height of the concrete block would match the height of the existing bearings, while the size of the new steel angles and the width of the new concrete block would be determined in final design to resist the applied forces. Figure 52 compares the existing condition and the proposed repair.

In addition to providing the longitudinal restraints, repair of the unsound concrete behind the bearings is recommended. The repair would consist of removing the loose concrete and saw cutting a small pocket under and behind the bearing. This pocket would be filled with concrete (or shotcrete for shallow repairs). Depending on the size of the concrete spalling, temporary support of the girders may be required prior to concrete removal.

These repairs would be implemented below the bridge deck and would be hidden by the existing superstructure and abutment walls. The existing bearings would be retained and would be cleaned and repainted.

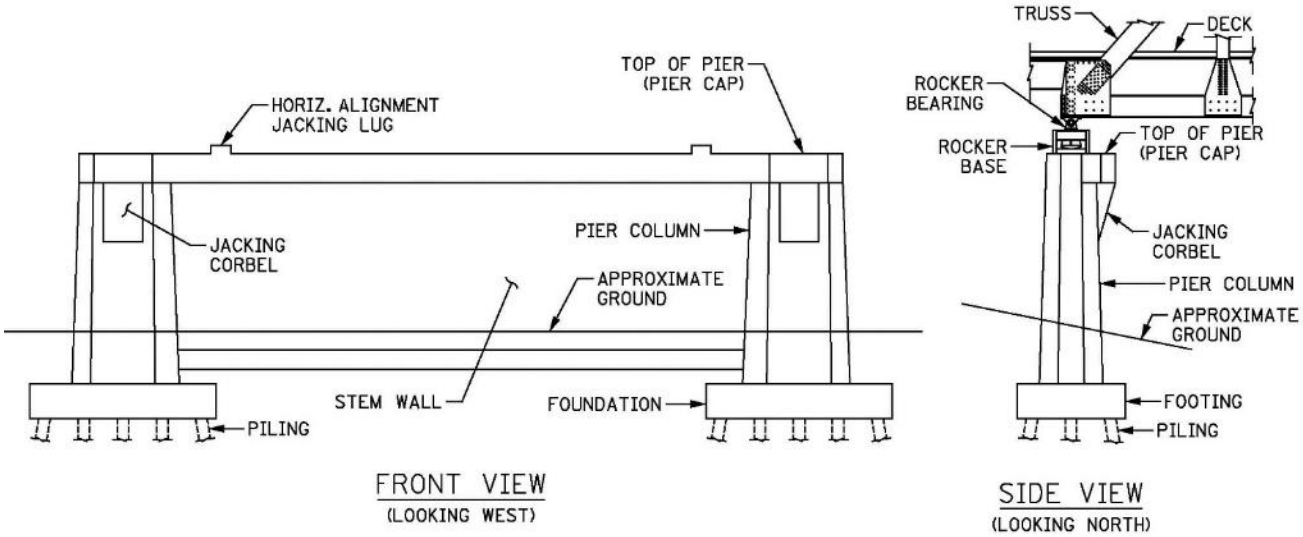
Figure 52. Abutment Bearings – Proposed Rehabilitation



9.8 Pier 6

Pier 6 is a mirror image of the cast-in-place concrete pier (pier 8) 560 feet away on the adjacent bank of the river. Additionally, pier 6 is very similar in construction to pier 7, 280 feet away in the center of the river channel. A diagram describing the components of pier 6 is shown in Figure 53. The purpose of the pier is to support spans 5 and 6 of the bridge. Vertical loads (structure weight, traffic loads, etc.) are transferred from the bearings to the pier caps and into the pier columns. Wind and other horizontal loads are transferred from the bearings to the rocker base and into the pier cap. The stem wall transfers load to each pier column and contributes to the stability of the pier columns.

Figure 53. Pier 6 Components



Pier 6 has undergone significant movement and is tilted (Figure 54). This movement has caused the stem wall to bow and crack (Figure 20 and Figure 21). Furthermore, analysis has shown that the stresses in the existing piling are exceeding the limits prescribed by AASHTO's *LRFD Bridge Design Specifications*. Since further movement of the pier could reduce the load carrying capacity of approach span 5 and truss span 6, repair or replacement of pier 6 is warranted. Since the priority for this rehabilitation is to preserve historic fabric, pier repair was investigated.

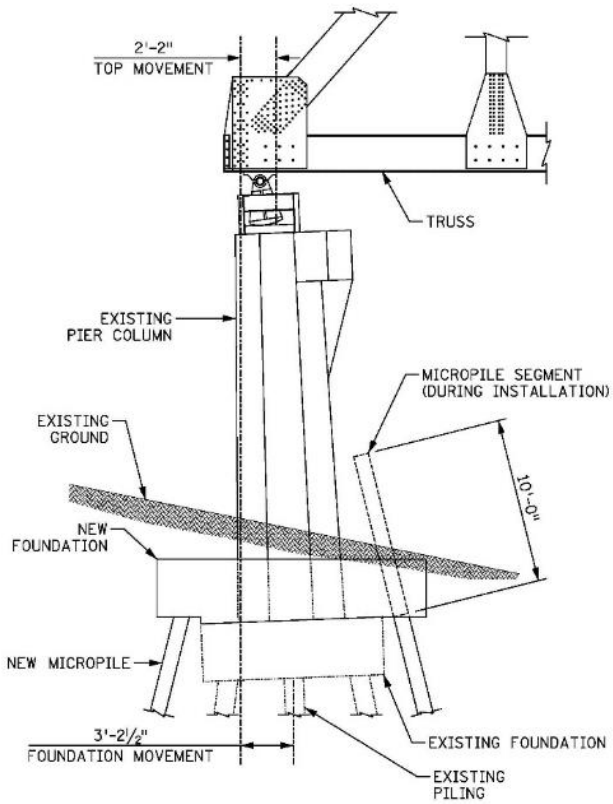
Figure 54. Pier 6 South Column Tilt (Looking North)

Pier 6 Foundation Repair

Any repair undertaken on pier 6 must address the fact that the existing foundation is no longer providing reliable vertical and horizontal support. To support the existing pier, underpinning the foundation with new piles would be required as shown in Figure 55.

Underpinning would involve installing the new piling at a slight angle (or batter) to provide both vertical and horizontal support. Since the existing foundation would remain in place, micropiles would be required in lieu of traditional piling as they can be installed in short, 10-foot-long segments that won't interfere with the existing pier. A new concrete foundation would be cast and anchored into the existing pier column above the existing foundation and just below the ground as shown in Figure 55. For the underpinning repair, the existing foundation and piling would remain attached to the pier column.

Figure 55. Pier 6 Foundation Repair



To provide an equivalent level of vertical support to the pier, a total of 16 to 25 micropiles (depending on size) would be required. Future soil movements would exert pressure on both the original and new piles and could increase the lateral force on the pier by 120-150 percent. This increased pressure could increase the rate of pier movement and cause the bearing to reach its maximum adjustment (as shown in Figure 31) in the near future. Therefore, it is recommended that pier 6 be replaced with a new cast-in-place concrete pier with footings that are not connected to the existing piling.

Pier 6 Replacement

While an in-kind replacement of Pier 6 would best preserve the bridge's historic integrity, it is recommended that the pier geometry be modified in two ways to address deficiencies in the original pier design:

Pier Width: To accommodate future movements of the replacement pier, maintenance personnel will need to periodically adjust the truss bearings (Figure 30 and Figure 31). These operations require placing jacks on the pier cap near the jacking corbels. Working space on the pier cap is very limited as shown in Figure 57, and using a man lift to improve access is often precluded because of high water or saturated soil due to flooding. MnDOT bridge maintenance staff have indicated that adding an extra foot of pier cap width toward the river would allow for safer access. The extra width would also allow maintenance crews to better spread out loads along the bottom chord of the truss during jacking procedures.

Two options for achieving the extra width while minimizing effects to the historic bridge were explored. Option 1 would widen the entire pier (pier cap, pier columns, and stem wall) by 12 inches. The widening would be symmetrical about the pier midline, with the size and shape of the jacking corbels matching those on the original pier. The center line of the new pier would be aligned 6 inches closer to the river than existing. Option 2 would achieve the extra width by only increasing the width of the jacking corbels. In both options the safety railings on the ends of the pier cap (Figure 57) would be closely resemble the original railings while accommodating the new geometry. Comparison of the two options is shown in Figure 56.

An evaluation of the visual impacts determined that Option 2, which widened only the jacking corbels, significantly altered the aesthetics of the replacement pier when compared with the original. With Option 1, the visual change was less dramatic; the proportions more closely resembled the original pier if all elements (excluding the jacking corbel) were widened.

Bottom of Stem Wall: Pier 6's existing stem wall is buried approximately 7 feet when measured from the ground line on the non-river side of the pier. Preliminary analysis of the effect of the moving soil indicates that approximately 25 percent of the lateral force on the pier (which leads to pier movement) is due to soil pressure on the stem wall. Figure 54 shows how the soil builds up on the non-river side of the pier, thus increasing the lateral force. Raising the bottom of the stem wall by approximately 3 to 4 feet could reduce the lateral force thus decreasing future pier movement and extending the life of the replacement pier. Because soil could better 'flow' under the raised wall, there would be less differential in the ground line between the river and non-river sides of the pier. At its new elevation, the bottom of the stem wall would only be visible if severe flooding lowered the ground line on the river side of the pier. The stem wall bottom would only be exposed for a short period of time as MnDOT maintenance staff periodically fills in the ground line in front of the pier after flooding events.

The recommended geometry of the replacement pier is shown in Figure 58.

Figure 56. Pier Cap Widening Options

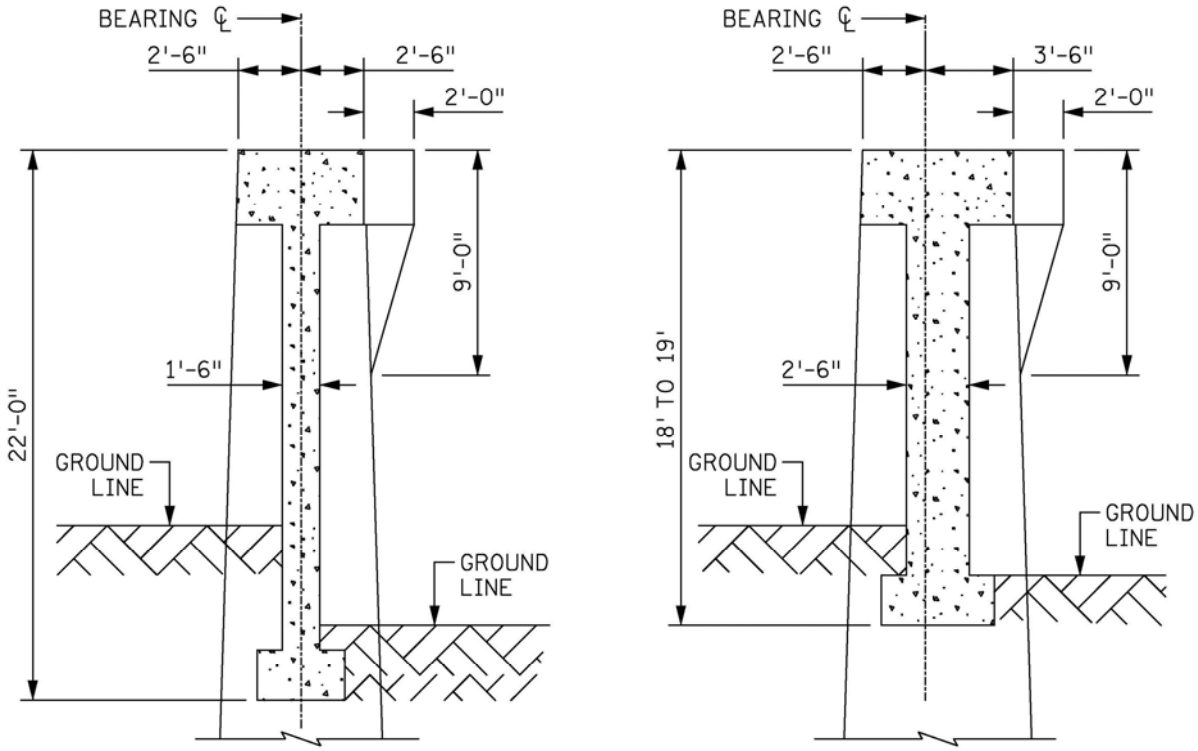
EXISTING PIER:

CAP WIDENING - OPTION 1:
(entire pier widened 1'-0")

CAP WIDENING OPTION 2:
(jacking corbel widened 1'-0")

Figure 57. Pier Cap Working Space and Safety Railing

Figure 58. Existing vs. Proposed Pier Dimensions



EXISTING PIER **PROPOSED PIER**

9.9 Drainage System

The existing deck drainage system would be replaced with the deck. As designed, the existing system succeeds in removing water from the relatively level deck. However, the deck drains are also a significant source of deterioration on the bridge as they allow water and deicing chemicals to access structural steel members.

The existing drainage system consists of 4 inch x 6 inch pipes embedded in the deck at an angle as shown in Figure 59. The original deck drains are spaced approximately 15-16 feet apart along the gutter line (the interface of the bridge deck and the safety walkway).

The proposed geometry of the new deck would require the configuration of the new deck drains to be modified from the original. In the new cross section, the southern gutter line would be located 30 inches farther south than existing due to the elimination of the safety walkway. The northern gutter line would be located at the barrier between the roadway and the pedestrian/bicycle trail.

Figure 59. Existing Deck Drainage

APPROACH SPAN DRAIN

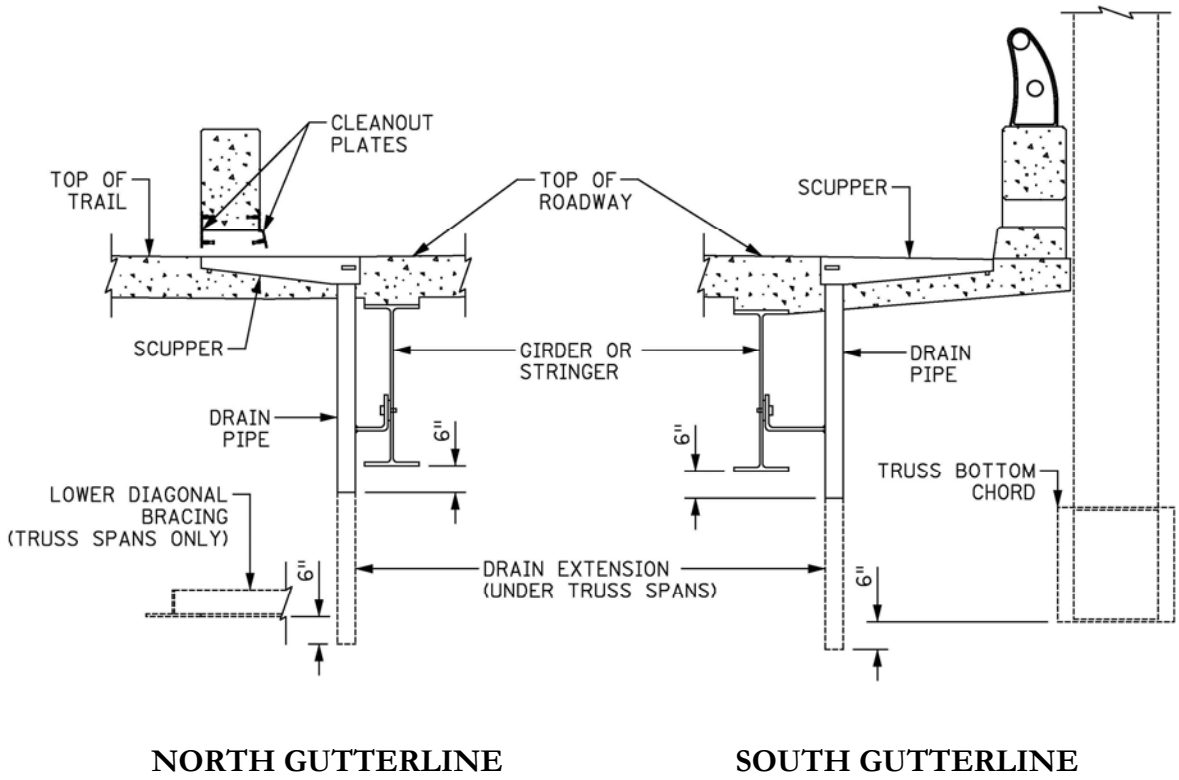
TRUSS SPAN DRAIN

The configuration of the proposed deck drains are shown in Figure 60. A scupper style deck drain is proposed to minimize the number of drains. The drain pipes would be extended approximately 6 inches below the bottom of the bridge girders and truss members (about 24 inches lower than the existing pipes on the approach spans and about 4 inches lower than the existing pipes on the truss spans) to prevent water and salt from deteriorating the structural steel.

The scuppers on the north side of the bridge would be located below the barrier separating the roadway and pedestrian/biycle trail. To facilitate drainage of the trail, small openings in the railing would allow water to flow into the drains. To facilitate maintenance when drains become clogged, removable cover plates would be placed over the gaps in the barrier.

The size and spacing of the drains will be determined during the final design process. In order to minimize the number of drains, the rectangular pipe elements may be slightly larger than the existing drainage pipes.

Figure 60. Proposed Deck Drainage



9.10 Lighting

Traffic and Trail Lighting

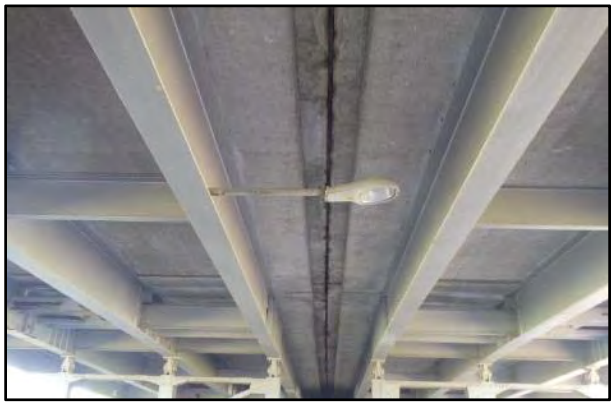
The bridge has four pole-mounted lights, all original. The two pole mounted lights on the east approach spans are approximately 30 feet above the roadway whereas the lights on the west approach spans are approximately 40 feet above the roadway. While the lighting is, on average, bright enough for traffic, the distribution of the light is inconsistent on the east approach spans. To reduce the differential between the bright and dim spots on the east approach, it is recommended that the two light poles be replaced with poles that will put the lights approximately 40 feet above the roadway. The existing mast arms should be reused (if compatible with the new fixtures). The existing mounting brackets (Figure 61) should be reused or modified to fit the new poles and to attach to the new bridge railing.

To provide a lighting source that is energy efficient and dark sky compliant, it is recommended that the lights be replaced with LED fixtures. Various fixture styles were explored and the Autobahn Series ATBM LED Light Fixture (Figure 62) was determined to most closely resemble the existing fixtures.

Figure 61. Existing Bridge Lighting

Truss Span Light

Approach Span Light



Trail Light

Light Pole Mounting

Figure 62. Autobahn Series ATBM LED Light Fixture

Aesthetic Lighting

Local communities have requested that the trusses be aesthetically lit at night to create a visual landmark linking the two cities and to celebrate the Red River corridor which was reclaimed as public green space after the 1997 flood. The bridge was not lit in this way historically. To reduce the degree to which the historic integrity of the bridge and nearby historic properties are diminished, it is recommended that the aesthetic lighting be modest, that the fixtures and conduit be inconspicuous, and that the fixtures and conduit be removable and installed without altering or damaging the steel trusses. A lighting scheme that is brighter or more complex than is needed to simply highlight the architecture should be avoided, and the lights should not change, "move," or flash. White light is preferred to protect historic integrity of feeling and association, but local communities have requested the use of colored lights, for example, to celebrate holidays. To accommodate this desire and minimize the degree to which historic integrity is affected, it is recommended that white light be used most of the time and, when colored lights are used, the scheme be single-colored (e.g., all blue lights).

The recommended lighting scheme (shown in Figure 63 and Figure 64) was designed to meet the above parameters, avoid potential flood damage, and accommodate maintenance needs. It would gently wash light on the vertical and diagonal members of the north and south trusses as well as on portal members at the four truss entrances. LED fixtures would be used to conserve energy and reduce maintenance. Light levels would be adjustable to avoid inadvertent over-lighting of historic properties. Fixtures would be hidden in gusset plate areas and metal conduit would be hidden as much as possible. An agreement between MnDOT, NDDOT, and local communities would predetermine a schedule and colors for seasonal or special occasion lighting.

Figure 63. Aesthetic Lighting – Exterior Truss View

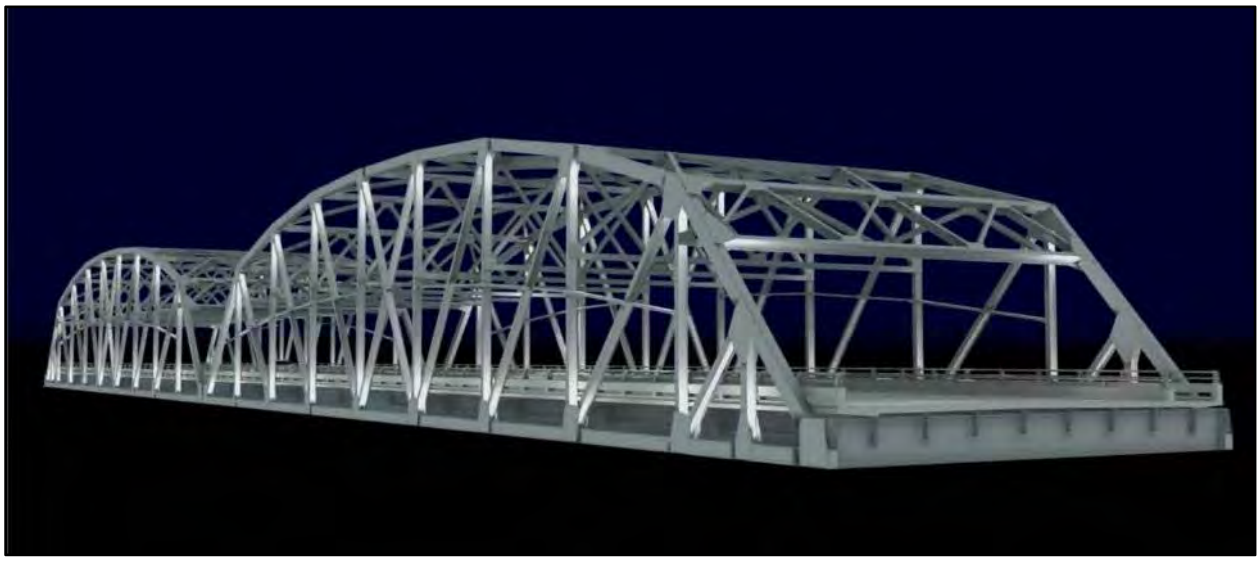
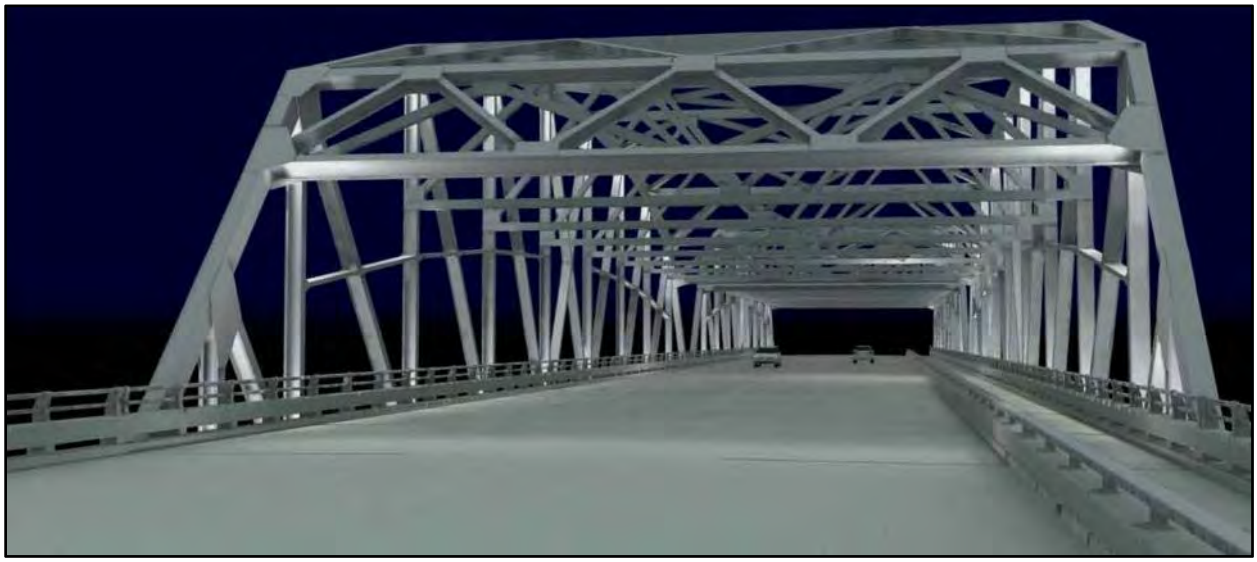


Figure 64. Aesthetic Lighting – Interior Truss View



9.11 Pigeon Abatement

Approximately 1,740 perforations were provided on the truss chords, end posts, and sway bracing to allow welders access during fabrication of the truss (see Figure 65 and Figure 66). These perforations have allowed the truss members to become infested with pigeons. Pigeon droppings are corrosive and can result in steel section loss. Additionally, the pigeon droppings interfere with bridge inspections and are a health hazard to bridge inspectors and maintenance crews.

As part of a 1996 painting project, plastic covers were placed over the perforations. These covers have deteriorated and are now allowing pigeons to access the inside of truss members (Figure 67). To prevent further steel deterioration and to facilitate future inspections, installation of a system to prevent pigeon access is recommended.

Figure 65. Sway Bracing Perforations

Figure 66. End Post Perforations

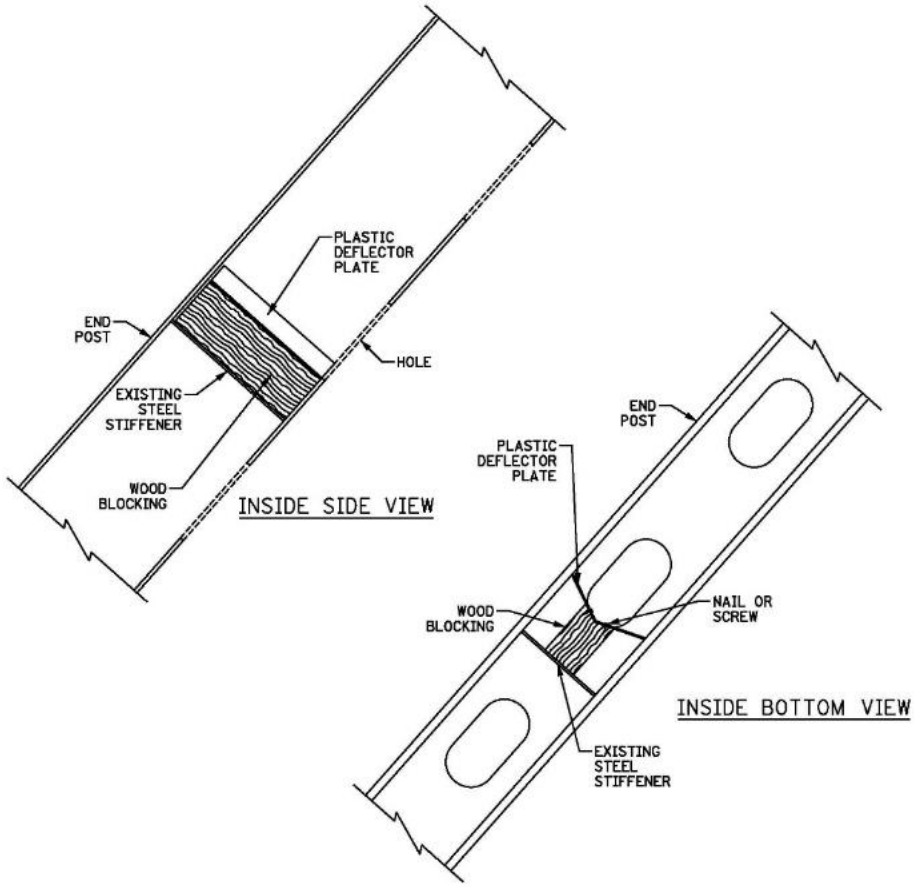
Figure 67. Deteriorated Cover and Pigeon Droppings

The truss member perforations are a distinctive feature of the Kennedy Bridge and represent construction methods common in the 1950s and 1960s. However, the majority (approximately 1,630) of the perforations are either below the deck (in the bottom chord) or 30-40 feet above the top of the deck (top chord and sway bracing) and, therefore, less visible to the public. To prevent pigeon access at these locations, installation of a removable, dark gray, plastic cover is recommended. Covers similar to those that have been used on MnDOT Bridge 9030 (the Blatnik Bridge in Duluth) are recommended (Figure 68).

Figure 68. Blatnik Bridge (MnDOT No. 9030) Cover Plate

The 112 endpost perforations are more visible from the bridge deck and, therefore, would remain uncovered. To prevent buildup of pigeon droppings, placement of a 'deflector plate' inside the truss members is proposed as shown in Figure 69. In two locations in each of the 8 end posts, a deflector plate would rest on the two, existing, internal stiffeners that are located at the lower end and the middle of the end post. The plates would direct any droppings out of the perforations. They would also prevent pigeons from nesting on the stiffeners. These plates would be hidden inside the members.

Figure 69. End Post Pigeon Waste Deflector



9.12 Signage

State Welcome Signs

At the outer end of each of the two truss spans is a modestly-sized state entry welcome sign (Figure 70 and Figure 71. It is recommended that these signs be removed to repaint the truss and reinstalled in their approximate current locations. To protect the historic integrity of the bridge, it is recommended that any portal signage be limited to a single flat steel sign no larger than the existing signs.

Memorial Plaques

The two original, aluminum, John F. Kennedy memorial plaques (Figure 72) should be covered during painting operations, or removed and reinstalled in their original locations. They should not be cleaned to retain their patina of age.

Traffic/Roadway Signs

FHWA's *Manual on Uniform Traffic Control Devices* requires that fixed objects directly adjacent to the roadway be marked with a 12-inch x 36-inch, Type 3 Object Marker (Figure 73). To comply with these provisions, it is recommended that these signs be placed at the four locations where the railing adjacent to the roadway terminates. These signs would be placed such that the bottom of the object marker is 4 feet above the roadway surface.

Figure 70. Welcome Sign (North Dakota)

Figure 71. Welcome Sign (Minnesota)

Figure 72. John F. Kennedy Memorial Plate

Figure 73. Type 3 Object Marker

9.13 Surface Coatings and Treatments

Steel Superstructure

In order to protect the structure from future corrosion and to provide an acceptable design life, it is recommended that the existing paint system and any flaking steel corrosion be removed with gentle sandblasting that will not damage the steel. It is also recommended that any accessible pack rust be routed out between gussets and connection plates. A multiple coat, zinc-rich paint system would be applied to all steel members of the rehabilitated bridge including all truss members, gusset plates, girders, pier bents, stringers, floor beams, lateral bracing, and bearings. The gussets and connection plates on the bottom chord should be sealed from moisture after painting with a caulking compound.

The color of the paint system should match the color of the existing paint system which is consistent with the original color.

Deck and Railings

The original deck and concrete railings are uncoated. The concrete portion of the railings received a "Rubbed Surface Finish" according to the original construction plans. It is recommended that the new deck overhangs and the concrete portions of the inner and outer railings remain uncoated and receive a "Sack Rubbed Surface Finish" (similar to the original finish) in accordance with MnDOT specifications. A clear, silane sealer that will not alter the color of the concrete is recommended on railing surfaces to prevent any corrosion of the barrier reinforcement that would be initiated by deicing chemicals. The underside of the deck should be formed with plywood sheets to match the original deck.

It is recommended that all new aluminum posts be finished by grinding off any mold lines and sandblasting in accordance with the original construction plans.

Pier 6

No surface coatings were applied to the original, pier 6 concrete. Although the new pier 6 concrete will not exactly match the concrete color of pier 7 or pier 8, the surfaces are not adjacent to one another and slight color differences would not be noticeable. However, the color of newer concrete mixes may be a whiter, or brighter gray, color than concrete used in the 1960s. Therefore, it is recommended that pier 6 be constructed with an integrally colored concrete. The integral color should create a gray color that is more typical of aged concrete.

The original pier 6 was constructed using forms constructed on long wood boards. Therefore, it is recommended that the replacement pier 6 be constructed with board forms placed in a pattern that matches the existing pier.

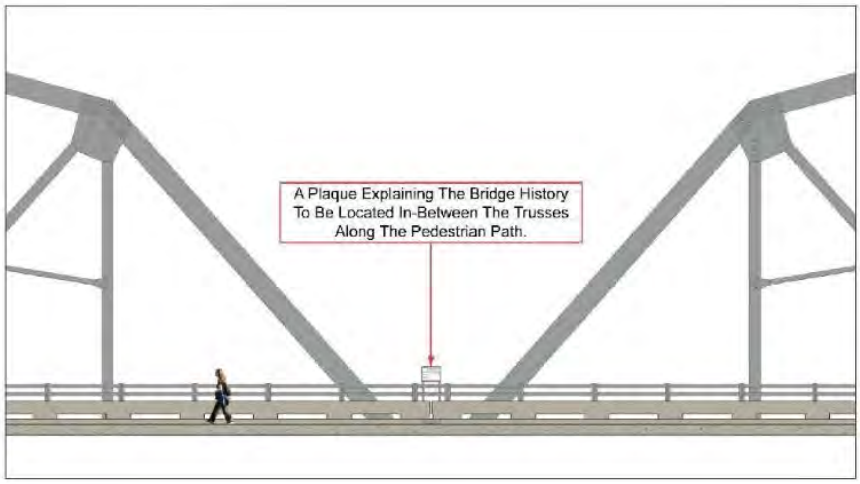
9.14 Signature Features

In addition to aesthetic lighting (see Section 9.10 above), local communities requested other "enhancements" to increase public interest in, and appreciation for, the rehabilitated bridge. To

reduce impacts to the historic property, two modest enhancement elements were chosen by MnDOT, NDDOT, and local communities, while several other options were eliminated from consideration because of the degree to which they affected historic integrity.

The first element recommended is a single interpretive plaque mounted on the north railing between the two trusses (Figure 74). Located along the shared use path, it would explain the history and significance of the bridge. The plaque would be appropriately scaled and would be mounted in a way that balances accessibility needs with bicycle/pedestrian safety, does not damage historic fabric, and is removable.

Figure 74. Interpretive Plaque

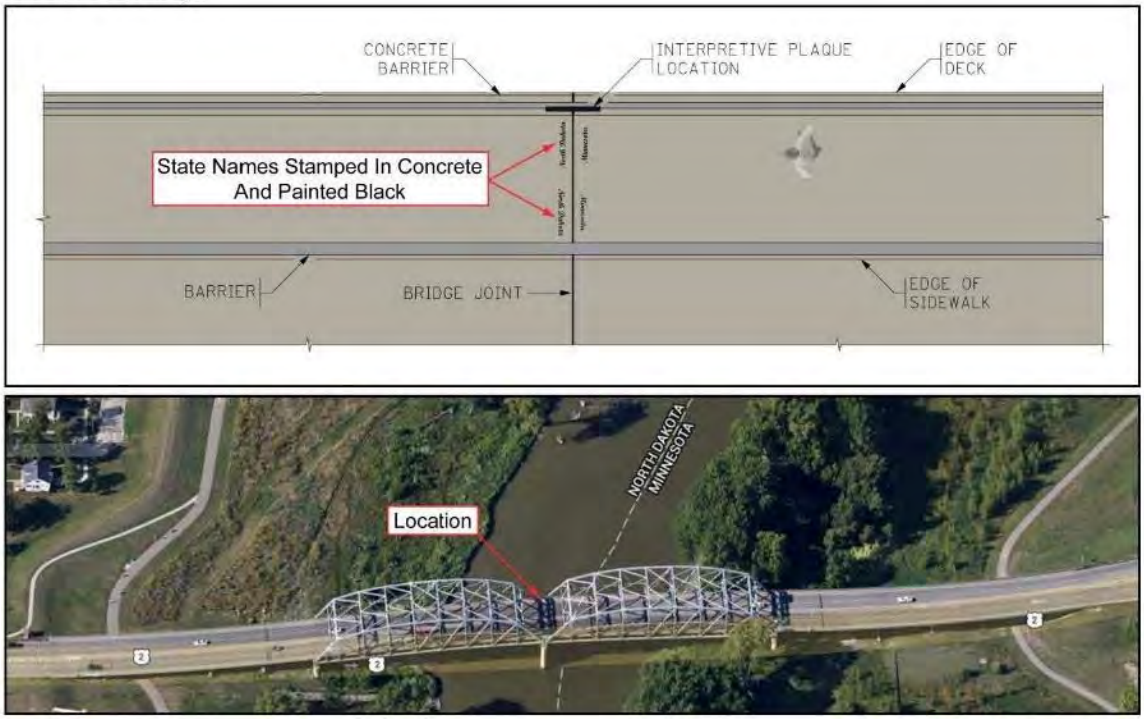


The second element recommended is a set of black letters stamped into the new concrete surface of the shared use path (similar to that shown in Figure 75). The scheme would be symmetrical with lettering in three locations. The words "Minnesota" and "North Dakota" would appear in the path at the center of the bridge (the approximate state line) and "Welcome to Grand Forks" and "Welcome to East Grand Forks" would appear just outside the ends of the each truss span. To protect historic integrity, the lettering would be modest in scale and color.

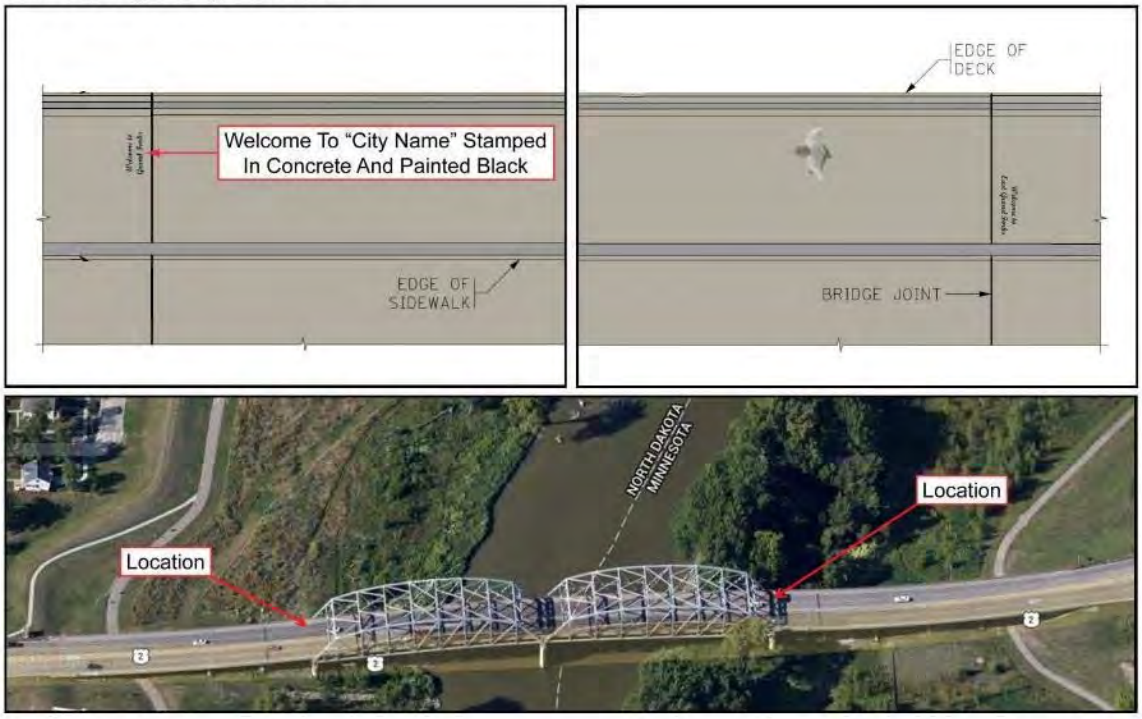
Figure 75. Stamped Border/Entrance Markings

Figure 76. Signature Features Locations

Border Crossing



End of Bridge / City Entrance



9.15 Off Bridge Changes

Trail

To provide access to non-motorized traffic, it is recommended that the 8-foot 9-inch trail on the bridge be extended on both the west and east approaches to the bridge. The trail on the east side of the river would follow the US2/4th Street entrance ramp and terminate outside the historic boundaries of the Kennedy Bridge. The trail on the west side of the river would tie into the existing sidewalk on the north side of US 2, approximately 70 feet west of the bridge and outside the boundaries of the Riverside Neighborhood Historic District.

Approach Roadway

During flood events, a low point on US 2, just east of the bridge, is susceptible to road closure. Reconstruction of the road to minimize this low point is recommended as the Kennedy Bridge provides the only connection between Grand Forks, North Dakota and East Grand Forks, Minnesota during heavy flooding. These modifications would be outside the historic boundaries of the Kennedy Bridge.

10.0 CONCLUSIONS AND RECOMMENDATIONS

10.1 Summary of Recommended Rehabilitation

A summary of the recommended rehabilitation and associated service life for each element of Bridge No. 9090 is given in Table 6. Unless noted otherwise, the expected service life of individual repairs is taken as that defined by MnDOT's *Fiscal Year 2016 through 2020 Bridge Preservation and Improvement Guidelines*.

The service life listed for each repair is based on an anticipated use of the bridge consistent with projected traffic patterns and typical weather patterns in Minnesota. Increased traffic volumes, increase traffic loads, significant traffic impacts, extreme weather events, flooding, and other acts of god may reduce the expected service life of a given repair.

Table 6. Recommended Rehabilitation Summary

Feature	Recommended Rehabilitation	Expected Service Life (years)
Deck	• Replace 7-inch concrete deck with a new cast-in-place deck that is 9 inches, tapering to 7 inches at the edges • Add an 8-foot 9-inch at-grade pedestrian/bicycle path on the north side only • Remove longitudinal joint and median	50+
Deck Joints	• Replace strip seal joint and finger joints with similar joints	25
Railings	• Replace exterior railings with a Nebraska open concrete railing • Salvage and reinstall existing aluminum rail pipes and post • Replace curved end posts with a similar design • Add a new inner rail to separate pedestrian/bicycle path from the roadway	20-30
Pin and Hanger Assemblies	• Replace existing C12x25 hanger channels with new MC12x45 channels • Replace existing pins with stainless steel pins • Replace pin nuts and pin plates in-kind	50+
Swivel Assemblies	• Install a new swivel assembly between the center girders at piers 6 and 8 • Remove pins from the four pier 6 and 8 swivel assemblies and plug pin holes	50+
Truss Floor System Members	• Add new shear studs to top of existing stringers and floor beams (to be encased in the deck)	50+
Approach Pier Bents	• Move existing connection plates west 6 inches at pier bent 5	See note ($)
Abutments	• Remove and patch concrete spalls under and behind abutment bearings	50+
Abutment Bearings	• Install concrete block/steel angle restraints behind existing bearings	50+

Feature	Recommended Rehabilitation	Expected Service Life (years)
Pier 6	• Replace the pier with a new, cast-in-place concrete pier, widened by 1 foot • Raise the bottom of the stem wall by 3-4 feet (bottom of wall to remain buried) • Install pier cap safety railings that match existing	See note ($)
Drainage System	• Replace the existing deck drainage system with a system that will minimize the number of deck drains • Utilize scupper style drains to increase drainage capacity	50+
Lighting	• Replace two traffic lights on the east approach spans with similar poles that raise the fixture 10 feet • Replace traffic light fixtures with a new, LED fixture that most resembles the existing fixtures • Add aesthetic wash lighting to truss vertical, diagonal, and portal members	20 See note (#)
Pigeon Abatement	• Add removable, dark grey, plastic covers over the perforations of all truss members, excluding the end posts • Install 'deflector plates' inside truss end posts	10-15 See note (◊)
Signage	• Reinstall or replace the state welcome signs with a single sign no larger than existing • Install Type 3 object marker signs at the ends of railings adjacent to the roadway	12-30 See note (&)
Steel Surface Coatings and Treatments	• Gently sandblast existing steel to remove paint • Repaint steel bridge members with a paint matching the existing/original paint color	15-25
Concrete Surface Coatings and Treatments	• Provide a "sack rubbed surface finish" on concrete surfaces of the new inner and outer railing and the deck overhangs • Construct pier 6 using integrally colored, gray concrete using board formwork	20-30
Signature Features	• Add interpretive plaque between the truss spans to the north railing • Add a black, stamped concrete border crossing marker in the trail between the truss spans • Add a black, stamped concrete city welcome marker in the trail just outside the ends of each truss span	NA
Off Bridge Changes	• Extend trail to 4th Street/River Road on the east side of the bridge • Tie trail into existing sidewalk on the west side of the bridge • Raise the low point on the westbound road, east of the bridge	NA

($) The service life of the chosen foundation system will be a function of the pile type and layout; the magnitude, frequency, and duration of future flood events; changing soil conditions due to these events; and future construction within the vicinity of the Kennedy Bridge.

(◊) Based on typical manufacturer's warranty for UV resistant polycarbonate sheets in exterior applications.

(#) Lighting service life is based on an average manufacturer's stated fixture life of 100,000 hours operating at an average of 12 hours per day.

(&) Sign life from *Minnesota's best practices of Traffic Sign Maintenance/Management Handbook*

10.2 Compatibility with the Secretary of the Interior's Standards for Rehabilitation

The proposed rehabilitation has been designed to be consistent with the Secretary of the Interior's Standards for Rehabilitation and accompanying Guidelines. (See Appendix D for the Rehabilitation Standards.)

The property would be used as it was historically, meeting Rehabilitation Standard 1. New uses, namely the addition of a new pedestrian/bicycle path, would require only minor changes to the bridge's distinctive materials, features, and spatial relationships.

In general, the historic integrity of the bridge would be preserved per Standard 2. With the exception of the railing, the removal of distinctive features would be largely avoided. The replacement railing has been designed to diminish the bridge's historic character as little as possible. Consistent with Standard 5, the bridge's distinctive materials, engineering features, and finishes will be retained.

Per Standard 6, deteriorated historic features would be repaired, rather than replaced, wherever possible. All repairs would significantly extend the life of the historic structure. The deck and pier 6 replacements were designed to resemble the originals in design, materials, color, and texture.

Consistent with Standard 7, repair methods such as sandblasting of the steel will be as gentle as possible.

New additions – the pedestrian/bike trail, inner railing, aesthetic lighting, interpretive sign, and trail lettering – would be compatible with the bridge's materials, features, size, scale, and proportions per Standard 9. If needed, the additions could be removed in the future, making these changes reversible.

Per the Rehabilitation Guidelines, health and safety upgrades – that is, safely accommodating pedestrians and making the outside railing crashworthy – will occur in a way that does not interfere with the bridge's character-defining spaces, features, and finishes.

Further details on compatibility with the Standards and Guidelines are summarized below.

Deck Replacement

The existing deck exhibits, cracking, delamination, chloride saturation, and corrosion of the steel reinforcement. Given the life expectancy of repairs, and because chlorides cannot be removed and deterioration of rebar cannot be reversed, it was determined that deck replacement rather than repair is warranted. Replacement of the deck would alter a feature that comprises a significant amount of historic fabric in highly-visible location and would not be reversible.

Because of the need to add pedestrian facilities to a deck whose width is constrained by the existing trusses, the new deck would not have a center median, longitudinal joint, cantilevered edge treatment, or symmetrical cross-section. To help visually mitigate the loss of the cantilevered edge treatment, the new 9-inch deck would taper to 7 inches at the edges. To help preserve the bridge's

historic character, the deck edges would be finished with a "Sack Rubbed Surface Finish" similar to the original, the deck underside would have form lines resembling those of the original, and the concrete would remain unpainted.

The new deck would have a new pedestrian/bicycle path on the north side only. In terms of the Rehabilitation Standards, the new path would be an expansion of use but also a health and safety upgrade since the bridge is now closed to pedestrians. (The 28-inch-wide safety walkways are too narrow for safe use.) The inner rail separating the new path from the roadway would have a visually neutral design that does not distract from the rest of the bridge. The concrete portion of the new rail would be coated with clear silane sealer rather than being protected from deicing chemicals with an opaque coating.

Railing Replacement

Preserving the historic railing is not feasible because it is structurally integrated with the deck and must be removed when the deck is replaced. Replacement of the railing would alter a character-defining feature that, from many locations, is one of the most visually dominant design elements. The change would not be reversible. In terms of the Standards and Guidelines, the replacement could be considered a health and safety upgrade since the current rail does not meet modern standards for crashworthiness.

The change was designed to be as least disruptive to the bridge's historic integrity as possible. While taller and bulkier than the concrete portion of the current rail, the modified Nebraska Open Rail meets TL-4 test levels, generally resembles the original rail, and would reproduce an important aspect of the original rail's design – the longitudinal spacing of openings to posts near the base.

To retain as much historic fabric as possible, the aluminum portion of the historic railing would be salvaged and remounted. The new railing's end posts would be curved like the originals and no guard rail would be used. To preserve the bridge's unpainted concrete finish, the concrete portion of the new rail would be coated with clear silane sealer rather than an opaque coating. A new bridge plate indicating the rehabilitation date would be mounted on an end post to help differentiate new work from old.

Pier 6 Replacement

The replacement of pier 6, which has tilted, bowed, and cracked, would alter an important component of historic fabric in highly-visible location and would not be reversible. Repair of the pier was explored but determined infeasible. To protect the bridge's historic integrity, the new pier would be as close to an in-kind replacement as possible but with two changes: First, the bottom of the stem wall, which is currently buried about 7 feet, would be raised 3 to 4 feet to relieve lateral pressure by allowing soil to move under the wall toward the river. The change would be made below grade and, except temporarily after extreme flood events, the bottom of the raised wall would not be visible. Second, the pier would be widened about 1 foot from existing to provide safer access to workers who need to operate on top of the pier cap to periodically jack up the truss and reset the

bearings. To help the new pier match the original as closely as possible, it would be made of integrally colored concrete, would have a board form finish, and would be unpainted.

Provisions for Bridge Movement

There would be some alteration of the bridge's character-defining provision for movement. The replacement pins and hangars would be as close to in-kind replacements as possible. The longitudinal joint would be removed. Two new swivel assemblies would be added and the pins removed from four assembles to make them nonfunctional. The pier 5 bent columns would be reset with two new holes drilled and small stiffeners added. The abutment bearings would be repaired and strengthened with largely-hidden restraints. Most of the repairs would be in visually inconspicuous locations. They would only slightly alter the way the bridge moves in response to the shifting Red River soils.

Other Repairs

The shear studs added to the upper flanges of the stringers and floor beams to increase their capacity would be hidden within the new concrete deck.

To remove corrosion and preserve the steel, the bridge's steel members would be gently sandblasted and repainted to match the original color.

To prevent the accumulation of corrosive pigeon droppings, the trusses' distinctive oval perforations would be covered with removable dark gray plastic covers in less-visible upper locations. In the more highly visible end posts, the perforations would remain uncovered and small plates would be added inside the end post members to deflect droppings and protect the steel.

Lighting, Aesthetic Enhancements, and Off-Bridge Changes

Traffic lighting changes would be minimal to preserve the historic character of the bridge and nearby historic properties. Two of four light poles on the bridge would be replaced with taller poles whose design matches the originals. All fixtures would be replaced for energy efficiency and dark-sky compliance using new fixtures that generally resemble the originals.

To reduce the impact of the aesthetic lighting that would be added to the trusses, the lighting scheme would be relatively modest and the fixtures hidden as much as possible. Two other aesthetic enhancements were designed for minimal visual impact: a modestly-sized interpretive plaque would be added to the center of the bridge, and black lettering would be stamped into the floor of the new path in three locations. Off the ends of the bridge, the new path would inconspicuously tie into local sidewalks and trail systems.

11.0 REFERENCES

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Rehabilitation Report for the

John F. Kennedy Memorial Bridge

FINAL REPORT – APPENDICES

MnDOT Bridge No. 9090/NDDOT Bridge No. 02-350.220
over the Red River in Grand Forks, ND & East Grand Forks, MN

Prepared for

MINNESOTA DEPARTMENT OF TRANSPORTATION

October 2015

SRF

GEMINI RESEARCH
Cultural Resource Consultants

SRF No. 8727

Appendix A:
Bridge Photographs

Photo A1. Truss Spans – Looking South

Photo A2. Truss Span 6 – Looking South

Photo A3. Truss Span 6 & Pier 6 – Looking Northeast

Photo A4. West Approach Spans & Truss Spans – Looking West

Photo A5. East Approach Spans – Looking West

Photo A6. East Approach Spans & Truss Spans – Looking West

Photo A7. West Abutment – Looking West

Photo A8. West Approach Spans Pier Bents– Looking East

Photo A9. West Approach Span Pier Bent– Looking East

Photo A10. Pier Bent 5 – Looking Southwest

Photo A11. Pier 6 – Looking South

Photo A12. Pier 6 – Looking Northeast

Photo A13. Safety Walk, Railing, & Drain

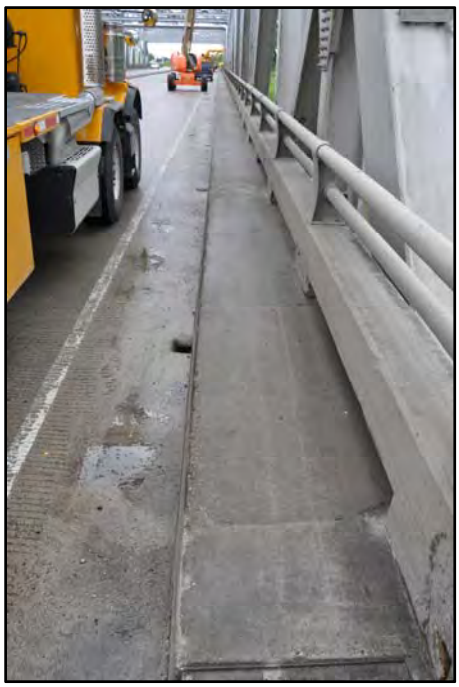

Photo A14. West Abutment Concrete Endpost & Bridge Name Plate – Looking South

Photo A15. Approach Span Light Mounting

Photo A16. Truss Endpost – Looking West

Photo A17. Truss Portal – Looking North

Photo A18. Inside Truss Span 7 – Looking West

Photo A19. Inside Truss Span 7 – Looking East

Photo A20. Typical Truss Sway Bracing

Photo A21. Typical Truss Sway Bracing

Photo A22. Truss Span Traffic Light & Sway Bracing

Photo A23. West Approach Span Traffic Light – Looking Northwest

Photo A24. Greenway Trail & West Approach Spans – Looking South

Appendix B:
MnDOT 2015 Structural Inventory and Bridge Inspection Reports

Mn/DOT Structure Inventory Report

Bridge ID: 9090 **US 2 over RED RIVER**

Date: 09/15/2015

+ GENERAL +

Agency Br. No.	
District	2
Maint. Area	2B
County	60 - POLK
City	EAST GRAND FORKS
Township	
Desc. Loc.	AT N DAKOTA STATE LINE
Sect., Twp., Range	02 - 151NN - 50W
Latitude	47d 55m 59.63s
Longitude	97d 02m 14.42s
Custodian	STATE HWY
Owner	STATE HWY
Inspection By	DISTRICT 2
BMU Agreement	
Year Built	1963
Year Fed Rehab	
Year Remodeled	
Temp	
Plan Avail.	CENTRAL

+ STRUCTURE +

Service On	HWY;PED
Service Under	STREAM
Main Span Type	STEEL HIGH TRUSS
Main Span Detail	PARKER
Appr. Span Type	STEEL BM SPAN
Appr. Span Detail	
Skew	
Culvert Type	
Barrel Length	

Number of Spans

MAIN: 2 APPR: 11 TOTAL: 13

Main Span Length	279.0 ft
Structure Length	1,261.0 ft
Deck Width	65.0 ft
Deck Material	C-I-P CONCRETE
Wear Surf Type	LOW SLUMP CONC
Wear Surf Install Year	1984
Wear Course/Fill Depth	0.17 ft
Deck Membrane	NONE
Deck Protect.	NONE
Deck Install Year	
Structure Area	81,965 sq ft
Roadway Area	70,611 sq ft
Sidewalk Width - L/R	2.5 ft 2.5 ft
Curb Height - L/R	0.75 ft 0.75 ft
Rail Codes - L/R	19 19

+ ROADWAY +

Bridge Match ID (TIS)	1
Roadway O/U Key	1-ON
Route Sys/Nbr	USTH 2

Roadway Name or Description

US 2

Roadway Function	MAINLINE
Roadway Type	2 WAY TRAF
Control Section (TH Only)	6018
Ref. Point (TH Only)	000+00.000
Date Opened to Traffic	01-01-1963
Detour Length	4 mi.
Lanes	4 Lanes ON Bridge
ADT (YEAR)	20,740 (2008)
HCADT	1,867
Functional Class.	URB/OTH PR ART

+ RDWY DIMENSIONS +

If Divided	**NB-EB**	**SB-WB**
Roadway Width	28.0 ft	28.0 ft
Vertical Clearance	19.8 ft	19.8 ft
Max. Vert. Clear.	19.8 ft	19.8 ft
Horizontal Clear.	27.9 ft	27.9 ft
Lateral Clr. - Lt/Rt		
Appr. Surface Width	60.0 ft	
Roadway Width	56.0 ft	
Median Width	4.0 ft	

+ MISC. BRIDGE DATA +

Structure Flared	NO
Parallel Structure	NONE
Field Conn. ID	BOLTED
Cantilever ID	PIN & HANGER

Foundations

Abut.	CONC - FTG PILE
Pier	STEEL - FTG PILE
Historic Status	NOT ELIGIBLE
On - Off System	ON

+ PAINT +

Year Painted	1996	**Pct. Unsound**	7 %
Painted Area	183,102 sf		
Primer Type	OTHER		
Finish Type	URETHANE		

+ BRIDGE SIGNS +

Posted Load	NOT REQUIRED
Traffic	NOT REQUIRED
Horizontal	OBJECT MARKERS
Vertical	NOT REQUIRED

+ INSPECTION +

Deficient Status	S.D.
Sufficiency Rating	48.2
Last Inspection Date	06-04-2015
Inspection Frequency	12
Inspector Name	DISTRICT2
Structure	A-OPEN

+ NBI CONDITION RATINGS +

Deck	10 % UNSOUND	5
Superstructure		6
Substructure		4
Channel		6
Culvert		N

+ NBI APPRAISAL RATINGS +

Structure Evaluation	4
Deck Geometry	5
Underclearances	N
Waterway Adequacy	6
Approach Alignment	7

+ SAFETY FEATURES +

Bridge Railing	0-SUBSTANDARD
GR Transition	N-NOT REQUIRED
Appr. Guardrail	N-NOT REQUIRED
GR Termini	N-NOT REQUIRED

+ IN DEPTH INSP. +

Frac. Critical	Y	24 mo	06/2015
Underwater	Y	60 mo	08/2012
Pinned Asbly.	Y	48 mo	06/2011
Spec. Feat.			

+ WATERWAY +

Drainage Area	
Waterway Opening	29000 sq ft
Navigation Control	NO PRMT REQD
Pier Protection	NOT APPL
Nav. Vert./Horz. Clr.	
Nav. Vert. Lift Bridge Clear.	
MN Scour Code	L-STBL;LOW RISK
Scour Evaluation Year	1997

+ CAPACITY RATINGS +

Design Load	HS20
Operating Rating	HS 26.80
Inventory Rating	HS 16.00
Posting	
Rating Date	07-22-2008

Mn/DOT Permit Codes

A: 1 B: 1 C: 2

Mn/DOT BRIDGE INSPECTION REPORT

Inspected by: DISTRICT 2

BRIDGE 9090 US 2 over RED RIVER **INSP. DATE: 06-04-2015**

County: POLK
City: EAST GRAND FORKS
Township:
Section: 02 Township: 151NN Range: 50W
Span Type: STEEL HIGH TRUSS
NBI Deck: 5 Super: 6 Sub: 4 Chan: 6 Culv: N

Location: AT N DAKOTA STATE LINE
Route: USTH 2 Ref. Pt.: 000+00.000
Control Section: 6018 Maint. Area: 2B
Local Agency Bridge Nbr:

Length: 1,261.0 ft
Deck Width: 65.0 ft
Rdwy. Area / Pct. Unsnd: 70,611 sq ft 10 %
Paint Area/ Pct. Unsnd: 183,102 sq ft 7 %
Culvert N/A

Open, Posted, Closed: OPEN

Appraisal Ratings - Approach: 7 Waterway: 6
MN Scour Code: L-STBL;LOW RISK Def. Stat: S.D. Suff. Rate: 48.2
Required Bridge Signs - Load Posting: NOT REQUIRED Traffic: NOT REQUIRED
Horizontal: OBJECT MARKERS Vertical: NOT REQUIRED

STRUCTURE UNIT: 0

ELEM NBR	ELEMENT NAME	ENV	INSP. DATE	QUANTITY	QTY CS 1	QTY CS 2	QTY CS 3	QTY CS 4	QTY CS 5
22	LS O/L (CONC DECK)	2	06-04-2015	81,185 SF	0	81,185	0	0	0
			05-27-2014	81,185 SF	0	81,185	0	0	0

Notes: |Cracks in overlay at west finger jt. Small spall in median concrete on bottom side @ E. abut. br/wall w/rbar exposed. Major deterioration of overhangs w/rust staining on approach spans underside. Concrete spalling at the ends of many floorbeams.There appears to be a dip at the joint above the second bent from the west.**Repaired approx 12 sq. of delam concrete at west end EBL. on 8/08. DSH
Epoxied deck cracks GK 5/2012
[2013] No significant change in condition. Quantity reduced to reflect area of 6' deck slabs on each end of bridge.
[2015] No change. Cracks in deck have been epoxy sealed.|

48	LS O/L (CONC SLAB)	2	06-04-2015	780 SF	0	780	0	0	0
			05-27-2014	780 SF	0	780	0	0	0

Notes: |This element reflects the 6' approach spans between the abutments and the bearings at each end of the bridge; both in CS2 due to spall and cracks on the sides and undersides of the slabs.
[2015] No change.|

300	STRIP SEAL JOINT	2	06-04-2015	700 LF	687	0	13	N/A	N/A
			05-27-2014	700 LF	689	0	11	N/A	N/A

Notes: |First two joints from the ND end in the E.B.L. are leaking above the centering pins and are causing them to Rust and deterioration of the paint system.
[2013] 1 foot of gland is cut on each joint.
Bridge crew is to make repairs to strip seals in 2014 GK 5/2014
[2015] Strip seal at joint #4 torn for 2 LF at south gutter.|

301	POURED DECK JOINT	2	06-04-2015	1,400 LF	280	1,120	0	N/A	N/A
			05-27-2014	1,400 LF	280	1,120	0	N/A	N/A

Notes: |**Sealed jts./ pourable on 11/09. DSH
The poured joints located above the truss span floorbeams show evidence of leakage below (corrosion on the top flange of the floorbeams).FC 6/11
[2013] No significant change.
Poured jt material is showing adhesion failures and should be resealed. Bit jts in west panels"overlaid panel" need sealingGK 5/2014
[2015] No change.|

303	ASSEMBLY DECK JOINT	2	06-04-2015	1,260 LF	0	1,245	15	N/A	N/A
			05-27-2014	1,260 LF	0	1,245	15	N/A	N/A

Notes: |This element is to be used to describe the condition of the longitudinal median joint running along the full length of the bridge (installed in 1984). This is a 4" wide "cross linked ethylene vinyl acetate" joint material bonded to the concrete with epoxy. The joint has scattered areas of leakage (approximately half the length of the bridge)FC 6/11
[2013]15' in CS3 due to leakage at the transverse expansion joints.
Jt is completely removed / failed above finger jts CS3 GK 5/2014
[2015] No change.|

411	OPEN FINGER JOINT	1	06-04-2015	140 LF	140	0	0	N/A	N/A
			05-27-2014	140 LF	140	0	0	N/A	N/A

Notes: |Rubber diaphrams are deteriorating, there are leaks at 10 ft. and 30 ft. from the north in the NE gland that were repaired in 2009.This is not endangering any bearings. GK 5/18/10
[2013] No significant change.
Finger jt is open 6.75 inches in the EBL and 5.5 inches in the WBL GK 5/2014
[2015] No change.|

Mn/DOT BRIDGE INSPECTION REPORT

Inspected by: DISTRICT 2

BRIDGE 9090 US 2 over RED RIVER INSP. DATE: 06-04-2015

STRUCTURE UNIT: 0

ELEM NBR	ELEMENT NAME	ENV	INSP. DATE	QUANTITY	QTY CS 1	QTY CS 2	QTY CS 3	QTY CS 4	QTY CS 5
321	CONC APPROACH SLAB	2	06-04-2015	2 EA	2	0	0	0	N/A
			05-27-2014	2 EA	1	1	0	0	N/A

Notes: |W.B.L. has been overlaid with bituminous.
[2013] The west approach slab has a 28" x 2" spall forming at the centerline joint.
Spall in west panel has been filled with bit and should be repaired GK 5/2014
[2015] Spall has been repaired with concrete.|

333	RAILING - OTHER	2	06-04-2015	8,274 LF	0	8,268	6	N/A	N/A
			05-27-2014	8,274 LF	0	8,268	6	N/A	N/A

Notes: | Minor spalls in rails. Minor spall in South rail @ West end w/rbar exposed. Two rail anchor castings on the same post,.NE end have been hit & are broken. Several spalls on the bottom of the railposts.
8 ft. cracked @ the south rail, mid span, west truss.GK 5/18/10
The concrete rail bases have scattered areas of cracking, delamination, and spalling (mainly on the approach spans). The north rail in the west truss span (L3'-L4') has impact damage - the upper rail casting is bent and fractured.FC 6/11
[2013] Horizontal cracks with staining in the majority of the concrete rail sections. Scattered spalls in the posts and lower portion of the horizontal railing. North side rail second section from the east end, the west end post is missing. North side rail 11th section from the west end, the horizontal metal rail is pushed to the north between the 2nd and 3rd posts. The surface treatment on the metal rail sections is primarily intact. All rail sections on the bridge were intact except on the north rail on the east approach, where 1 rail post is damaged and not intact.
No significant change from 2013, concrete rail has numerous areas of moderate deterioration, cracking spalling and delams, concrete is showing its age. CS2 GK 5/2014
[2015] Concrete rail has delamination at several locations. Tubular rail at Span 6 (between L2'S and L3'S) and 6 metal rail posts at various locations have been damaged by vehicle impact.|

107	PAINTED STEEL GIRDER	2	06-04-2015	5,600 LF	3,260	2,000	280	60	0
			05-27-2014	5,600 LF	3,320	2,000	280	0	0

Notes: |Rust Beginning to Form. Several of the centerline rotational pins appear to be frozen due to corrosion. There is a bend in the bottom flange of the south fascia beam, span 12, no cracks present.
2003 FC Inspection:Several of the rotational pin assemblies appear to be frozen due to corrosion. Beams 1 and 2 (from the south) in span 2 have impact damage due to a high load hit. Magnetic Particle examination of those areas revealed no cracks.
2007 FC Inspection:The approach span beams are in relatively good condition, with the exception of some advanced corrosion of the bottom flange on some of the fascia beams.
See Notes and Pictures on file in the Engineers office. Some paink chalking and corrosion present. Joe F 6/25/09
Facia bottom flange continues to rust in approach spans at various locations. GK 5/2012
[2013] No significant change.
[2015] Moderate corrosion on fascia girders in west approach spans.|

113	PAINT STEEL STRINGER	2	06-04-2015	4,464 LF	2,278	2,166	20	0	0
			05-27-2014	4,464 LF	2,298	2,166	0	0	0

Notes: |Rust Beginning to Form.
2003 FC Inspection:There is a square cope detail at the top of the web on all of the stringers, which is fatigue prone and should be monitored for cracking. There is scattered rust beginning to form on the stringers, but no areas of significant section loss.
2007 FC Inspection:There are isolated areas of paint failure and surface rust, but no pitting or section loss. The square cope detail shouldl still be monitored for cracks during in-depth inspections.
See Pictures and Notes on file in the engineers office.
2009 FC report, recommended changing the lin. ft. from 97 to 4464 , and quant 1 & 2 respectivly. GK 4/10
[2013] No significant change.
All copes looked at closely in 2014, no significant change GK 5/2014
[2015] Corrosion occurring at ends of stringers at connections to floorbeams.|

Mn/DOT BRIDGE INSPECTION REPORT

Inspected by: DISTRICT 2

BRIDGE 9090 US 2 over RED RIVER INSP. DATE: 06-04-2015

STRUCTURE UNIT: 0

ELEM NBR	ELEMENT NAME	ENV	INSP. DATE	QUANTITY	QTY CS 1	QTY CS 2	QTY CS 3	QTY CS 4	QTY CS 5
121	P/STL THRU TRUSS/BOT	2	06-04-2015	1,116 LF	0	780	220	116	0
			05-27-2014	1,116 LF	0	780	220	116	0

Notes: |Pack rust is forming at the gusset plates and batten plates on the bottom of both chords. The bolting plate @ the bottom of the 4th vert chord in the NW cor of the truss is twisted down approx 1in. where cross brace ties in. The cross brace coming into this plate is also twisted. 3 cross bracing hanger rods are bent. The X-bracing on the SW cor of truss is also bent. Bott chord boxes are infested with pigeons. Several interior welds of the lower chord box members were inspected after blasting & cleaning-no defects were noted. There are some nuts missing on diag bracing hanger rods. Two broken hanger rods 1 at MN (east) Pier, 1 at ND (west) pier.
2003 FC Inspection:There are scattered areas of surface pitting on the chords. There is minor section loss with moderate pitting at the bottom panel point connections on gusset plates and truss members (<5 %) Pack rust is forming at the gusset plates and batten plates on teh bottom of the chords.
There is some rust and some minor section loss beginning to form, elpecially along the curb line and at the bottom chord connections on the verticals and diagonals. The total area affected is less than 5%.
2007 FC Inspection:There are scattered areas of surface pitting on the chords. Magnetic particle was performed on the gusset plate to lower chord welds. A crack approx. 2" long was found on the at the L0 south side of the west truss. Consulted w/CO & was instructed to grind out crack and prime & apply Dow 888 to prevent rusting. Re-inspected in December 07 & again in March of 08. DSH.***Ground out crack was re-inspected on 11/19/07 by DSH and found to have no further propagation. No propagation on may 13th 2008.
See Pictures and Notes on the FC reports in engineers office.
2009 FC inspection: Pack rust, corrosion blisters, flaking rust and minor section loss on bottom chords and panel points. Joe F 6/25/2009
East truss, 3rd bay, no. side 1 wind bracing anchor rod snapped off

126	P/STL THRU TRUSS/TOP	2	06-04-2015	1,116 LF	540	516	60	0	0
			05-27-2014	1,116 LF	540	516	60	0	0

Notes: |Rust Beginning to Form. Minimal sect loss on top chord verticals & diagonals. Pigeons nesting in upper chord box members. There are broken welds on the angle stiffeners on top of the inside gusset plates (deck height) @ the 1st panel points W of the NE & SE end posts.**Bridge Maint. added angle iron stiffeners to all (8) Lo locations to the un-supported lengths, also the in-place stiffeners were repaired / installed where needed on 5/20/2010, (4) bolts that were missing on the East truss -no. side @ Lo location were replaced. DSH.** Bridge Maint. incorporated debris drains @ all (8) Lo locations in order to adequately flush areas that were not able to clean; this was done on 6/10/2010. DSH
2003 FC Inspection:The top chords are in good condition with only minor scattered areas of isolated paint loss and surface rust (<5 %). There is some rust and some minor section loss beginning to form, especially along the curb line and at the bottom chord connections on the verticals and diagonals. The total area affected is less than 5%.
2007 FC Inspection:No Significant shange from previous inspection. Verticals and Diagonals No significant changes.
2009 FC inspection: Corrosion blisters on top chords and panel points Joe F 6/25/2009.
Br. crew added /repaired stiffeners to some bottom chord gusset platesGK 5/18/10.
North side of west truss, vert. members are within 1/4 inch of conc. sidewalk slab, south side shows 1-2 inches of clearance.GK 5/18/10.
[2013] No significant change.
Pigeons continue to nest causing corrosion. GK 5/2014
[2015] Rust blisters, pitting and corrosion on most vertical and diagonal members, especially in areas facing the rail and deck edge. Pigeon nests and debris, mostly in upper members, causing deterioration and blocking some areas from inspection.|

152	PAINT STL FLOORBEAM	2	06-04-2015	1,380 LF	70	550	620	140	0
			05-27-2014	1,380 LF	0	620	620	140	0

Notes: |Rust Beginning to Form. Top Flanges of the floor beams are rusting.
2003 FC Inspection:The floor beams are in generally good condition, with some scattered surface rust. Some of the floor beams are starting to develop pack rust at the chord connections.
2007 FC Inspection:There is minor scattered surface rust. Rust is forming on the top flanges where the deck is leaking. Pack rust continues to develop at the horizontal bracing gussets at the end of the floor beams but section loss is minimal. See Pictures and notes on file in the engineers office.
2009 FC inspection: Top flange corrosion on all floorbeams at ends and center joint; isolated section loss at lower flange connections to panel points Joe F 6/25/2009
Each Floorbeam has 17 stiffeners.Typical of these FB's is rust w/ minor sect. loss at the bottom of the 2 exterior stiffeners. East truss 4 th FB from Pier 7 "center pier" has an area of corrosion with sect loss 1/8 + inches on bottom flange 4 ft. from south lower truss chord.
[2013] Moved 620' into CS3 due to corrosion on the top flanges due to deck leakage.
[2014] No significant change from 2013,few pics of corrosion and locations added. GK 5/2014
[2015] Most floorbeams have moderate to heavy top flange corrosion at the ends and under the center joint. Scattered areas of pitting and corrosion on webs and lower flanges.|

Mn/DOT BRIDGE INSPECTION REPORT

Inspected by: DISTRICT 2

BRIDGE 9090 US 2 over RED RIVER INSP. DATE: 06-04-2015

STRUCTURE UNIT: 0

ELEM NBR	ELEMENT NAME	ENV	INSP. DATE	QUANTITY	QTY CS 1	QTY CS 2	QTY CS 3	QTY CS 4	QTY CS 5
422	PAINTED BEAM ENDS	1	06-04-2015	14 EA	8	2	0	4	0
			05-27-2014	14 EA	10	2	0	2	0

Notes: |There is some rust forming. 2009 FC inspection: Quantity changed to correspond to number of strip-seal and finger expansion joints.
Increased the element quantity by 2 to include the “splash zone” on the truss spans (one for north side and one for the south side). The truss splash zones should be rated as condition 4. FC 6/11
[2013] No significant change.
[2015] TBD.|

161	PIN & HANGER-PAINTED	2	06-04-2015	56 EA	0	55	1	0	0
			05-27-2014	56 EA	0	56	0	0	0

Notes: |Rust Beginning to Form. Crack in Tack Weld on Nut to Hanger bottom pin 4th beam from north, 4th bent from the east.
2003 ultrasonic with no indications.
2003 FC Inspection:Ultrasonic straight beam examination was performed on all of the pins, utilizing a Panametrics Epoch III protable flaw detector and a 1/2" diameter 5 Mhz normal beam transducer. The pins were checked from both ends. A signal was noted on most of the pins, emanation from the shoulder area. This signal location is not from a stressed area of the pin, and is probably caused by a machined chamfer in the shoulder (see figure 1) No crack indications were noted on any of the pins.
2007 FC Inspection:No change from previous inspection.
See Notes and Pictures on file in the Engineer's office.
2009 FC inspection: Several cracked tack welds between nut and hanger channel noted Full UT inspection of pin/hanger assemblies will be done in 2011 on 4 year cycle. Joe F 6/25/2009
Span 4, 2 cotter keys in pins are 1/2 broke off, but enough remaining that nut can not come loose.GK 5/18/10
Ultrasonic examination was performed on the hanger pins during the 2011 FC inspection (no significant findings). The pin & hanger assemblies all have minor surface corrosion (condition state 2) - they are all functioning as intended.FC 6/11.
[2013] No significant change. NDT not required until next FC inspection.
[2014] No change, cracks at tack welds not propagating GK 5/2014
[2015] Ultrasonic examination of all hanger pins from both ends revealed no defects. The P&H joint in Span 3, Girder 7, has moderate to heavy corrosion. Grounding cables across joint at Beam 1 in Spans 11 & 12 are missing.|

373	STEEL HINGE	2	06-04-2015	32 EA	0	32	0	0	0
			05-27-2014	32 EA	0	32	0	0	0

Notes: |This element should be used to describe the condition of the hinge bearings supporting the suspended spans (in approach spans #5 & 8). There are swivel hinges at the truss ends (Piers #6 & 8), with expansion hinges at the other end of Spans #5 & 8 (4 joints, total of 32 assemblies). The hinge assemblies all have minor surface corrosion (condition state 2) - they are all functioning as intended.FC 6/11
Close inspection of swivel jts in 2012 show no signs of problems. GK / RH 5/2012
[2013, 15] No significant change.|

423	GUSSET PLATE (PAINT)	1	06-04-2015	80 EA	0	36	44	0	0
			05-27-2014	80 EA	0	36	44	0	0

Notes: |** A (3) stage spot painting of gussets was completed in the week of 9/14/09. DSH
The bottom chord gusset plates have old pitting (painted over), with some active corrosion – all should be rated as condition state 3. Some of the upper chord gusset plates staining, surface corrosion, and isolated pitting (painted over).FC 6/11
[2013] Quantity of 36 in CS2 due to minor corrosion starting to form on the interior surfaces of the gusset plates.
[2015] No significant change.|

380	SECONDARY ELEMENTS	1	06-04-2015	1 EA	0	1	0	0	N/A
			05-27-2014	1 EA	0	1	0	0	N/A

Notes: |2009 FC inspection: New element. Bottom gusset plates showing some corrosion and pack rust. Joe F 6/25/2009
[2013] There is corrosion on the lower lateral braces and 2 hanger rods are broken.
Same in 2014 GK 5/2014
[2015] No change.|

Mn/DOT BRIDGE INSPECTION REPORT

Inspected by: DISTRICT 2

BRIDGE 9090 US 2 over RED RIVER INSP. DATE: 06-04-2015

STRUCTURE UNIT: 0

ELEM NBR	ELEMENT NAME	ENV	INSP. DATE	QUANTITY	QTY CS 1	QTY CS 2	QTY CS 3	QTY CS 4	QTY CS 5
311	EXPANSION BEARING	2	06-04-2015	6 EA	0	4	2	N/A	N/A
			05-27-2014	6 EA	0	4	2	N/A	N/A

Notes: |SW rocker is tipped all of the way toward the west. East rockers tipped slightly to the E. Base plates 3rd Beam from the N.@ E. expansion jt. & 3rd Beam from S. on N side @ W. expansion all are fractured. West Pier rockers tipped away from river @ 40 degrees Base plate on No. one beam @ W. finger jt. also fractured on inside face. Bearings and pins need cleaning and greasing. Bearing holder plate is broken loose, crack between beam and Bearing Holder east finger Jt.
2003 FC Inspection:The Rocker bearing at the SouthWest corner of the west truss span is fully expanded. The remainder of the bearings appear to be functioning properly.
2007 FC Inspection:The Southwest bearing is now only slightly in expansion. No other changes were noted.
See Notes and Pictures on file in the Engineers office.
I went to 9090 on 03/24/09 at 40 degrees to observe the bearing configurations, below are the results.
Both West Rockers have been moved from the original Positions;
The southwest Rocker has been moved twice on the base, to the west 5 3/4" for a total of 11.5 inches. and moved once on top attachment to the lower chord, to the east 7".
There is room for additional top movement four times for a total of 28" and no room for the base.
The bearing is currently out of plumb 4" to the west.
The northwest Rocker has been moved once on the base, to the west 5 3/4" and never moved on the top attachment to the lower chord.
There is room for additional top movement five times for a total of 35" and room for one move on the base for 5 3/4".
At Pier 6, SW rocker, The bearing is currently out of plumb 5" to the west.
The top movement in the South West top was done March 8th 2004, one of the bottom movements appear to have been done in 1999, it is not known when the other bottom movement was done.
The bolt pattern on the Lower chord is 7" and the pattern on the bearing is 21".
RN & MG 03/24/2009
SW rocker tilt/angle is the same as 2009 inspe

313	FIXED BEARING	2	06-04-2015	18 EA	10	8	0	N/A	N/A
			05-27-2014	18 EA	10	8	0	N/A	N/A

Notes: |Abutment Bearings have been blasted and painted. 2009 FC inspection: Fixed bearings at west abutment are tipped toward tiver, with unsound concrete below them.
Bottom of bearings are corroding & rusting GK 5/18/10
Continuing to corrode. RH 5/2012.
[2013] The fixed bearings at the West Abutment have uplift issues as the concrete around the bearings has now spalled off. Bases continue to corrode w/ scattered rust on bearings. GK 5/2014
[2015] No change.|

202	PAINT STL COLUMN	2	06-04-2015	72 EA	22	18	32	0	0
			05-27-2014	72 EA	22	18	32	0	0

Notes: |2 cotter keys missing bent 4 4 @ bent 5 &2 missing at bent 11GK 5/18/10.
There appears to be a dip at the joint above the second bent from the west. is this bent settled? Needs an evaluation.
[2013] No significant change. All bents are tilted from 2-5 degrees.
[2014] Mn. bents 13 - 9 all tipping west from 1 - 3.75 inches, bent 9 3.75 inches, and ND bents 5-2 all tipping east 2-4 inches bent 5 4 inches.CS3 refers to bents slightly out of alignment over approx. 3 inches GK 5/2014
[2015] Tipping on all bents measured; largest was 4.5 degrees to west on Bent #13, south half. All bottom bearings have corrosion and bent plates due to pack rust. Bottom of column 1 on Bent #12 has moderate corrosion on lower 2 LF.|

210	CONCRETE PIER WALL	2	06-04-2015	210 LF	140	0	70	0	N/A
			05-27-2014	210 LF	140	0	70	0	N/A

Notes: |The ND Pier 6 is moving towards the river and is twisting causing cracks in Pier wall, there are cracks in the east Pier but not as severe as Pier 6. There is 6 feet of debris and soft silt at the bottom of the center pier - 2004 underwater inspection.
2009 FC inspection: Evidence of movement and twisting in west main pier (pier 6) Joe F 6/25/2009
A baseline was established using 2 eyebolts,one on no. side one on so. side of west end of pier 6 cap, a stringline streched between the 2 eyebolts show a bow of 5.5 inches in the pier wall.GK 5/18/10
Cracks continue in 2012
Bow in wall 5 5/8 inches in 2012 GK
[2013] No significant change. Cracks in wall are more pronounced at Pier 6. See notes for element 234.
No significant change, CS3 refers to P6 with extensive cracking and 5.5 inch bow. GK 5/2014
[2015] No significant change. Pier 6 curvature increasing.|

Mn/DOT BRIDGE INSPECTION REPORT

Inspected by: DISTRICT 2

BRIDGE 9090 US 2 over RED RIVER INSP. DATE: 06-04-2015

STRUCTURE UNIT: 0

ELEM NBR	ELEMENT NAME	ENV	INSP. DATE	QUANTITY	QTY CS 1	QTY CS 2	QTY CS 3	QTY CS 4	QTY CS 5
215	CONCRETE ABUTMENT	2	06-04-2015	140 LF	70	67	3	0	N/A
			05-27-2014	140 LF	105	35	0	0	N/A

Notes: |Minor Hairline cracks @ W. abut. 1 sq. ft. of spall & deteriorated concrete @ S. end of W. Br/wall. Patch is coming out of conc. Bm. Water leaking through poured joint over parapets some cracking w/leaching of E. parapet. Horz. crack in S. half of W b/wall, 6 inches from the top. Small spall in top of E. br./wall w/rebar expsed.GK 5/18/10
Abut. 14, north 1/2 added an additional 8 inches to face of backwall, some wood forms still inplace. GK 5/18/10
Loose cork from cantilevered sect. working its way out. RH 5/2012
[2013] There are spalls behind the fixed bearings on the West Abutment due to likely uplift from the structure. This is reflected in the current rating.
Cracking with spalls delamming around fixed bearings in west abut continues. GK 5/2014
[2015] West abutment parapet wall cracked throughout, with heavy cracking at south end.|

220	CONCRETE FOOTING	2	06-04-2015	18 EA	14	2	2	0	N/A
			05-27-2014	18 EA	18	0	0	0	N/A

Notes: |This element should be used to describe the condition of the footing caps supporting the steel column bents. This is an each item - there are 9 bents, with a separate footing for each side (eastbound & westbound), so the total quantity should be 18 (each footing support 4 steel columns). Each footing cap is 30 ft. long, and is supported by two lower footings (each lower footing is supported by 2 steel H-piling). The lower footings should be below grade (not visible for inspection) - only the upper portions of the footing caps should be visible for inspection.FC 6/11
[2013] There is minor cracking on the footings.
[2015] Bents #4 and #5 have exposed footings due to ground washing away during flooding. The footing on Bent #5 are undermined, with piling exposed for a maximum of 18" at the south footing.|

234	CONCRETE CAP	2	06-04-2015	210 LF	140	0	70	0	N/A
			05-27-2014	210 LF	140	0	70	0	N/A

Notes: |Cracks in the west pier are at 1/16 inch wide.
As the bowing and cracking in Pier #6 extends through the “cap”, the ratings for this element should be the same as for element #210 (pier wall).FC 6/11
[2013] No significant change. The cracks on Pier 6 range from hairline to 1/4" and extend through the pier wall and cap. The string line attached to the pier wall measures 1-3/8" on the north end and 6" at the center, indicating a 4-5/8" bow.
[2014] No significant changes, bow in wall measures the same as 2013 GK 5/2014
[2015] Bow in Pier 6 increasing.|

387	CONCRETE WINGWALL	2	06-04-2015	4 EA	3	1	0	0	N/A
			05-27-2014	4 EA	3	1	0	0	N/A

Notes: |2009 FC inspection: 3 ft x 4 ft washout and separation from slope protection at NE wingwall. Joe F 6/25/2009
[2013, 15] No significant change.|

357	PACK RUST	2	06-04-2015	1 EA	0	1	0	0	N/A
			05-27-2014	1 EA	0	1	0	0	N/A

Notes: |Pack rust at chord splices and horizontal gussets connecting diagonal bracing on lower chord.
[2013] No significant change.
[2015] Pack rust distorting corners of lower chord web splice plates up to 3/8"|

358	CONC DECK CRACKING	2	06-04-2015	1 EA	1	0	0	0	N/A
			05-27-2014	1 EA	0	0	1	0	N/A

Notes: |Deck is cracked with leaching, and has been epoxied in 08. DSH
[2013] There is extensive unsealed cracking throughout the deck surface.
Chain dragged deck and found numerous delammed areas . GK 5/2014
[2015] Deck cracks have been epoxy sealed. Very minor unsealed cracking remains.|

Mn/DOT BRIDGE INSPECTION REPORT

Inspected by: DISTRICT 2

BRIDGE 9090 US 2 over RED RIVER INSP. DATE: 06-04-2015

STRUCTURE UNIT: 0

ELEM NBR	ELEMENT NAME	ENV	INSP. DATE	QUANTITY	QTY CS 1	QTY CS 2	QTY CS 3	QTY CS 4	QTY CS 5
359	CONC DECK UNDERSIDE	2	06-04-2015	1 EA	0	0	0	1	0
			05-27-2014	1 EA	0	0	0	1	0

Notes: |Spalling aat numerous places in the approach spans @ CenterLine @ 2nd Floor Beam W. of E. pier. Concrete spalling out over Floor Beams @ the 3rd and 4th from W. pier with rebar exposed. Large spalls in deck soffet, north side full length..
2009 FC inspection: spalling on deck overhang areas are less than 10% of total deck area. Joe F 6/25/2009
General appearance under deck is in very good condition other than north soffit.GK 5/18/10
As the spalling on the underside of the deck is generally confined to the fascia overhangs, it constitutes less than 10% of the total deck area, A rating of condition 3 would be appropriate FC 6/11
Bottom of deck in generally fair condition, poor condition on catilevered sections. RH 5/2012
[2013] Underside of deck has numerous areas of saturation, especially in the east truss span. This warranted an NBI of 5. Delamms and rust stains continue under soffit, over untraveled portion of bridge. Bridge crew removed delamms over traveled areas, and plan on removing the rest with snooper truck. GK 5/2014
[2015] Extensive spalling and exposed rebar on deck overhangs. Loose concrete is piled below overhang near both abutments. Isolated areas of map cracking present in other areas of the deck.|

360	SETTLEMENT	2	06-04-2015	1 EA	0	0	1	N/A	N/A
			05-27-2014	1 EA	0	0	1	N/A	N/A

Notes: |Appears south end of west main pier " P6 " has moved toward the river. Movement and twisting and continues to get worse.
There appears to be a dip at the joint above the second bent from the west.
While no change in the eastward movement of Pier #6 was observed during the 2011 inspection, the cumulative long-term movement of Pier #6 (25” to the east at the south end), probably warrants a rating of condition 3 for this smart flag.
[2013] No significant change.
[2014] Survey refectors mounted on P6 and being monitored by survey crew, comps on file GK 5/2014
[2015] Movement of Pier 6 is continuing.|

361	SCOUR	2	06-04-2015	1 EA	0	1	0	N/A	N/A
			05-27-2014	1 EA	0	1	0	N/A	N/A

Notes: |The undermining of the footing at Bent #5 has increased since 2009 FC 6/11
Same in 2012
[2013] No significant change.
[2015] Bent #5 footings undermined, with H piling under south footing exposed for 18 inches.|

362	TRAFFIC IMPACT	1	06-04-2015	1 EA	0	1	0	N/A	N/A
			05-27-2014	1 EA	0	1	0	N/A	N/A

Notes: |Bent Flange from impact on south fascia and 1st interior beams, east approach over road.
[2013, 15] No significant change.|

363	SECTION LOSS	2	06-04-2015	1 EA	0	1	0	0	N/A
			05-27-2014	1 EA	0	1	0	0	N/A

Notes: |. ***The through trusses were blasted & painted in 1996. The % rated down is to denote existing sect loss. This problem will be evaluated at the next snooper inspection.***
[2013] Areas of isolated section loss on the floorbeams and lower chords, but no significant loss of cross section.
Same in 2014 GK 5/2014
[2015] No change.|

964	CRITICAL FINDING	2	06-04-2015	1 EA	1	0	N/A	N/A	N/A
			05-27-2014	1 EA	1	0	N/A	N/A	N/A

Notes: |[2015] No critical findings were noted during this inspection.|

966	FRACTURE CRITICAL	2	06-04-2015	1 EA	1	0	0	N/A	N/A
			05-27-2014	1 EA	1	0	0	N/A	N/A

Notes: |Do Not Remove. See in-depth report for location of F/C members.
[2013, 15] No change.|

981	SIGNING	2	06-04-2015	1 EA	0	0	1	0	0
			05-27-2014	1 EA	0	0	1	0	0

Notes: | < none >
[2013] NE delineator is bent over to the west.
[2015] No change.|

Mn/DOT BRIDGE INSPECTION REPORT

Inspected by: DISTRICT 2

BRIDGE 9090 US 2 over RED RIVER INSP. DATE: 06-04-2015

STRUCTURE UNIT: 0

ELEM NBR	ELEMENT NAME	ENV	INSP. DATE	QUANTITY	QTY CS 1	QTY CS 2	QTY CS 3	QTY CS 4	QTY CS 5
984	DRAINAGE	2	06-04-2015	1 EA	0	1	0	N/A	N/A
			05-27-2014	1 EA	0	1	0	N/A	N/A

Notes: |Downspouts missing on NE corner of the main span. Slope washouts @ E. River bank.
Downspouts were extended from the deck drains by slipping square tubing over the existing and fastening with 2 screws. Rust and corrosion is present at this connection and where fastened to the lower chord with a welded strap. The straps are rusting badly and a few are broken, completely thru, should be repaired. GK 5/2012
[2013] No significant change.
Bridge crew repaired broken straps on deck drains GK 5/2014
[2015] Deck drains are open and functioning properly.|

985	SLOPES	2	06-04-2015	1 EA	0	1	0	N/A	N/A
			05-27-2014	1 EA	0	1	0	N/A	N/A

Notes: |Major transverse cracking in slope paving @ West end w/some heaving. Erosion at the 4th bent from the west. The Dike on the MN side has been removed. There are Gophers or something tunnelling in and around the abutment on the ND side. Abut. 14, washout undermining NE br. seat. GK 5/18/10
ND built a rock flume type ditch alongside Bent 5 / pier 6 south side . GK 5/2012
[2013, 15] No significant change.|

986	CURB & SIDEWALK	2	06-04-2015	1 EA	0	1	0	N/A	N/A
			05-27-2014	1 EA	0	1	0	N/A	N/A

Notes: |cracks are present, with a 4 ft. area spalled out w/ rebar @ the west end of C&G.GK 5/18/10
[2013] No significant change.
Spalls and delamms continue to get worse 4 ft area is now 10 ft with numerous locations spalling in the curb faces.Repairs may be needed as rebar is jutting out. GK 5/2014
[2015] No significant change.|

988	MISCELLANEOUS	2	06-04-2015	1 EA	0	1	0	N/A	N/A
			05-27-2014	1 EA	0	1	0	N/A	N/A

Notes: |Conduit seperated from light pole, E. approach on the North Rail
2009 FC inspection: Crack to to lack of fusion in a plug weld in outer gusset plate L-12, east truss, south side was discovered. Per consultation with Todd Nieman, crack was ground out. Crack penetrated full thickness of gusset plate, but did not extend into lower chord. Crack was caulked and painted after grinding.
Out of plumb measurements for the Bents are listed below: Measurement show the distance between a plumb line centered on the pin and the center of the lower bearing plate, in inches.

Bent	North	South
13	4.0	2.250
12	2.375	4.250
11	1.125	2.625
10	2.125	2.50
9	1.750	2.50
5	4.750	3.750 - 4.2 in 2012
4	1.250	1.250
3	3.750	2.125
2	3.375	3.750

In sept 2010, meas. were taken from Bent 5 to Pier 6.Bent 5 has an eyebolt protruding from the south concrete base near centerline, approx 5 ft. above ground and meas. were taken from the center of this eyebolt to the center of Pier 6,"Paintmark with an X" and to two eybolts on Pier 6 both approx 1 ft. above ground and approx. 4 ft. from the outside edge of the pier wall, both painted .
Meas. were taken with a steel chain.
So. meas. 72.15 ft.
Center 63.35 ft
North 68.95 ft.
Triang. meas. continue to remain close to orig. meas. in 2012 GK
Bents on west side /ND are tipping east,worst being bent 4 at 4.25 inches out of plumb and Mn side are tipping west worst being bent 12, 3 1/2 inches out of plumb.
Bents out of plumb meas. are on file GK 5/2012
[2013] No significant change.
[2015] Conduit box at NE Wingwall has broken cover.|

Mn/DOT BRIDGE INSPECTION REPORT

Inspected by: DISTRICT 2

BRIDGE 9090 **US 2 over RED RIVER** **INSP. DATE: 06-04-2015**

STRUCTURE UNIT: 0

ELEM NBR	ELEMENT NAME	ENV	INSP. DATE	QUANTITY	QTY CS 1	QTY CS 2	QTY CS 3	QTY CS 4	QTY CS 5
967	GUSSET DISTORTION	1	06-04-2015	1 EA	0	1	0	0	N/A
			05-27-2014	1 EA	0	1	0	0	N/A

Notes: |2009 FC inspection: new element. , Top plates have minor distortion due to construction fit-up. Bottom plates have distortion due to pack rust. Joe F6/25/2009
[2013, 15] No significant change.|

General Notes: FC June 2011 entered as a routine inspect. until FC portion in the new program SIMS is functioning as intended GK 6/11 11/08 Changed Channel NBI code from 7 to 6 per Rog H NORTH SOUTH BENT 13 2-1/2" 0" in 2010, 2" BENT 12 2" 1-1/2" BENT 11 0" 1-1/2" Tipping west BENT 10 3/4 2" BENT 9 1" 1-1/2" PIER 8 1-3/4 1-3/4" PIER 7 1-1/2" 2-1/4" PIER 6 2" 6-1/2" Tipping east BENT 5 4" 3-1/4" BENT 4 0" 0" BENT 3 2-3/4" 0" BENT 2 1 1/2" 3" In 2010 meas. were within 1 inch except bent 13 south GK 5/18/10 Snooper bridge with many Pigeon nests within portals. Bridge layout= west end/ abut. 1,bent 2-5,pier 6, pier 7, pier 8, bent 9-13, abut 14 @ east end.GK 5/18/10
Snooper 5/14/2012 5/2014
[2015] FC inspection performed 6/3/15 and 6/4/15 by Joe Fishbein, Pete Wilson, Bill Nelson, Farrell Potter, Ken Rand and Jennifer Zink. Heavy accumulations of pigeon nests and droppings in many areas, especially upper truss members, are preventing inspection of these areas as well as accelerating corrosion.

Inspector's Signature

Reviewer's Signature / Date

Appendix C:
MnDOT 2013 Routine and Fracture Critical Inspection Report

2013 ROUTINE AND FRACTURE CRITICAL BRIDGE INSPECTION REPORT

BRIDGE # 9090
US 2 over RED RIVER

DISTRICT: District 2 **COUNTY: Polk** **CITY/TOWNSHIP: East Grand Forks**

Date(s) of Inspection: 06/03/2013 - 06/06/2013

Equipment Used: Manlift, A-62, A-62

Owner: State Highway Agency

Inspected By: Evens, Eric; Klein, Gregory; Nelson, Bill; Potter, Farrell; Rand, Ken; Walker, Scott; Wilson, Pete; Zink, Jennifer

Report Written By: Bill Nelson

Report Reviewed By: Jennifer Zink

Final Report Date: 12/16/2013

MnDOT Bridge Office
3485 Hadley Avenue North
Oakdale, MN 55128

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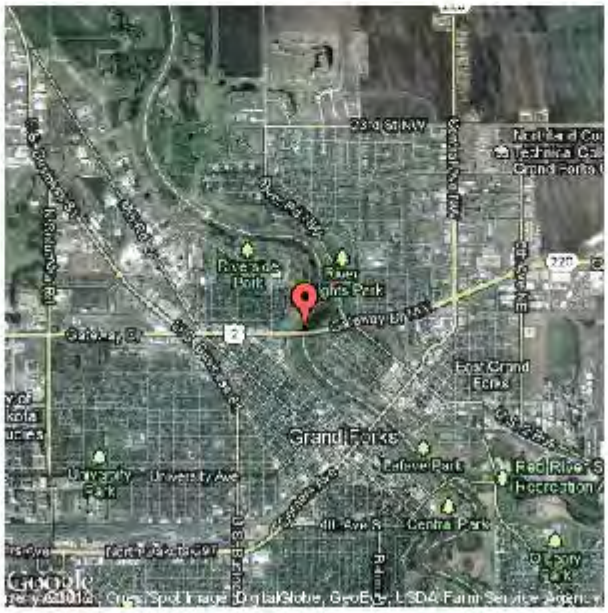
Bridge 9090 Location Map



I. Findings Summary

This report documents the findings of the fracture critical and routine inspection of Bridge 9090 located on US 2 over the Red River of the North between Grand Forks North Dakota and East Grand Folks Minnesota. The inspection took place on June 3-6, 2013.

All photos from the 2013 Fracture Critical Inspection are available for viewing in the SIMS database for this bridge.

General condition photos are referenced in the field notes and are included in Section V of this report.

Significant Findings

1. Pier 6 continues to show signs of significant movement (Photos 11-13) . The center of the pier is 4-5/8" out of alignment and there are cracks in the pier cap and pier wall up to 1/4" wide (Photos 14-20).

   Recommendation: Continue to monitor Pier 6 for movement.

2. The deck has areas of saturation with cracking and efflorescence, especially the east spans (Photo 59). Cores were taken in 4 of the saturated areas and sent to the Maplewood lab for analysis. The tests showed extremely high chloride levels (See Appendix F). The layer of concrete under the overlay was crumbling.

   A rehabilitation study for the bridge has recommended that the deck be replaced.

3. Due to a 2009 finding, magnetic particle testing of plug welds was performed at panel points L0 and L-1/2 in both trusses on both spans. No new cracks or propagation was found.

   Recommend continued testing and monitoring of these areas during future fracture critical inspections.

4. No significant loss of section was noted in primary structural members since 2009 although there are areas of active corrosion. See the Inspection Field Notes in Section IV for specific locations and photo references.

   Recommend continued monitoring for section loss.

NBI Condition Summary

While District 2 will have the final authority in determining the NBI element condition ratings, the ratings recommended by the MnDOT Bridge Office are summerized below and on the attached preliminary report (Appendix B).

NBI Condition Ratings			
Item	Current	Suggested	Comments
Deck	7 (Good Condition)	5 (Fair Condition)	2013 - Lower to 5. Scattered areas of saturation and efflorescence, especially in east bound lane of east truss span (Photos 1-5). Spalling and exposed rebar on the fascias throughout. Deck should be analyzed by the District via GPR to determine extent of deterioration. Rating lowered from 7 to 6 in 1993. Overlay has isolated concrete patches, with one section of delamination in Span #5 (eastbound). Underside of deck overhang has spalling (exposed rebar), particularly on the east approach spans.
Superstructure	6 (Satisfactory Condition)	6 (Satisfactory Condition)	2013 - No change. Scattered areas of active corrosion and old pitting (painted over) on the bottom chord, bottom chord gusset plates and truss verticals & diagonals located near the deck level. Any section loss is minor.
Substructure	5 (Fair Condition)	4 (Poor Condition)	2013 - Lower to 4 due to continued movement and cracking of Pier 6 and spalling behind the fixed bearings on the West Abutment (it appears that the bearings are being pulled out (Photos 6-14).
Channel	6 (Bank slump; minor damage)	6 (Bank slump; minor damage)	2013 - No change (recommended in the 2012 Underwater Report).

II. Inspection Logistics

The focus of this report is the presentation of the routine and fracture critical inspection findings for Bridge No. 9090 – US Highway 2 over the Red River of the North between East Grand Forks, MN (Polk County) and Grand Forks, ND (Grand Forks County). The inspection took place on June 3-6, 2013.

Bridge Description

Bridge No. 9090 consists of two truss spans and eleven approach spans. Each truss span is a 9-panel bolted steel "Parker" high truss constructed in 1963. Each truss span is 279' long, with a 4-lane roadway width of 65' (two 28' roadways, 4' raised median and two 2'-6" sidewalks). The trusses are supported by three reinforced concrete piers. There are 6 east approach spans and 5 west approach spans, all of multi-girder construction and supported on steel bents. Reinforced concrete abutments and wingwalls support the ends of the approach spans. The deck is cast-in-place concrete. The deck is supported by a floorbeam/stringer structure in the truss spans and rolled steel wide-flange beams in the approach spans.

The upper and lower truss chords are constructed of four plates welded into a box section, with oblong access holes in the bottom plates. Vertical and diagonal members are 18" deep wide-flange rolled shapes. Floorbeams are 66" deep welded plate girders, connected to the truss panel points at the webs and bottom flanges. Stringers are 30" wide flange rolled shapes, connected to the floor beams with web clip angles.

There are 40 gusset plate connections on each truss span (20 per truss). All gusset plates are 5/8" thick. The gusset plates and lower chord at panel points L0 to L-1/2 are constructed of quenched and temepered 100 KSI (T1) steel. The L0 and L 1/2 gusset plates are plug-welded to the lower chords, all other gusset plate connections are bolted.

The approach spans are constructed of eight 36" wide-flange rolled shapes, supported on bolted steel bents. Pin/hanger joints are present at several locations.

The current ADT is 20,740 (2008). The bridge is not load posted. The current rating, performed in 2008, is HS26.8 (Operating) and HS16.0 (Inventory). Ratings are governed by truss member L4'-U4 (See Appendix D). A gusset plate analysis was performed in 2008, but did not change the existing ratings.

The bridge is owned and maintained by MnDOT District 2-B and North Dakota. Please reference the Structure Inventory Report for further information on the bridge (Appendix A).

Inspection Access

Inspection of the underside of the bridge and the truss members was performed using 2 Aspen Aerials A-62 Under Bridge Inspection Units ("Snoopers") and a rented 60' "Snorkel" lift. Traffic control, consisting of a lane closure, was provided by District 2.

Inspection Procedures

Inspection Procedures

1. Fracture Critical Members: Lower chord, vertical and diagonal truss members, and floorbeams.
   a. Visually inspect the lower chord and lower chord panel point connections.
   b. Ultrasonically test the lower gusset plates at panel points which appeared to have the worst section loss.
   c. Visually inspect the vertical and diagonal members.
   d. Visually inspect the floorbeams.
   e. Visually inspect the attachment of the floorbeam to the panel point.
   f. Visually inspect the gusset plates. Check gusset plates for distortion using a 48" straight edge.

2. Fatigue Prone Details: Focus mainly on D, E, and E' categories – See details list for locations
   a. Visually inspect all welded and bolted connections for integrity.
   b. Visually inspect all fracture critical members for shop welds, bolts, and tack welds.

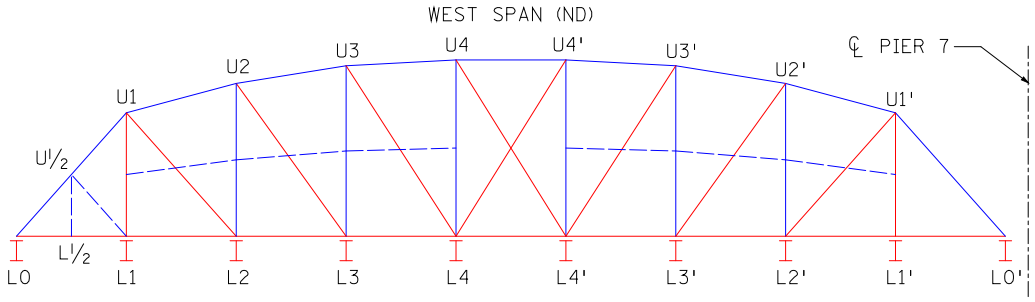
3. Other Miscellaneous Inspection Details:
   a. Visually inspect all bearings for corrosion and proper functioning.
   b. Check degree of tipping on rocker bearings.
   c. Visually inspect the underside of deck for cracking, spalling and exposed rebar.
   d. Visually inspect the substructure for exposed footing and/or piling, scaling and cracking of concrete and debris accumulation.
   e. Visually inspect the substructures for evidence of settlement or movement.
   f. Check steel bents on approach spans for out-of-plumb tipping.

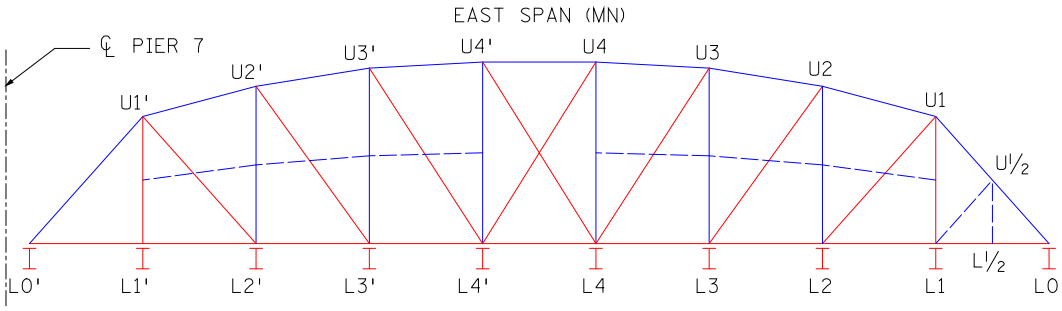
4. Additional NDT verification:
   a. Check plug welds in gusset plates at panel points L0 and L-1/2 with magnetic particle.
   b. Pins are not due for UT inspection until 2015.

III. Fracture Critical Members/Fatigue Prone Details

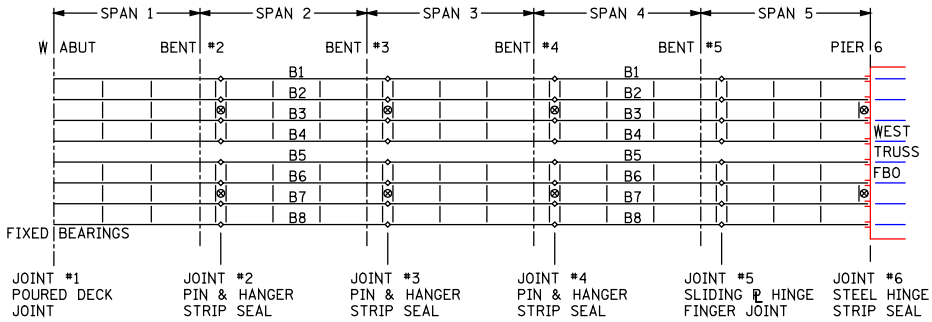
The National Bridge Inspection Standards (NBIS) defines a “Fracture Critical Member” (FCM) as “A steel member in tension, or with a tension element, whose failure would likely cause a portion of or the entire bridge to collapse.” The fracture critical members on this bridge are the truss superstructure members, floorbeams, and numbered gusset plates shown in red on the diagrams below.

This section identifies fatigue prone details present on the primary structural steel members of this bridge. Steel structural members subjected to tension or reversal stresses can develop fatigue cracks. The three primary parameters affecting fatigue crack propagation are stress range, the number of stress cycles, and the type of detail. Other factors, such as out-of-plane bending, heat straightening, or field-welded repairs can increase the likelihood of fatigue cracking. For the purpose of designing bridges for fatigue caused by in-plane bending stress, AASHTO describes weld details and connections using an alphabetical designation ranging from stress category “A” (best fatigue resistance) to stress category “F” (most susceptible to fatigue crack growth). Fatigue detail categories are defined in Table 6.6.1.2.3-1 of the 2009 AASHTO LRFD Bridge Design Specifications. See Table Below.

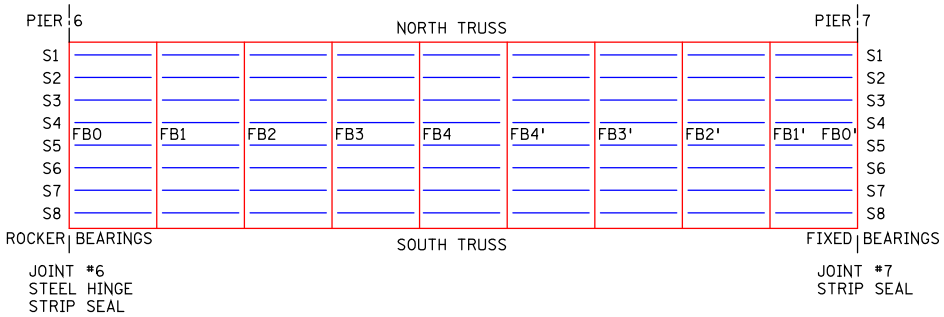

WEST SPAN (ND)
℄ PIER 7
U1
U2
U3
U4
U4'
U3'
U2'
U1'
U1/2
L1/2
L0
L1
L2
L3
L4
L4'
L3'
L2'
L1'
L0'


EAST SPAN (MN)
℄ PIER 7
U1'
U2'
U3'
U4'
U4
U3
U2
U1
U1/2
L1/2
L0'
L1'
L2'
L3'
L4'
L4
L3
L2
L1
L0

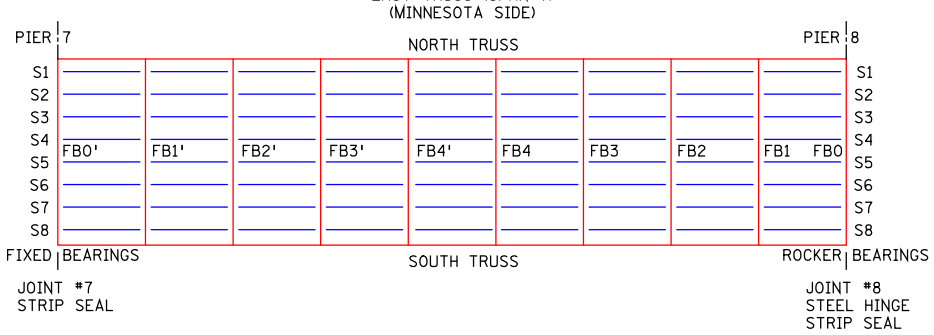
WEST APPROACH SPANS
(CURVATURE NOT SHOWN FOR CLARITY)



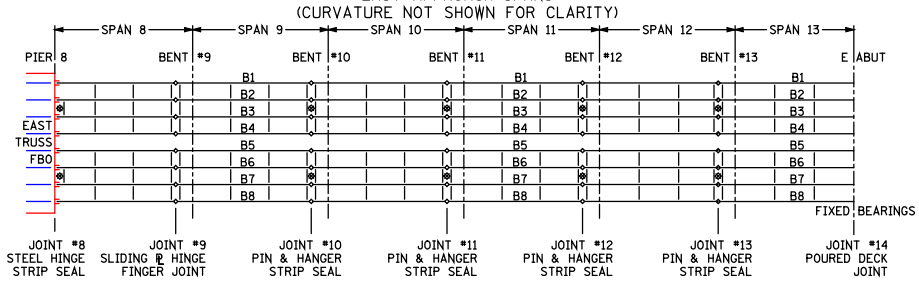
WEST TRUSS (SPAN 6)
(NORTH DAKOTA SIDE)



EAST TRUSS (SPAN 7)
(MINNESOTA SIDE)



EAST APPROACH SPANS
(CURVATURE NOT SHOWN FOR CLARITY)



Fatigue Prone Details (per Table 6.6.1.2.3-1, AASHTO, 2009)

9090 Fatigue Prone Detail Chart

Span No.	FCM No.	AASHTO Detail No.	Fatigue Detail Description (General condition/location)	AASHTO Category	Remarks (Situation)
TRUSSES					
East & West Truss Span	North and South Truss	1.1	Lower/upper chord, diagonal & vertical member tension zones and all gusset plates.	**A**	Inspect all areas for distortion, section loss, and integrity of bolted connections.
East & West Truss Span	North and South Truss	3.1	Lower chord welded box members.	**B**	Inspect all areas for cracks at areas of weld discontinuity.
East & West Truss Span	North and South Truss	2.1	Bolted connections.	**B**	At net plate sections of bolted connections.
East & West Truss Span	North and South Truss	AWS D1.1 (8.3, 8.4)	Plug welds on fracture-critical members	**E-Base Metal** **F-Shear**	Gusset Plates at L0, L-$^{1}/_{2}$ and on members near gusset plates.
East & West Truss Span	North and South Truss	N/A	Inspect tack welds on fracture-critical members.	**E' Equivalent**	Field-identified.
FLOORBEAMS					
East & West Truss Span	Floorbeams	1.1	Bottom flanges and web tension zones.	**A**	Inspect all areas for distortion, section loss, and integrity of bolted connections.
East & West Truss Span	Floorbeams	2.1	Bolted connections of all floorbeams (flanges and web) to floorbeam connection plates.	**B**	At net plate sections of bolted connections.
East & West Truss Span	Floorbeams	4.1	Transverse stiffeners welded to floorbeams.	**C'**	At ends of welds, especially intermittent welds.
East & West Truss Span	Floorbeams	N/A	Inspect tack welds on fracture-critical members.	**E' Equivalent**	Field-identified.

9090 Fatigue Prone Detail Chart (continued)

Span No.	FCM No.	AASHTO Detail No.	Fatigue Detail Description (General condition/location)	AASHTO Category	Remarks (Situation)
STRINGERS					
East & West Truss Span	Stringers	1.1	Bottom flanges and web tension zones.	**A**	Inspect all areas for distortion, section loss, and integrity of bolted connections.
East & West Truss Span	Stringers	2.1	Bolted connections of all stringers to floorbeams.	**B**	At net plate sections of connections.
East & West Truss Span	Stringers	1.3	Top flange copes where stringers frame into floorbeams.	**C**	At both ends of each stringer.
APPROACH GIRDERS					
East & West Approach	Girders	1.1	Bottom flanges and web tension zones.	**A**	Inspect all areas for distortion, section loss, and integrity of bolted connections.
East & West Approach	Girders	4.1	Transverse stiffeners welded to girders at hinge joints.	**C'**	At ends of welds, especially intermittent welds.
East & West Approach	Girders	2.4	Pin & Hanger joints.	**E**	Channel sections used as pin plates.
East & West Approach	Girders	N/A	Inspect tack welds on fracture-critical members and on Pin/Hanger joints.	**E' Equivalent**	Field-identified. Cracked tack welds were noted between Pin & Hanger nuts and channels.

Bridge #9090 Inspection Field Notes

West Approach Spans #1-5 (North Dakota Side)			
Location	**Detail**	**South (Eastbound) Side**	**North (Westbound) Side**
West Abutment	Abutment	[2013] 30" long x 16" wide at top of parapet wall south corner (Photo 1). 21' on top of parapet between Beams 2 and 5 is delaminated 6" down (Photo 2). [2011] Tilted 2° to the East. There is a 25' X $^{1}/_{8}$ "crack.	
	Wingwalls	Southwest Wingwall [2011/2013] No significant findings.	Northwest Wingwall [2011/2013] No significant findings.
	Seat	[2013] Spalls on the back side of bearings at Beams 2, 3, 4, 6 and 7. It appears that the anchor rods trying to pull out (Photos 3-7) [2009] Unsound concrete behind bearings.	
	Fixed Bearings	[2013] All bearings have faded paint and surface corrosion (CS2). [2009] Bearings tipped slightly towards river.	
	Slope Protection	[2013] Concrete slope has transverse and longitudinal crack (Photo 8). [2009] West slope has transverse cracks.	
Span #1	Span #1 (6 ft. slab at abutment)	Poured joints and both ends of slab.	
	Deck & General	[2013] 2"x28" spall forming at joint. Poured joint at end of deck has scattered areas (3') of failed adhesion.	[2011/2013] No significant findings.
	Rolled Steel Beams (56 ft. span)	[2013] 100% CS2.	[2013] 100% CS2.
Bent #2	Footings	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Pinned Columns	[2013] Tilted 3° to east.	[2013] Tilted 2° to east.

Bridge #9090 Inspection Field Notes

West Approach Spans #1-5 (North Dakota Side)			
Location	**Detail**	**South (Eastbound) Side**	**North (Westbound) Side**
Span #2 Rolled Beams	Pin & Hanger Joint #1 (Swivel)	[2013] UT not required until 2015. [2009] Center swivel pin has heavy corrosion & flaking rust from leaking deck joint. [2007] UT examination of pin.	[2013] Swivel joint on girder 2 is in CS4. UT not required until 2015. [2007] UT examination of pins.
	Deck Joint #1 (Strip Seal)	[2013] Slit on south end of gland.	[2013] 1’ cut in north end of gland.
	Deck & General	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Rolled Steel Beams (64 ft. span)	[2013] 12’ of CS4 on bottom flange of Girder 1 west end.	[2013] 15’ of CS4 on Girder 2 from hinge east bottom flange.
Bent #3	Footings	[2011/2013] No significant findings.	
	Pinned Columns	[2013] Tilted 3° to east.	[2013] Tilted 4° to east.
Span #3	Pin & Hanger Joint #2 (Swivel)	[2013] No change. UT not required until 2015. [2009] Center swivel pin has heavy corrosion & flaking rust from leaking deck joint.	[2011/2013] No significant findings.
	Deck Joint #2 (Strip Seal)	[2011/2013] No significant findings.	[2013] 1’ cut in north end of gland.
	Deck & General	[2013] Slit on south end of gland.	[2011/2013] No significant findings.
	Rolled Steel Beams	[2013] 12 of CS4 on bottom flange of Girder 1 west end.	

Bridge #9090 Inspection Field Notes

West Approach Spans #1-5 (North Dakota Side)			
Location	**Detail**	**South (Eastbound) Side**	**North (Westbound) Side**
Bent #4	Footings	[2011/2013] No significant findings. [2011] 7” of undermining at bearing 3.	[2011/2013] No significant findings.
	Pinned Columns	[2013] Tilted 1° to east.	[2013] Tilted 2° to east.
Span #4	Pin & Hanger Joint #3 (Swivel)	[2013] Swivel hinge is corroded (Photo 10) UT not required until 2015.	[2013] UT not required until 2015.
	Deck Joint #3 (Strip Seal)	[2013] Slit on south end of gland.	[2013] 1’ cut in north end of gland.
	Deck & General	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Rolled Steel Beams (64 ft. span)	[2013] 12’ of CS4 on bottom flange of Girder 1 west end.	[2011/2013] No significant findings.
Bent #5	Footings	[2013] Footings are exposed (Photo 9). [2009] Concrete footing undermined from flooding, w/exposed piling.	[2013] Footings are exposed (Photo 9). [2009] Concrete footing undermined from flooding.
	Pinned Columns	[2013] Tilted 3° to east.	[2013] Tilted 4° to east.
Span #5	Expansion Hinge (Joint #4)	[2013] UT not required until 2015.	[2013] UT not required until 2015.
	Deck Joint #4 (Finger Joint)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Deck & General	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Rolled Steel Beams (64 ft. span)	[2013] 100% CS2.	[2013] 100% CS2.
	Swivel Hinge (Joint #5)	[2013] UT not required until 2015.	[2013] UT not required until 2015. [2009] Heavy accumulation of pigeon debris.
	Deck Joint #5 (Strip Seal at Pier #6)	[2011/2013] No significant findings.	[2013] 18” cut in north end of gland.

Bridge #9090 Inspection Field Notes

Pier #6 (West end of West Truss Span):

Pier Notes:
[2013] The pier is severely twisted and bowed. There is 4-5/8” of deflection at the center of the pier. There are diagonal cracks up to ¼” wide. The east face pier cap has cracks 1/8” – ¼” wide (Photos 11-20).
[2011] No significant change.
[2009] Diagonal cracks in both faces - cap bowing 2” to east. Long-term movement (south column has moved 25” to the east).

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
Pier #6 Truss Expansion Rocker Bearings	[2013] Almost vertical, 0.6° tilt east. [2009] Missing bolt at bottom chord connection. [1972] Masonry plate reset approximately 5" to the west. Bearing tipped W, 1.13"/3.75" tip hts [2004] Upper rocker bearing plate reset 7" at bottom chord connection. Note* see D2 bearing measurement records. [1999] Masonry plate reset an additional 6" to the west. Take measurements to compare current position of masonry plate with pier centerline .	[2013] Bearing tipped east 3.6°. [2009] Bearing tipped W, 0.88"/4.25" tip hts.
L0 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] Corrosion on interior of panel point (Photo 21) [2009] MT detected no cracks in plug welds, both plates. [2007] UT detected lack of fusion in plug welds: 3 of 6 in inner plate.	[2013] No significant change. [2009] MT detected no cracks in plug welds, both plates. [2007] UT detected no lack of fusion in plug welds, both plates..
Floorbeam #0 (Pier #6)	[2013] Staining on north end. 6' of scattered CS3 on bottom flange.	
L0-U1 End Diagonal (50 KSI Steel)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
West Portal Brace	[2011/2013] No significant findings.	
L0-L1 Bottom Chord (100 KSI Steel)	[2013] No significant change. [2009] No further cracking found. [2007] Crack in chord at lower east corner of outer L0 gusset plate (ground out).	[2013] 1/2" pack bottom plate at L1. [2009] ½" pack rust, top plate @ end of truss
L0-L1 Stringers	[2013] 100% CS2	
L0-L1 Lower Lateral Bracing	[2013] Hanger rod broken. Pitting and corrosion.	
U1/2 Gusset Plates 5/8" thick (50 KSI Steel)	[2011/2013] No significant findings. [2011] Outer and outer plate bowed in $^{1}/_{8}$" on upper east free edge. [2009] Pitting & corrosion on inner plate and on edge stiffening angles.	[2011/2013] No significant findings. [2009] Outer plate edge stiffener, E side, bent @ top. Corroded tack weld on inner plate, W side.
U1/2-L1/2 Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
U1/2-L1 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L1/2 Gusset Plates	[2013] No significant change. [2009] MT detected no cracks in plug welds. Corrosion & thickness loss, inner plate @ chord shear line. [2007] UT detected lack of fusion in plug welds: 2 of 8 in outer plate, 5 of 8 in inner plate.	[2013] ½" long fusion defect in plug weld unchanged (Photo 22-23). [2007] UT detected lack of fusion in plug welds: 1 of 8 in outer plate. [2009] MT detected no cracks in plug welds.
U1 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] No significant change. [2009] 1/8" high spot on inner plate. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.
U1 Sway Frame	[2013] No significant change. [2011] Lots of pigeon waste on the top and bottom horizontal gusset plate.	
U1-L1 Vertical	[2013] Pitting in splash zone (Photo 24).	[2013] No significant change.
L1 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] ¼" pits [2011] $^{1}/_{8}$" bow outer plate west free edge. Pitting and corrosion on inside of panel point.	[2011/2013] No significant findings.
Floorbeam 1	[2013] Areas of flaking rust on bottom flange (10' CS3). 1' of CS4 on south end. [2011] Poured deck joint is leaking. Scattered freckled rust throughout.	
U1-U2 Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U1-U2 Upper Lateral Bracing	[2011/2013] No significant findings.	
U1-L2 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
1-2 Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
L1-L2 Bottom Chord (100 KSI steel west of field splice, 50 KSI to east)	[2013] Corrosion on splice at L2 – 2” x 2” x 3/16” pit (Photos 25-26).	[2011/2013] No significant findings.
L1-L2 Stringers	[2011/2013] No significant findings.	
L1-L2 Lower Lateral Bracing	[2011/2013] No significant findings. [2009] Deep pitting in angle vertical leg @ L1-S connection.	
U2 Gusset Plates 5/8” thick (50 KSI Steel)	[2013] No significant change. [2011] External plate has $^{1}/_{8}$” bow on east edge. There is pigeon waste in the box, most of the gusset plates have corrosion staining.	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Rust staining from sway frame connection, inner plate.
U2 Sway Frame	[2013] No significant change.	
U2-L2 Vertical		[2009] Active & arrested pitting @ lower end.
L2 Gusset Plates 5/8” thick (50 KSI Steel)	[2013] Corrosion on interior of panel point (Photo 27).	[2013]Small are of pitting around 1 bolt on interior gusset plate.
Floorbeam 2	[2013] 6’ of scattered CS3 on bottom flange. ½” pit on bottom flange on north end (Photo 28).	
U2-U3 Top Chord	[2013] No significant change. 2011] No significant findings.	[2013] No significant change.
U2-U3 Upper Lateral Bracing	[2013] No significant change. [2009] Small dents & distortion of top flange of diagonal bracing member.	
U2-L3 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
2-3 Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
L2-L3 Bottom Chord (50 KSI Steel)	[2011/2013] No significant findings.	[2013] Corrosion at drain connection.
L2-L3 Stringers	[2011/2013] No significant findings.	
L2-L3 Lower Lateral Bracing	[2013] CS4 on PPL2S horizontal gusset plate.	
U3 Gusset Plates 5/8" thick (50 KSI steel)	[2013] No significant change. [2009] Small bow (< 1/8") along bottom edge of inner plate. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.
U3 Sway Frame	[2011/2013] No significant findings.	
U3-L3 Vertical	[2011/2013] No significant findings.	[2013] No significant change. [2009] Corrosion blisters, W inner flange near bottom.
L3 Gusset Plates 5/8" thick (30 KSI Steel)	[2013] No significant change. [2009] .470" UT reading, inner plate. [2011] No change.	[2011/2013] No significant findings. [2009] PB UT readings of .365" and .375" on inner plate were checked; measured .620" and .614" respectively.
Floorbeam 3	[2013] Corrosion on top flange between Stringers 3-5 due to deck leakage (Photo 29). [2009] Top flange corrosion @ L3N.	
U3-U4 Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U3-U4 Upper Lateral Bracing	[2011/2013] No significant findings.	
U3-L4 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L3-L4 Lower Chord	[2013] Pack rust at horizontal gusset plate (Photo 30). [2009] Pack rust distortion, bottom plate of chord splice.	[2011/2013] No significant findings.
3-4 Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L3-L4 Stringers	[2011/2013] No significant findings.	

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
L3-L4 Lower Lateral Bracing	[2013] 2' of scattered CS4 north end (Photo 30).	
Deck L3-L4	[2011/2013] No significant findings.	
U4 Gusset Plates 5/8" thick (50 KSI steel)	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. External plate has $^{1}/_{8}$" box on west free edge.	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.
U4 Sway Frame	[2011/2013] No significant findings.	
U4-L4 Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings. [2009] Corrosion blisters, W inner flange and S outer flange, near bottom.
L4 Gusset Plates 5/8" thick (30 KSI S	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Floorbeam 4	[2013] 3' of scattered CS3 (Photos 31-32). [2009] Flaking rust & pack rust distortion in lower connection plate @ L4N.	
U4-U4' Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U4-U4' Upper Lateral Bracing	[2011/2013] No significant findings.	
U4-L4' Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U4'-L4 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L4-L4' Bottom Chord (50 KSI steel)	[2011/2013] No significant findings.	[2013] Corrosion at drain connections (typical).
L4-L4' Stringers	[2011/2013] No significant findings.	
L4-L4' Lower Lateral Bracing	[2011/2013] No significant findings.	
U4' Gusset	[2011/2013] No significant findings. [2011] There is pigeon waste in the box, most	[2013] No significant change. [2011] There is pigeon waste in the box,

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
Plates 5/8" thick (50 KSI steel)	of the gusset plates have corrosion staining.	most of the gusset plates have corrosion staining. Interior plate has $^{1}/_{8}$" bow on the east free edge.
U4' Sway Frame	[2011/2013] No significant findings.	
U4'-L4' Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L4' Gusset Plates 5/8" thick (30 KSI Steel)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Floorbeam 4'	[2013] Pitting and corrosion on bottom flange south end (Photo 33). 10' of staining and CS3 on north end top flange. [2009] Corrosion & pack rust distortion in lower connection plate @ L4'S.	
U4'-U3' Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings. [2009] 6 miss-drilled holes in outer vertical plate @ U3'N.
U4'-U3' Upper Lateral Bracing	[2011/2013] No significant findings.	
U3'-L4' Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
4'-3' Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L4'-L3' Bottom Chord (50 KSI steel) *Field Splice*	[2013] 1/8"-3/16" pitting @ L3' splice.	[2011/2013] No significant findings. [2009] Pack rust and fit-up distortion, top plate of chord splice.
L4'-L3' Stringers	[2011/2013] No significant findings.	
L4'-L3' Lower Lateral Bracing	[2011/2013] No significant findings.	

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
U3' Gusset Plates 5/8" thick (50 KSI steel)	[2011/2013] No significant findings. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. Exterior plate has $^{1}/_{8}$" bow on the east free edge.	[2011/2013] No significant findings. [2009] Arrested pitting on inner plate @ sway bracing connection. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.
U3' Sway Frame	[2011/2013] No significant findings.	
U3'-L3' Vertical	[2011/2013] No significant findings.	[2009] Paint failure & corrosion on inner face near bottom end.
L3' Gusset Plates 5/8" thick (30 KSI Steel)	[2013] Minor active pitting on interior of panel point.	[2013] Corrosion on interior of panel point (Photo 34).
Floorbeam 3'	[2013] No significant change. [2009] Top flange corrosion @ center deck joint.	
U3'-U2' Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U3'-U2' Upper Lateral Bracing	[2011/2013] No significant findings.	
U2'-L3' Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings. [2009] Corrosion blisters on S outer flange.
3'-2' Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L3'-L2' Bottom Chord (50 KSI steel)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L3'-L2' Stringers	[2011/2013] No significant findings.	
L3'-L2' Lower Lateral Bracing	[2011/2013] No significant findings.	
U2' Gusset Plates 5/8" thick (50 KSI steel)	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. Interior plate has $^{1}/_{8}$" bow on the west edge; the exterior has a $^{1}/_{8}$" bow on the east edge.	[2013] No significant change. [2009] Rust staining & arrested pitting @ sway frame connection. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
U2' Sway Frame	[2011/2013] No significant findings.	
U2'-L2' Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L2' Gusset Plates 5/8" thick (30 KSI Steel)	[2013] 2 sq ft of pitting and corrosion on ext gusset plate (Photo 35). [2011] UT thickness .661 west side inner plate.	[2013] No significant change. [2009] Extra drilled hole, inner plate.
Floorbeam 2'	[2013] No significant change. [2011] Transverse poured deck joint is leaking. [2009] Pitting and isolated thickness loss in bottom flange @ conn. to L2'N. UT .566" reading on 1.06" nom. flange.	
U2'-U1' Upper Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U2'-U1' Upper Lateral Bracing	[2011/2013] No significant findings.	
U1'-L2' Diagonal	[2013] No significant change. [2009] Corrosion blisters, inner flange, lower end.	[2011/2013] No significant findings.
2'-1' Horizontal Brace	[2011/2013] No significant findings.	
L2'-L1' Bottom Chord	[2011/2013] No significant findings. [2009] 3/8" pack rust in chord splice outer vertical plate.	[2011/2013] No significant findings.
L2'-L1' Stringers	[2011/2013] No significant findings.	
L2'-L1' Lower Lateral Bracing	[2011/2013] No significant findings.	
U1' Gusset Plates 5/8" thick (50 KSI steel)	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
U1' Sway Frame	[2011/2013] No significant findings.	
U1'-L1' Vertical	[2011/2013] No significant findings. [2009] Active & arrested pitting at bottom.	[2011/2013] No significant findings. [2009] Corrosion blisters on both flanges near bottom. Heavy arrested pitting on web facing E.
L1' Gusset Plates 5/8" thick (30 KSI Steel)	[2013] No significant change. [2009] Active & arrested pitting, inner plate.	[2011/2013] No significant findings.
Floorbeam 1'	[2013] Staining on south end due to leaking joint. [2011] Transverse poured deck joint is leaking. 50% of the top flange has corrosion.	
U1'-L0' End Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings. [2009] Isolated corrosion blisters on inner face near bottom.
East Portal Brace	[2011/2013] No significant findings.	
L1'-L0' Bottom Chord (30 KSI Steel)	[2011/2013] No significant findings. [2009] Corrosion & pitting in bottom plate @ L1'. Deck drain brackets rusted & separating from chord. [2011] steel laminations adjacent to 3rd hole east of L1.	[2011/2013] No significant findings. [2009] PR distortion in top plate @ end of truss.
L1'-L0' Stringers	[2011/2013] No significant findings.	
L1'-L0' Lower Lateral Bracing	[2011/2013] No significant findings.	
L0' Gusset Plates	[2013] Corrosion on interior of interior gusset plate. [2009] .552" UT reading, inner plate; .538" UT reading, outer plate.	[2011/2013] No significant findings.
Floorbeam 0'	[2013] No significant findings. [2011] 50% of the top flange has corrosion.	
Pier #7 Bearings	South truss has a rocker expansion bearing [2013] Rocker bearings have minor to moderate surface corrosion (Photo 36). Bearings tipped to the east 1.50" and 3/25" SW bearing and 2" and 3" SE bearing low and high tip points respectively. [2011] The bearings are the same as 2009, 1 3/8" and 2 3/8" low and high tip points for both bearings.	North truss has a fixed bearing. [2011/2013] No significant findings. There is a swivel pin connecting the truss ends at the north bearing. [2013] No change. [2009] Heavy corrosion in swivel joint above bearings.

Bridge #9090 Inspection Field Notes

Pier #7 (Center Pier)

Pier Notes: [2011/2013] No significant findings. [2009] Minor vertical cracking, both faces.

Deck Joint #6 (Strip Seal at Pier #7): [2013] No change. [2009] Tear in gland near center.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
Pier #7 Bearings	South truss has a rocker expansion bearing. [2013] Rocker bearings have minor to moderate surface corrosion. Bearings tipped to the east but ½" less than previous years due to lower temperatures likely (52 degrees) – see bearing spreadsheet (page 39). [2011] The bearings are the same as 2009, 1 3/8" SW bearing and 2 3/8" SE bearing on the low ends tipped to the east.	There is a swivel pin connecting the truss ends at the north bearing. [2013] No change. [2009] Heavy corrosion in swivel joint above bearings. North truss has a fixed bearing. [2013] Fixed bearings have minor surface corrosion.
Floorbeam 0'	[2013] Top flange corrosion north end (Photo 37).	
L0' Gusset Plates 5/8" thick (50 KSI steel)	[2013] Outside plate leaking corrosion from top (Photo 38). [2008] Weld crack at bottom chord removed.	[2013] Leaking corrosion at pin between east and west truss (Photo 39). [2009] Isolated pitting in both plates, top edge of chord.
L0'-U1' End Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
West Portal Brace	[2013] No change. [2009] Paint failure on upper horizontal member.	
L1'-L0' Bottom Chord (30 KSI Steel)	[2011/2013] No significant findings.	[2013] Broken drain near L1'N. [2009] Corrosion & pack rust between chord & lower diagonal bracing conn. plate.
L0'-L1' Stringers	[2011/2013] No significant findings.	
L0'-L1' Lower Lateral Bracing	[2013] Bottom horizontal connection plate heavy corrosion and pack rust at L0'S (Photo 40).	
U1' Gusset Plates 5/8" thick (50 KSI steel)	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Bowing 1/8" on east edge of inner plate (fit-up).	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Paint failure with duct tape, outer plate. Rust staining on inner plate from sway frame connection.
U1' Sway Frame	[2013] No change. [2009] Paint failure on lower horizontal member @ U1'N.	
U1'-L1' Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
L1' Gusset Plates 5/8" thick (30 KSI Steel)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Floorbeam 1'	[2013] South bottom flange paint failure and water ponding (Photo 41). Bottom of web on south end has corrosion and loss up to 1/8" (Photo 42). SE face of bottom web has an undercut at the bottom flange (Photo 43). [2011] Top flange corrosion the full length. [2009] Isolated paint failure & heavy corrosion in bottom flange near L2'N.	
U1'-U2' Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U1'-U2' Upper Lateral Bracing	[2011/2013] No significant findings.	
U1'-L2' Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
1'-2' Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L1'-L2' Bottom Chord (30 KSI Steel) *Field Splice*	[2011/2013] No significant findings.	[2013] Broken drain near L1'N.
L1'-L2' Stringers	[2011/2013] No significant findings.	
L1'-L2' Lower Lateral Bracing	[2011/2013] No significant findings.	
U2' Gusset Plates 5/8" thick (50 KSI steel)	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. The exterior plate has a $^{1}/_{8}$" bow on the East edge.	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Rust staining on inner plate from sway frame connection.
U2' Sway Frame	[2011/2013] No significant findings.	
U2'-L2' Vertical	[2013] No change. [2009] Corrosion blisters, lower end.	[2011/2013] No significant findings.
L2' Gusset Plates 5/8" thick (30 KSI Steel)	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
Floorbeam 2'	[2013] Bottom flange corrosion along entire length (Photo 44). [2009/2013] Top flange corrosion, north end, due to deck leakage (Photo 45).	
U2'-U3' Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U2'-U3' Upper Lateral Bracing	[2011/2013] No significant findings.	
U2'-L3' Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
2'-3' Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L2'-L3' Bottom Chord (50 KSI steel)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L2'-L3' Stringers	[2011/2013] No significant findings.	
L2'-L3' Lower Lateral Bracing	[2011/2013] No significant findings.	
U3' Gusset Plates 5/8" thick (50 KSI steel)	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Bowing < 1/8" bottom edge of outer plate (fit-up).	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.
U3' Sway Frame	[2011/2013] No significant findings.	
U3'-L3' Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L3' Gusset Plates 5/8" thick (30 KSI Steel)	[2013] No significant change. [2009] Pitting, inner plate, UT .423" reading.	[2011/2013] No significant findings.
Floorbeam 3'	[2013] No change. [2011] The top flange has corrosion the entire length, is worse at the panel point. [2009] Corrosion & pitting in lower part of web, E side near L3'N connection.	
U3'-U4' Upper Chord	[2011/2013] No significant findings.	[2013] No change. [2011] There are miss-drilled holes at the U3' connection.
U3'-U4' Upper Lateral Bracing	[2011/2013] No significant findings.	
U3'-L4' Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
3'-4' Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L3'-L4' Bottom Chord (50 KSI steel) *Field Splice*	[2011/2013] No significant findings.	[2013] No change. [2009] Isolated corrosion blisters, inner vertical plate.
L3'-L4' Stringers	[2011/2013] No significant findings.	
L3'-L4' Lower Lateral Bracing	[2013] Support from Stringer 2 rod is broken off near FB 4'.	
U4' Gusset Plates 5/8" thick (50 KSI steel)	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Bowing < 1/8" west edge of inner plate (fit-up).	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. There is a $^{1}/_{8}$" bow on the west edge of the exterior plate.
U4' Sway Frame	[2011/2013] No significant findings.	
U4'-L4' Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L4' Gusset Plates 5/8" thick (30 KSI Steel)	[2013] No change. [2011] External gusset plate east free edge bowed $^{1}/_{16}$"	[2013] Leaking rust from corrosion spots on outside plate – inside plate. [2011] Pack rust along the horizontal gusset plate at PP.
Floorbeam 4'	[2013] No change. [2011] Pack rust along the horizontal gusset plate at PP. The top flange has flaking rust and pitting. [2009] Flaking rust & pitting in lower flange @ conn. to L4'S.	
U4'-U4 Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U4'-U4 Upper Lateral Bracing	[2011/2013] No significant findings.	
U4'-L4 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U4-L4' Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L4'-L4 Bottom Chord (50 KSI steel)	[2011/2013] Minor paint/corrosion blistering in the interior and exterior.	[2011/2013] Spotted surface corrosion throughout, isolated pitting (Photos 46-47).
L4'-L4 Stringers	[2011/2013] No significant findings.	

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
L4'-L4 Lower Lateral Bracing	[2011/2013] No significant findings.	
U4 Gusset Plates 5/8" thick (50 KSI steel)	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Bowing < 1/8" (fit-up) on east side of outer plate.	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.
U4 Sway Frame	[2011/2013] No significant findings.	
U4-L4 Vertical	[2013] No change. [2009] Corrosion blisters on north face @ deck level.	[2011/2013] No significant findings.
L4 Gusset Plates 5/8" thick (30 KSI Steel)	[2013] No change. [2011] Exterior gusset plate east top free edge bowed in $^{1}/_{16}$".	[2013] No change. [2009] Isolated corrosion around connecting bolts to L4-U3, outer plate. Inner plate UT .540" min.
Floorbeam 4	[2013] Corrosion and pack rust at bottom flange to connection at L4S (Photos 48). Top flange corrosion at L4N (Photo 49). [2009] Corrosion & pack rust in bottom flange @ connection to L4N.	
U4-U3 Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U4-U3 Upper Lateral Bracing	[2011/2013] No significant findings.	
U3-L4 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
4-3 Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L4-L3 Bottom Chord (50 KSI steel) *Field Splice*	[2011/2013] No significant findings.	[2011/2013] Spotted surface corrosion throughout. [2009] Isolated pitting, several locations on outer plate, 1/8" – 3/16" deep.
L4-L3 Stringers	[2011/2013] No significant findings.	
L4-L3 Lower Lateral Bracing	[2013] No change. [2009] Broken hanger bar near FB4.	
U3 Gusset Plates 5/8" thick (50 KSI steel)	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Bowing < 1/8" (fit-up) on east edge of outer plate.	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
U3 Sway Frame	[2011/2013] No significant findings.	
U3-L3 Vertical	[2011/2013] No significant findings.	[2009/2013] Corrosion blisters on web and inner flange, bottom end (Photo 50). There are miss-drilled holes at the mid lateral bracing connection.
L3 Gusset Plates 5/8" thick (30 KSI Steel)	[2011/2013] No significant findings.	[2013] No change. [2009] Corrosion & minor S/L, inner plate.
Floorbeam 3	[2013] No change. [2009] Corrosion & pack rust in bottom flange @ connection to L3N. Corrosion in top flange, N end, due to deck leakage.	
U3-U2 Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U3-U2 Upper Lateral Bracing	[2011/2013] No significant findings.	
U2-L3 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
3-2 Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L3-L2 Bottom Chord (50 KSI Steel)	[2013] No change. [2009] Freckled rust on bottom chord plate @ L2S.	[2013] Spotted corrosion throughout (Photo 51).
L3-L2 Stringers	[2011/2013] No significant findings.	
L3-L2 Lower Lateral Bracing	[2011/2013] No significant findings.	
U2 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Cracked tack weld, inner plate. Bow in outer plate bottom edge approx. 1/8".	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.
U2 Sway Frame	[2011/2013] No significant findings.	
U2-L2 Vertical	[2013] No change. [2009] Paint failure and corrosion on north face near bottom.	[2013] No change. [2009] Corrosion blisters on web and inner flange, bottom end.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
L2 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] Previous loss inside of both plates painted over – leaking rust now through paint. [2009] Active & arrested pitting, inner plate. UT 0.41" min. inner plate @ top of chord.	[2011/2013] No significant findings.
Floorbeam 2	[2013] Top flange corrosion at L2N (Photo 52). [2009] Pitting & minor S/L in bottom flange @ L2S. 8' heavy corrosion on top flange.	
U2-U1 Top Chord	[2013] No change. [2009] Large area of paint failure on south face near U1S.	[2011/2013] No significant findings.
U2-U1 Upper Lateral Bracing	[2011/2013] No significant findings.	
U1-L2 Diagonal	[2013] ¼" pack rust at L2S connection. [2009] Isolated corrosion blisters on north face near bottom. Heavier corrosion on inner surface of north flange.	[2011/2013] No significant findings.
2-1 Horizontal Bracing	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L2-L1 Bottom Chord (100 KSI steel east of field splice, 50 KSI to west)	[2011/2013] No significant findings.	[2013] No change. [2009] Corrosion & pack rust on bottom of splice near PP L2N.
L2-L1 Stringers	[2011/2013] No significant findings.	
L2-L1 Lower Lateral Bracing	[2011/2013] No significant findings.	
U1 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Freckling rust on inside surface of outer plate.	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Rust staining on inner plate from sway frame connection.
U1 Sway Frame	[2011/2013] No significant findings.	
U1-L1 Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L1 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] No change. [2009] Large area of pitting (arrested) on inner plate. UT .591" min. @ top of chord.	[2013] No change. [2011] Corrosion near bolts, west side. Interior gusset plated top east free edge bowed in $^{1}/_{8}$".

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
Floorbeam 1	[2013] Top flange section loss of 1/8" on south end (Photo 53). [2009] Corrosion & pitting in web @ bottom flange, S end. Flaking rust on top flange between deck edge and truss, both ends.	
U1/2 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] No change. [2009] Bowing <1/8" on west edge of outer plate (fit-up). West stiffening angle has cracked tack welds and partial separation from plate due to pack rust (has been repaired by bolting).	[2013] No change. [2009] Bowing < 1/8" on west edge of outer plate. Corrosion and cracked tack welds on edge stiffening angles. Bolted repairs made. The interior plate is bowed $^{1}/_{8}$" in on the west edge.
U1/2-L1/2 Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U1/2-L1 Diagonal	[2013] No change. [2009] Isolated surface corrosion on outer face near bottom.	[2013] No significant findings.
L1/2 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] No change. [2009] 1" crack in plug weld on outer plate. Crack was ground out; slag inclusions found between plate and chord. No propagation into chord.	[2013] No change. [2009] MT detected no cracks in plug welds. Stiffening angle on inner plate detached due to pack rust, E side. UT 0.42" min. Corrosion blisters & pack rust on outer plate, E side. W side stiffening angles partly detached due to pack rust. [2007] UT detected lack of fusion in plug welds: 1 of 8 in outer plate. Outer plate has surface repair weld.
U1-L0 End Diagonal	[2011/2013] No significant findings.	[2013] No significant change. [2009] Heavy corrosion & paint failure on inner flange @ PP L0N. Isolated corrosion blisters on south face near bottom.
East Portal Brace	[2011/2013] No significant findings.	
L1-L0 Bottom Chord (100 KSI Steel)	[2013] No change. [2008] Transverse crack through fillet weld between lower edge L0S outer gusset and chord, propagated into full-pen. Flange/web weld of chord. Crack was fully ground out. Erroneously labeled as PP L0'S in 2008 reports.	[2011/2013] No significant findings.
L1-L0 Stringers	[2011/2013] No significant findings.	
L1-L0 Lower Lateral Bracing	[2013] Heavily corroded bracing on south end and cracked drain bracket (Photo 54). Hanger broke off near FB1S (Photo 55). North inside brace corrosion.	

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
L0 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] Top plate still warped. Outside plate leaking rust from top. [2009] Top cover plate warped 1/2"-3/4" from pack rust.	[2013] Top plate between inner and outer gusset is warped downward ½". [2009] Heavy corrosion on inner plate above chord line. UT .509" min.
Floorbeam 0	[2013] Heavy leaking rust from top flange at Pier 8.	
Pier #8 Truss Expansion Rocker Bearings	There are no records of this bearing ever being reset - take measurements to compare current position of masonry plate with pier centerline? [2013] Bearing tipped east 1 7/8" and 2 7/8" low and high tip points. [2011] Same measurements as in 2009. [2010] Bearing tipped east 1.50" and 3.25" low and high tip points. [2009] Bearing tipped E, 1.75"/3.00" tip hts.	There are no records of this bearing ever being reset - take measurements to compare current position of masonry plate with pier centerline? [2013] Bearing tipped east 1 ½ " and 3 5/14" low and high tip points. [2011] Same measurements as in 2009. [2010] Bearing tipped east 1.75" and 3" low and high tip heights. [2009] Bearing tipped E, 1.50"/3.25" tip hts.

Pier #8 (East end of East Truss Span)

Pier Notes: [2013] Previous graffiti painted over, but new graffiti is present – west face.
[2009] Minor cracking & staining W face.

Bridge #9090 Inspection Field Notes

Br #9090 East Approach Spans #8-13 (Minnesota Side)			
Location	**Detail**	**South (Eastbound) Side**	**North (Westbound) Side**
Span #8	Deck Joint #7 (Strip Seal at Pier #8)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Swivel Hinge (Joint #7)	[2013] Underdeck cracking, spalling, and efflorescence	[2013] Underdeck cracking and efflorescence.
	Deck & General	[2011/2013] No significant findings.	[2013] Overhang delamination typical in all spans (Photo 56).
	Rolled Steel Beams (64 ft. span)	[2011/2013] No significant findings.	[2013] Beam 4 leaking rust from top flange at Pier 8 (Photo 57).
	Deck Joint #8 (Finger Joint)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Expansion Hinge (Joint #8)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Bent #9	Footings	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Pinned Columns	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Span #9	Deck & General	[2013] Underdeck cracking, spalling, and efflorescence.	[2013] Overhang delamination typical in all spans (Photo 58).
	Rolled Steel Beams (64 ft. span)	[2011/2013] No significant findings.	[2013] Beams 2 & 6 bottom flange flaking rust at pin and hanger.
	Pin & Hanger (Swivel) Joint #9	[2011/2013] No significant findings.	[2013] Girder 3 has a note on the bottom pin from 5-2-01 of a crack…just a tack weld crack on the nut.
	Deck Joint #9 (Strip Seal)	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br #9090 East Approach Spans #8-13 (Minnesota Side)			
Location	**Detail**	**South (Eastbound) Side**	**North (Westbound) Side**
Bent #10	Footings	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Pinned Columns	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Span #10	Deck & General	[2013] Underdeck cracking, spalling, and efflorescence. Saturation also present (Photo 59).	[2013] Overhang delamination typical in all spans.
	Rolled Steel Beams (64 ft. span)	[2011/2013] No significant findings.	[2013] No change. [2009] Surface corrosion, bottom flange B1.
	Pin & Hanger (Swivel) Joint #10	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Deck Joint #10 (Strip Seal)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Bent #11	Footings	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Pinned Columns	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Span #11	Deck & General	[2013] Underdeck cracking, spalling, and efflorescence.	[2013] Overhang delamination typical in all spans.
	Rolled Steel Beams (64 ft. span)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Pin & Hanger (Swivel) Joint #11	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Deck Joint #11 (Strip Seal)	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br #9090 East Approach Spans #8-13 (Minnesota Side)			
Location	**Detail**	**South (Eastbound) Side**	**North (Westbound) Side**
Bent #12	Footings	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Pinned Columns	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Span #12	Deck & General	[2013] Underdeck cracking, spalling, and efflorescence. [2003] Roadway removed and replaced with Greenway Trail.	[2013] Overhang delamination typical in all spans – exposed rebar. [2003] Roadway removed and replaced with Greenway Trail.
	Rolled Steel Beams (64 ft. span)	[2009/2013] No change. [2003] Bottom flange impact damage, B7 & B8. MT detected no cracks.	[2011/2013] No significant findings.
	Pin & Hanger (Swivel) Joint #12	[2011/2013] No significant findings.	[2013] Beam 3 offset ¾”.
	Deck Joint #12 (Strip Seal)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Bent #12	Footings	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Pinned Columns	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Span #13	Deck & General	[2013] Underdeck cracking, spalling, and efflorescence.	[2013] Overhang delamination typical in all spans – exposed rebar.
	Rolled Steel Beams (56 ft. span)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Span #13 (6 ft. slab at abutment)	Poured joints and both ends of slab.	Poured joints and both ends of slab.

Bridge #9090 Inspection Field Notes

Br #9090 East Approach Spans #8-13 (Minnesota Side)			
Location	**Detail**	**South (Eastbound) Side**	**North (Westbound) Side**
East Abutment	Slope Protection	[2013] Graffiti painted over. [2009/2013] East slope separating from abutment at north side 2”-4”, center panels heaving.	
	Fixed Bearings	[2013] Center fixed bearing corrosion.	[2013] Center fixed bearing corrosion. North fixed bearing heavy leaking corrosion at beam end (Photo 60).
	Seat	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Wingwalls	[2011/2013] No significant findings.	[2011/2013] No significant findings.

General Notes

Deck:
[2013] 8” spall 15’ west of strip seal 9 east bound lane. Open cracks and scattered areas of delamination between strip seal 7 & 8. 47” x 28” patch east of strip seal 7 at fog line. This section of deck has the worst area of open cracks. 93” x 69” patch east of west finger joint. East end of patch is deteriorating.
[2009] Spalling w/exposed rebar in overhang, both sides, both approaches. Lesser spalling in truss spans. Heavy spalling over floorbeams @ most panel points.

Median:
[2013] Cracked, spalled, delaminated and deteriorated concrete along sides. 25’ of spalled concrete along median joint on east end of bridge (Photo 61).

Curbs:
[2013] There are minor spalls at several of the poured joints (Photo 62). There is a 15’ on the north curb (Photo 63).

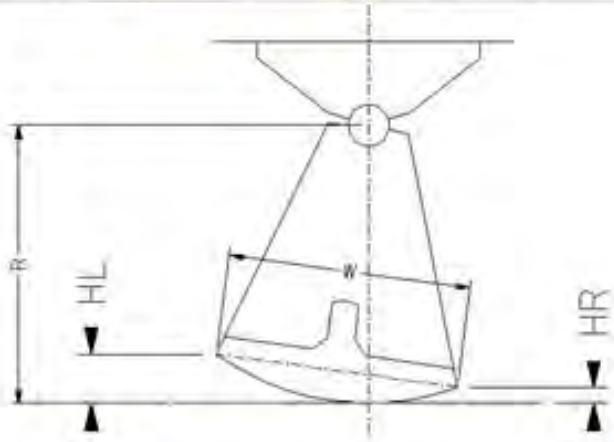
Railing:
[2013] The metal railings are in CS2. There are cracks throughout the concrete railing face (Photo 64). There is a broken rail post on the north rail on the east approach (Photo 65).

Signing:
[2013] No significant findings.

Bridge #9090 Inspection Field Notes

Measurement of Rocker Bearing Tipping Angles

Bridge	9090	Measured By	JLZ
Inspection Date	6/3/2013	Calculations By	JMF



Angle (q) = ARCSIN [(HL - HR) / W]
Displacement = (p * q * R) / 180

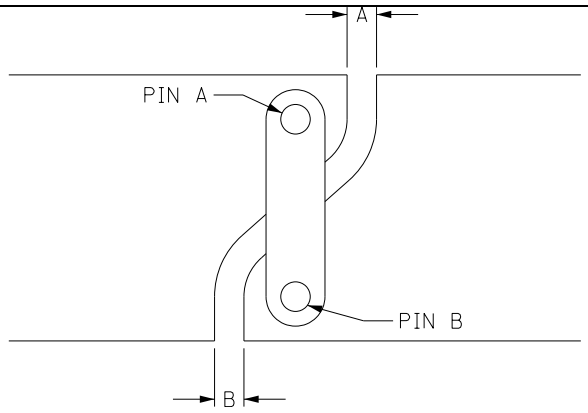
Truss	Sub.	Bearing	W (in)	R (in)	HL (in)	HR (in)	Angle	Displace-	Tip Dir.	Temp
2010	Pier 6	NW	24.00	32.00	3.75	1.13	6.3	3.5	W	83
2011	Pier 6	NW	24.00	32.00	4.75	1.00	9.0	5.0	W	70
2013	Pier 6	NW	24.00	32.00						
2010	Pier 6	SW	24.00	32.00	4.25	0.88	8.1	4.5	W	75
2011	Pier 6	SW	24.00	32.00	4.50	0.75	9.0	5.0	W	70
2013	Pier 6	SW	24.00	32.00						
2010	Pier 7	SW	24.00	32.00	2.38	1.38	2.4	1.3	E	83
2011	Pier 7	SW	24.00	32.00	2.38	1.38	2.4	1.3	E	70
2013	Pier 7	SW	24.00	32.00	3.25	1.50	4.2	2.3	E	52
2010	Pier 7	SE	24.00	32.00	2.38	1.38	2.4	1.3	E	75
2011	Pier 7	SE	24.00	32.00	2.38	1.38	2.4	1.3	E	70
2013	Pier 7	SE	24.00	32.00	3.00	2.00	2.4	1.3	E	52
2010	Pier 8	NE	24.00	32.00	3.00	1.75	3.0	1.7	E	85
2011	Pier 8	NE	24.00	32.00	3.25	1.50	4.2	2.3	E	60
2013	Pier 8	NE	24.00	32.00	3.36	1.50	4.4	2.5	E	52
2010	Pier 8	SE	24.00	32.00	3.25	1.50	4.2	2.3	E	80
2011	Pier 8	SE	24.00	32.00	3.00	1.75	3.0	1.7	E	60
2013	Pier 8	SE	24.00	32.00	2.88	1.88	2.4	1.3	E	52

Bridge #9090 Inspection Field Notes

Bent Angle Measurements							
Bent	North Bearing		Center Bearing		South Bearing		Comments
	2011	2013	2011	2013	2011	2013	
2	0°		0°		0°		
3	0°		0°		0°		
4	0°		1°W		1°W		
5	0°		0°		0°		
9	1°E	1°E	0°	1°W	0°	1°W	[2013] 61°F
10	1°E	0°	1°E	1°E	1°E	2°W	[2013] 61°F
11	2°W	0°	1.5°W	1°W	1.5°W	2°W	[2013] 61°F
12	0°	1°W	0°	1°W	0°	1°W	[2013] 61°F
13	2°W	1.5°W	2.5°W	1.5°W	3°W	2°W	[2013] 61°F

Bridge #9090 Inspection Field Notes

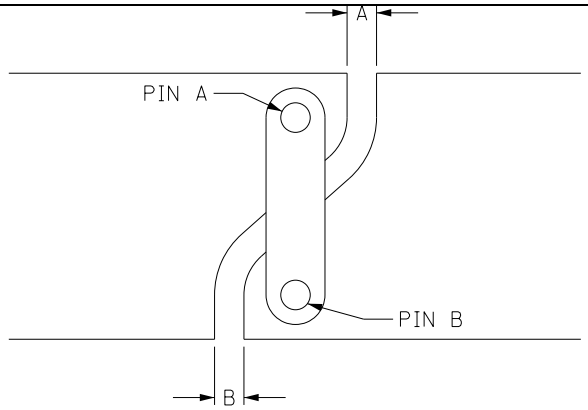
Expansion Joint Gap Measurements



Span		Girder 1		Girder 2		Girder 3		Girder 4		UT Result
		2011	2013	2011	2013	2011	2013	2011	2013	
2	A	5		5		5		5		OK
	B	2 1/2		2 1/2		2 ½		2 ¾		OK
3	A	7		6 ¾		6 ¾		6 ¾		OK
	B	5		4 5/8		4 ¼		4 5/8		OK
4	A	6 5/8		6 ¼		6 ¾		6 ¾		OK
	B	4 ½		4 ½		4 ¾		4 ¾		OK
5W	A									
	B									
8E	A									
	B									
9	A	4 ½	5 ¼	4 ½	5 ¼	5 ¾	5 ¼	5 ¼	5	OK
	B	2 ¼	3	2 5/8	2 3/4	3	2 5/8	3	2 3/4	OK
10	A	4 ½	5	5 3/8	5	5 ¼	5	5 ½	5	OK
	B	2	2 7/8	3	2 7/8	2 5/8	2 7/8	3 1/8	3 7/8	OK
11	A	5 3/8	5 1/8	5 1/8	5	4 7/8	5 ¼	4 ¾	5 ¼	OK
	B	3 1/16	2 7/8	3 1/16	2 7/8	3	3 1/8	2 ½	2 3/4	OK
12	A	2 3/8	2 5/8	4 7/16	5 1/8	5 ¼	4 7/8	5 ½	4 ¾	OK
	B	4 ¾	4 ¾	2 7/16	2 7/8	2 ½	2 7/8	3	2 3/4	OK

Bridge #9090 Inspection Field Notes

Expansion Joint Gap Measurements



Span	Girder 5		Girder 6		Girder 7		Girder 8		UT Result
	2011	2013	2011	2013	2011	2013	2011	2013	
2 A	5 ¼		5		5 ¼		5 ½		OK
B	2 ¾		2 ½		2 ½		2 ¾		OK
3 A	6 ½		6 ¾		6 ¾		7 1/8		OK
B	4 ½		4 ¾		4 7/8		4 7/8		OK
4 A	6 ½		6 ½		6 ¾		6 ½		OK
B	4 ½		4 ½		4 ¾		4 ½		OK
5W A							7 ½		OK
B							7 ½		OK
8E A									
B									
9 A	5 ¼	5 ¼	5 1/8	5 3/8	5 ½	4 ½	5 ¼	4 ¼	OK
B	2 ½	3 1/8	2 5/8	3 ¼	3	2 ½	2 ¾	2 1/8	OK
10 A	5	5 ½	5	5 1/8	5	4 ¾	5	4 ½	OK
B	2 7/8	3 1/4	2 7/8	2 5/8	2 7/8	2 ¾	2 5/8	2 1/8	OK
11 A	5	4 7/8	5 ¼	4 7/8	5	5	5 ¼	5	OK
B	2 ¾	2 5/8	3	3	2 ¾	3 1/8	2 7/8	2 ¾	OK
12 A	4 ¾	5 ½	4 5/16	5 ½	2 9/16	4 ¾	4 ¾	4 ¾	OK
B	2 7/16	3 3/8	2 7/8	2 ¾	5	2 ¼	2 9/16	2 ½	OK

Bridge #9090 Inspection Field Notes

West Approach Spans #1-5 (North Dakota Side)			
Location	**Detail**	**South (Eastbound) Side**	**North (Westbound) Side**
West Abutment	Abutment	[2013] 30" long x 16" wide at top of parapet wall south corner (Photo 1). 21' on top of parapet between beams 2 and 5 is delaminated 6" down (Photo 2). [2011] Tilted 2° to the East. There is a 25' X $^{1}/_{8}$ "crack.	
	Wingwalls	Southwest Wingwall [2011/2013] No significant findings.	Northwest Wingwall [2011/2013] No significant findings.
	Seat	[2013] Spalls on the back side of bearings at beams 2, 3, 4, 6 and 7. It appears that the anchor rods trying to pull out (Photos 3-7) [2009] Unsound concrete behind bearings.	
	Fixed Bearings	[2013] All bearings have faded paint and surface corrosion (CS2). [2009] Bearings tipped slightly towards river.	
	Slope Protection	[2013] Concrete slope has transverse and longitudinal crack (Photo 8). [2009] West slope has transverse cracks.	
Span #1	Span #1 (6 ft. slab at abutment)	Poured joints and both ends of slab.	
	Deck & General	[2013] 2"x28" spall forming at joint. Poured joint at end of deck has scattered areas (3') of failed adhesion.	[2011/2013] No significant findings.
	Rolled Steel Beams (56 ft. span)	[2013] 100% CS2.	[2013] 100% CS2.
Bent #2	Footings	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Pinned Columns	[2013] Tilted 3° to east.	[2013] Tilted 2° to east.

Bridge #9090 Inspection Field Notes

West Approach Spans #1-5 (North Dakota Side)			
Location	**Detail**	**South (Eastbound) Side**	**North (Westbound) Side**
Span #2 Rolled Beams	Pin & Hanger Joint #1 (Swivel)	[2013] UT not required until 2015. [2009] Center swivel pin has heavy corrosion & flaking rust from leaking deck joint. [2007] UT examination of pin.	[2013] Swivel joint on girder 2 is in CS4. UT not required until 2015. [2007] UT examination of pins.
	Deck Joint #1 (Strip Seal)	[2013] Slit on south end of gland.	[2013] 1’ cut in north end of gland.
	Deck & General	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Rolled Steel Beams (64 ft. span)	[2013] 12’ of CS4 on bottom flange of Girder 1 west end.	[2013] 15’ of CS4 on Girder 2 from hinge east bottom flange.
Bent #3	Footings	[2011/2013] No significant findings.	
	Pinned Columns	[2013] Tilted 3° to east.	[2013] Tilted 4° to east.
Span #3	Pin & Hanger Joint #2 (Swivel)	[2013] No change. UT not required until 2015. [2009] Center swivel pin has heavy corrosion & flaking rust from leaking deck joint.	[2011/2013] No significant findings.
	Deck Joint #2 (Strip Seal)	[2011/2013] No significant findings.	[2013] 1’ cut in north end of gland.
	Deck & General	[2013] Slit on south end of gland.	[2011/2013] No significant findings.
	Rolled Steel Beams	[2013] 12 of CS4 on bottom flange of Girder 1 west end.	

Bridge #9090 Inspection Field Notes

West Approach Spans #1-5 (North Dakota Side)			
Location	**Detail**	**South (Eastbound) Side**	**North (Westbound) Side**
Bent #4	Footings	[2011/2013] No significant findings. [2011] 7” of undermining at bearing 3.	[2011/2013] No significant findings.
	Pinned Columns	[2013] Tilted 1° to east.	[2013] Tilted 2° to east.
Span #4	Pin & Hanger Joint #3 (Swivel)	[2013] Swivel hinge is corroded (Photo 10) UT not required until 2015.	[2013] UT not required until 2015.
	Deck Joint #3 (Strip Seal)	[2013] Slit on south end of gland.	[2013] 1’ cut in north end of gland.
	Deck & General	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Rolled Steel Beams (64 ft. span)	[2013] 12’ of CS4 on bottom flange of Girder 1 west end.	[2011/2013] No significant findings.
Bent #5	Footings	[2013] footings are exposed (Photo 9). [2009] Concrete footing undermined from flooding, w/exposed piling.	[2013] Footings are exposed (Photo 9). [2009] Concrete footing undermined from flooding.
	Pinned Columns	[2013] Tilted 3° to east.	[2013] Tilted 4° to east.
Span #5	Expansion Hinge (Joint #4)	[2013] UT not required until 2015.	[2013] UT not required until 2015.
	Deck Joint #4 (Finger Joint)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Deck & General	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Rolled Steel Beams (64 ft. span)	[2013] 100% CS2.	[2013] 100% CS2.
	Swivel Hinge (Joint #5)	[2013] UT not required until 2015.	[2013] UT not required until 2015. [2009] Heavy accumulation of pigeon debris.
	Deck Joint #5 (Strip Seal at Pier #6)	[2011/2013] No significant findings.	[2013] 18” cut in north end of gland.

Bridge #9090 Inspection Field Notes

Pier #6 (West end of West Truss Span):

Pier Notes:
[2013] The pier is severely twisted and bowed There is 4-5/8” of deflection at the center of the pier. There are diagonal cracks up to ¼” wide. The east face pier cap has cracks 1/8” – ¼” wide (Photos 11-20).
[2011] No significant change.
[2009] Diagonal cracks in both faces - cap bowing 2” to east. Long-term movement (south column has moved 25” to the east).

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
Pier #6 Truss Expansion Rocker Bearings	[2013] Almost vertical, 0.6° tilt east. [2009] Missing bolt at bottom chord connection. [1972] Masonry plate reset approximately 5" to the west. Bearing tipped W, 1.13"/3.75" tip hts [2004] Upper rocker bearing plate reset 7" at bottom chord connection. Note* see D2 bearing measurement records. [1999] Masonry plate reset an additional 6" to the west. Take measurements to compare current position of masonry plate with pier centerline .	[2013] Bearing tipped east 3.6°. [2009] Bearing tipped W, 0.88"/4.25" tip hts.
L0 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] Corrosion on interior of panel point (Photo 21) [2009] MT detected no cracks in plug welds, both plates. [2007] UT detected lack of fusion in plug welds: 3 of 6 in inner plate.	[2013] No significant change. [2009] MT detected no cracks in plug welds, both plates. [2007] UT detected no lack of fusion in plug welds, both plates..
Floorbeam #0 (Pier #6)	[2013] Staining on north end. 6' of scattered CS3 on bottom flange.	
L0-U1 End Diagonal (50 KSI Steel)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
West Portal Brace	[2011/2013] No significant findings.	
L0-L1 Bottom Chord (100 KSI Steel)	[2013] No significant change. [2009] No further cracking found. [2007] Crack in chord at lower east corner of outer L0 gusset plate (ground out).	[2013] 1/2" pack bottom plate at L1. [2009] ½" pack rust, top plate @ end of truss
L0-L1 Stringers	[2013] 100% CS2	
L0-L1 Lower Lateral Bracing	[2013] Hanger rod broken. Pitting and corrosion.	
U1/2 Gusset Plates 5/8" thick (50 KSI Steel)	[2011/2013] No significant findings. [2011] Outer and outer plate bowed in $^{1}/_{8}$" on upper east free edge. [2009] Pitting & corrosion on inner plate and on edge stiffening angles.	[2011/2013] No significant findings. [2009] Outer plate edge stiffener, E side, bent @ top. Corroded tack weld on inner plate, W side.
U1/2-L1/2 Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
U1/2-L1 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L1/2 Gusset Plates	[2013] No significant change. [2009] MT detected no cracks in plug welds. Corrosion & thickness loss, inner plate @ chord shear line. [2007] UT detected lack of fusion in plug welds: 2 of 8 in outer plate, 5 of 8 in inner plate.	[2013] ½" long fusion defect in plug weld unchanged (Photo 22-23). [2007] UT detected lack of fusion in plug welds: 1 of 8 in outer plate. [2009] MT detected no cracks in plug welds.
U1 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] No significant change. [2009] 1/8" high spot on inner plate. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.
U1 Sway Frame	[2013] No significant change. [2011] Lots of pigeon waste on the top and bottom horizontal gusset plate.	
U1-L1 Vertical	[2013] Pitting in splash zone Photo 24).	[2013] No significant change.
L1 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] ¼" pits [2011] $^{1}/_{8}$" bow outer plate west free edge. Pitting and corrosion on inside of panel point.	[2011/2013] No significant findings.
Floorbeam 1	[2013] Areas of flaking rust on bottom flange (10' CS3). 1' of CS4 on south end. [2011] Poured deck joint is leaking. Scattered freckled rust throughout.	
U1-U2 Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U1-U2 Upper Lateral Bracing	[2011/2013] No significant findings.	
U1-L2 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
1-2 Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
L1-L2 Bottom Chord (100 KSI steel west of field splice, 50 KSI to east)	[2013] Corrosion on splice at L2 – 2” x 2” x 3/16” pit Photos 25-26).	[2011/2013] No significant findings.
L1-L2 Stringers	[2011/2013] No significant findings.	
L1-L2 Lower Lateral Bracing	[2011/2013] No significant findings. [2009] Deep pitting in angle vertical leg @ L1-S connection.	
U2 Gusset Plates 5/8” thick (50 KSI Steel)	[2013] No significant change. [2011] External plate has $^{1}/_{8}$” bow on east edge. There is pigeon waste in the box, most of the gusset plates have corrosion staining.	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Rust staining from sway frame connection, inner plate.
U2 Sway Frame	[2013] No significant change.	
U2-L2 Vertical		[2009] Active & arrested pitting @ lower end.
L2 Gusset Plates 5/8” thick (50 KSI Steel)	[2013] Corrosion on interior of panel point (Photo 27).	[2013]Small are of pitting around 1 bolt on interior gusset plate.
Floorbeam 2	[2013] 6’ of scattered CS3 on bottom flange. ½” pit on bottom flange on north end (Photo 28).	
U2-U3 Top Chord	[2013] No significant change. 2011] No significant findings.	[2013] No significant change.
U2-U3 Upper Lateral Bracing	[2013] No significant change. [2009] Small dents & distortion of top flange of diagonal bracing member.	
U2-L3 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
2-3 Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
L2-L3 Bottom Chord (50 KSI Steel)	[2011/2013] No significant findings.	[2013] Corrosion at drain connection.
L2-L3 Stringers	[2011/2013] No significant findings.	
L2-L3 Lower Lateral Bracing	[2013] CS4 on PPL2S horizontal gusset plate.	
U3 Gusset Plates 5/8" thick (50 KSI steel)	[2013] No significant change. [2009] Small bow (< 1/8") along bottom edge of inner plate. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.
U3 Sway Frame	[2011/2013] No significant findings.	
U3-L3 Vertical	[2011/2013] No significant findings.	[2013] No significant change. [2009] Corrosion blisters, W inner flange near bottom.
L3 Gusset Plates 5/8" thick (30 KSI Steel)	[2013] No significant change. [2009] .470" UT reading, inner plate. [2011] No change.	[2011/2013] No significant findings. [2009] PB UT readings of .365" and .375" on inner plate were checked; measured .620" and .614" respectively.
Floorbeam 3	[2013] Corrosion on top flange between stringers 3-5 due to deck leakage (Photo 29). [2009] Top flange corrosion @ L3N.	
U3-U4 Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U3-U4 Upper Lateral Bracing	[2011/2013] No significant findings.	
U3-L4 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L3-L4 Lower Chord	[2013] Pack rust at horizontal gusset plate (Photo 30). [2009] Pack rust distortion, bottom plate of chord splice.	[2011/2013] No significant findings.
3-4 Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L3-L4 Stringers	[2011/2013] No significant findings.	

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
L3-L4 Lower Lateral Bracing	[2013] 2' of scattered CS4 north end.	
Deck L3-L4	[2011/2013] No significant findings.	
U4 Gusset Plates 5/8" thick (50 KSI steel)	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. External plate has $^{1}/_{8}$" box on west free edge.	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.
U4 Sway Frame	[2011/2013] No significant findings.	
U4-L4 Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings. [2009] Corrosion blisters, W inner flange and S outer flange, near bottom.
L4 Gusset Plates 5/8" thick (30 KSI S	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Floorbeam 4	[2013] 3' of scattered CS3 (Photos 31-32). [2009] Flaking rust & pack rust distortion in lower connection plate @ L4N.	
U4-U4' Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U4-U4' Upper Lateral Bracing	[2011/2013] No significant findings.	
U4-L4' Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U4'-L4 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L4-L4' Bottom Chord (50 KSI steel)	[2011/2013] No significant findings.	[2013] Corrosion at drain connections (typical).
L4-L4' Stringers	[2011/2013] No significant findings.	
L4-L4' Lower Lateral Bracing	[2011/2013] No significant findings.	
U4' Gusset	[2011/2013] No significant findings. [2011] There is pigeon waste in the box, most	[2013] No significant change. [2011] There is pigeon waste in the box,

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
Plates 5/8" thick (50 KSI steel)	of the gusset plates have corrosion staining.	most of the gusset plates have corrosion staining. Interior plate has $^{1}/_{8}$" bow on the east free edge.
U4' Sway Frame	[2011/2013] No significant findings.	
U4'-L4' Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L4' Gusset Plates 5/8" thick (30 KSI Steel)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Floorbeam 4'	[2013] Pitting and corrosion on bottom flange south end (Photo 33). 10' of staining and CS3 on north end top flange. [2009] Corrosion & pack rust distortion in lower connection plate @ L4'S.	
U4'-U3' Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings. [2009] 6 miss-drilled holes in outer vertical plate @ U3'N.
U4'-U3' Upper Lateral Bracing	[2011/2013] No significant findings.	
U3'-L4' Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
4'-3' Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L4'-L3' Bottom Chord (50 KSI steel) *Field Splice*	[2013] 1/8"-3/16" pitting @ L3' splice.	[2011/2013] No significant findings. [2009] Pack rust and fit-up distortion, top plate of chord splice.
L4'-L3' Stringers	[2011/2013] No significant findings.	
L4'-L3' Lower Lateral Bracing	[2011/2013] No significant findings.	

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
U3' Gusset Plates 5/8" thick (50 KSI steel)	[2011/2013] No significant findings. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. Exterior plate has $^{1}/_{8}$" bow on the east free edge.	[2011/2013] No significant findings. [2009] Arrested pitting on inner plate @ sway bracing connection. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.
U3' Sway Frame	[2011/2013] No significant findings.	
U3'-L3' Vertical	[2011/2013] No significant findings.	[2009] Paint failure & corrosion on inner face near bottom end.
L3' Gusset Plates 5/8" thick (30 KSI Steel)	[2013] Minor active pitting on interior of panel point.	[2013] Corrosion on interior of panel point (Photo 34).
Floorbeam 3'	[2013] No significant change. [2009] Top flange corrosion @ center deck joint.	
U3'-U2' Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U3'-U2' Upper Lateral Bracing	[2011/2013] No significant findings.	
U2'-L3' Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings. [2009] Corrosion blisters on S outer flange.
3'-2' Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L3'-L2' Bottom Chord (50 KSI steel)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L3'-L2' Stringers	[2011/2013] No significant findings.	
L3'-L2' Lower Lateral Bracing	[2011/2013] No significant findings.	
U2' Gusset Plates 5/8" thick (50 KSI steel)	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. Interior plate has $^{1}/_{8}$" bow on the west edge; the exterior has a $^{1}/_{8}$" bow on the east edge.	[2013] No significant change. [2009] Rust staining & arrested pitting @ sway frame connection. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
U2' Sway Frame	[2011/2013] No significant findings.	
U2'-L2' Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L2' Gusset Plates 5/8" thick (30 KSI Steel)	[2013] 2 sq ft of pitting and corrosion on ext gusset plate (Photo 35). [2011] UT thickness .661 west side inner plate.	[2013] No significant change. [2009] Extra drilled hole, inner plate.
Floorbeam 2'	[2013] No significant change. [2011] Transverse poured deck joint is leaking. [2009] Pitting and isolated thickness loss in bottom flange @ conn. to L2'N. UT .566" reading on 1.06" nom. flange.	
U2'-U1' Upper Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U2'-U1' Upper Lateral Bracing	[2011/2013] No significant findings.	
U1'-L2' Diagonal	[2013] No significant change. [2009] Corrosion blisters, inner flange, lower end.	[2011/2013] No significant findings.
2'-1' Horizontal Brace	[2011/2013] No significant findings.	
L2'-L1' Bottom Chord	[2011/2013] No significant findings. [2009] 3/8" pack rust in chord splice outer vertical plate.	[2011/2013] No significant findings.
L2'-L1' Stringers	[2011/2013] No significant findings.	
L2'-L1' Lower Lateral Bracing	[2011/2013] No significant findings.	
U1' Gusset Plates 5/8" thick (50 KSI steel)	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #6 (West Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
U1' Sway Frame	[2011/2013] No significant findings.	
U1'-L1' Vertical	[2011/2013] No significant findings. [2009] Active & arrested pitting at bottom.	[2011/2013] No significant findings. [2009] Corrosion blisters on both flanges near bottom. Heavy arrested pitting on web facing E.
L1' Gusset Plates 5/8" thick (30 KSI Steel)	[2013] No significant change. [2009] Active & arrested pitting, inner plate.	[2011/2013] No significant findings.
Floorbeam 1'	[2013] Staining on south end due to leaking joint. [2011] Transverse poured deck joint is leaking. 50% of the top flange has corrosion.	
U1'-L0' End Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings. [2009] Isolated corrosion blisters on inner face near bottom.
East Portal Brace	[2011/2013] No significant findings.	
L1'-L0' Bottom Chord (30 KSI Steel)	[2011/2013] No significant findings. [2009] Corrosion & pitting in bottom plate @ L1'. Deck drain brackets rusted & separating from chord. [2011] steel laminations adjacent to 3rd hole east of L1.	[2011/2013] No significant findings. [2009] PR distortion in top plate @ end of truss.
L1'-L0' Stringers	[2011/2013] No significant findings.	
L1'-L0' Lower Lateral Bracing	[2011/2013] No significant findings.	
L0' Gusset Plates	[2013] Corrosion on interior of interior gusset plate. [2009] .552" UT reading, inner plate; .538" UT reading, outer plate.	[2011/2013] No significant findings.
Floorbeam 0'	[2013] No significant findings. [2011] 50% of the top flange has corrosion.	
Pier #7 Bearings	South truss has a rocker expansion bearing [2013] Rocker bearings have minor to moderate surface corrosion (Photo 36). Bearings tipped to the east 1.50" and 3/25" SW bearing and 2" and 3" SE bearing low and high tip points respectively. [2011] The bearings are the same as 2009, 1 3/8" and 2 3/8" low and high tip points for both bearings.	North truss has a fixed bearing. [2011/2013] No significant findings. There is a swivel pin connecting the truss ends at the north bearing. [2013] No change. [2009] Heavy corrosion in swivel joint above bearings.

Bridge #9090 Inspection Field Notes

Pier #7 (Center Pier)

Pier Notes: [2011/2013] No significant findings. [2009] Minor vertical cracking, both faces.

Deck Joint #6 (Strip Seal at Pier #7): [2013] No change. [2009] Tear in gland near center.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
Pier #7 Bearings	South truss has a rocker expansion bearing. [2013] Rocker bearings have minor to moderate surface corrosion. Bearings tipped to the east but ½" less than previous years due to lower temperatures likely (52 degrees) – see bearing spreadsheet. [2011] The bearings are the same as 2009, 1 3/8" SW bearing and 2 3/8" SE bearing on the low ends tipped to the east.	There is a swivel pin connecting the truss ends at the north bearing. [2013] No change. [2009] Heavy corrosion in swivel joint above bearings. North truss has a fixed bearing. [2013] Fixed bearings have minor surface corrosion.
Floorbeam 0'	[2013] Top flange corrosion north end (Photo 37).	
L0' Gusset Plates 5/8" thick (50 KSI steel)	[2013] Outside plate leaking corrosion from top (Photo 38). [2008] Weld crack at bottom chord removed.	[2013] Leaking corrosion at pin between east and west truss (Photo 39). [2009] Isolated pitting in both plates, top edge of chord.
L0'-U1' End Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
West Portal Brace	[2013] No change. [2009] Paint failure on upper horizontal member.	
L1'-L0' Bottom Chord (30 KSI Steel)	[2011/2013] No significant findings.	[2013] Broken drain near L1'N. [2009] Corrosion & pack rust between chord & lower diagonal bracing conn. plate.
L0'-L1' Stringers	[2011/2013] No significant findings.	
L0'-L1' Lower Lateral Bracing	[2013] Bottom horizontal connection plate heavy corrosion and pack rust at L0'S (Photo 40).	
U1' Gusset Plates 5/8" thick (50 KSI steel)	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Bowing 1/8" on east edge of inner plate (fit-up).	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Paint failure with duct tape, outer plate. Rust staining on inner plate from sway frame connection.
U1' Sway Frame	[2013] No change. [2009] Paint failure on lower horizontal member @ U1'N.	
U1'-L1' Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
L1' Gusset Plates 5/8" thick (30 KSI Steel)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Floorbeam 1'	[2013] South bottom flange paint failure and water ponding (Photo 41). Bottom of web on south end has corrosion and loss up to 1/8" (Photo 42). SE face of bottom web has an undercut at the bottom flange (Photo 43). [2011] Top flange corrosion the full length. [2009] Isolated paint failure & heavy corrosion in bottom flange near L2'N.	
U1'-U2' Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U1'-U2' Upper Lateral Bracing	[2011/2013] No significant findings.	
U1'-L2' Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
1'-2' Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L1'-L2' Bottom Chord (30 KSI Steel) *Field Splice*	[2011/2013] No significant findings.	[2013] Broken drain near L1'N.
L1'-L2' Stringers	[2011/2013] No significant findings.	
L1'-L2' Lower Lateral Bracing	[2011/2013] No significant findings.	
U2' Gusset Plates 5/8" thick (50 KSI steel)	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. The exterior plate has a $^{1}/_{8}$" bow on the East edge.	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Rust staining on inner plate from sway frame connection.
U2' Sway Frame	[2011/2013] No significant findings.	
U2'-L2' Vertical	[2013] No change. [2009] Corrosion blisters, lower end.	[2011/2013] No significant findings.
L2' Gusset Plates 5/8" thick (30 KSI Steel)	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
Floorbeam 2'	[2013] Bottom flange corrosion along entire length (Photo 44). [2009/2013] Top flange corrosion, north end, due to deck leakage (Photo 45).	
U2'-U3' Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U2'-U3' Upper Lateral Bracing	[2011/2013] No significant findings.	
U2'-L3' Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
2'-3' Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L2'-L3' Bottom Chord (50 KSI steel)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L2'-L3' Stringers	[2011/2013] No significant findings.	
L2'-L3' Lower Lateral Bracing	[2011/2013] No significant findings.	
U3' Gusset Plates 5/8" thick (50 KSI steel)	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Bowing < 1/8" bottom edge of outer plate (fit-up).	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.
U3' Sway Frame	[2011/2013] No significant findings.	
U3'-L3' Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L3' Gusset Plates 5/8" thick (30 KSI Steel)	[2013] No significant change. [2009] Pitting, inner plate, UT .423" reading.	[2011/2013] No significant findings.
Floorbeam 3'	[2013] No change. [2011] The top flange has corrosion the entire length, is worse at the panel point. [2009] Corrosion & pitting in lower part of web, E side near L3'N connection.	
U3'-U4' Upper Chord	[2011/2013] No significant findings.	[2013] No change. [2011] There are miss-drilled holes at the U3' connection.
U3'-U4' Upper Lateral Bracing	[2011/2013] No significant findings.	
U3'-L4' Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
3'-4' Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L3'-L4' Bottom Chord (50 KSI steel) *Field Splice*	[2011/2013] No significant findings.	[2013] No change. [2009] Isolated corrosion blisters, inner vertical plate.
L3'-L4' Stringers	[2011/2013] No significant findings.	
L3'-L4' Lower Lateral Bracing	[2013] Support from Stringer 2 rod is broken off near FB 4'.	
U4' Gusset Plates 5/8" thick (50 KSI steel)	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Bowing < 1/8" west edge of inner plate (fit-up).	[2013] No significant change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. There is a $^{1}/_{8}$" bow on the west edge of the exterior plate.
U4' Sway Frame	[2011/2013] No significant findings.	
U4'-L4' Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L4' Gusset Plates 5/8" thick (30 KSI Steel)	[2013] No change. [2011] External gusset plate east free edge bowed $^{1}/_{16}$"	[2013] Leaking rust from corrosion spots on outside plate – inside plate similar (Photo 46). [2011] Pack rust along the horizontal gusset plate at PP.
Floorbeam 4'	[2013] No change. [2011] Pack rust along the horizontal gusset plate at PP. The top flange has flaking rust and pitting. [2009] Flaking rust & pitting in lower flange @ conn. to L4'S.	
U4'-U4 Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U4'-U4 Upper Lateral Bracing	[2011/2013] No significant findings.	
U4'-L4 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U4-L4' Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L4'-L4 Bottom Chord (50 KSI steel)	[2011/2013] Minor paint/corrosion blistering in the interior and exterior.	[2011/2013] Spotted surface corrosion throughout, isolated pitting (Photo 47).
L4'-L4 Stringers	[2011/2013] No significant findings.	

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
L4'-L4 Lower Lateral Bracing	[2011/2013] No significant findings.	
U4 Gusset Plates 5/8" thick (50 KSI steel)	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Bowing < 1/8" (fit-up) on east side of outer plate.	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.
U4 Sway Frame	[2011/2013] No significant findings.	
U4-L4 Vertical	[2013] No change. [2009] Corrosion blisters on north face @ deck level.	[2011/2013] No significant findings.
L4 Gusset Plates 5/8" thick (30 KSI Steel)	[2013] No change. [2011] Exterior gusset plate east top free edge bowed in $^{1}/_{16}$".	[2013] No change. [2009] Isolated corrosion around connecting bolts to L4-U3, outer plate. Inner plate UT .540" min.
Floorbeam 4	[2013] Corrosion and pack rust at bottom flange to connection at L4S (Photos 48). Top flange corrosion at L4N (Photo 49). [2009] Corrosion & pack rust in bottom flange @ connection to L4N.	
U4-U3 Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U4-U3 Upper Lateral Bracing	[2011/2013] No significant findings.	
U3-L4 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
4-3 Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L4-L3 Bottom Chord (50 KSI steel) *Field Splice*	[2011/2013] No significant findings.	[2011/2013] Spotted surface corrosion throughout. [2009] Isolated pitting, several locations on outer plate, 1/8" – 3/16" deep.
L4-L3 Stringers	[2011/2013] No significant findings.	
L4-L3 Lower Lateral Bracing	[2013] No change. [2009] Broken hanger bar near FB4.	
U3 Gusset Plates 5/8" thick (50 KSI steel)	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Bowing < 1/8" (fit-up) on east edge of outer plate.	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
U3 Sway Frame	[2011/2013] No significant findings.	
U3-L3 Vertical	[2011/2013] No significant findings.	[2009/2013] Corrosion blisters on web and inner flange, bottom end (Photo 50). There are miss-drilled holes at the mid lateral bracing connection.
L3 Gusset Plates 5/8" thick (30 KSI Steel)	[2011/2013] No significant findings.	[2013] No change. [2009] Corrosion & minor S/L, inner plate.
Floorbeam 3	[2013] No change. [2009] Corrosion & pack rust in bottom flange @ connection to L3N. Corrosion in top flange, N end, due to deck leakage.	
U3-U2 Top Chord	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U3-U2 Upper Lateral Bracing	[2011/2013] No significant findings.	
U2-L3 Diagonal	[2011/2013] No significant findings.	[2011/2013] No significant findings.
3-2 Horizontal Brace	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L3-L2 Bottom Chord (50 KSI Steel)	[2013] No change. [2009] Freckled rust on bottom chord plate @ L2S.	[2013] Spotted corrosion throughout (Photo 51).
L3-L2 Stringers	[2011/2013] No significant findings.	
L3-L2 Lower Lateral Bracing	[2011/2013] No significant findings.	
U2 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Cracked tack weld, inner plate. Bow in outer plate bottom edge approx. 1/8".	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining.
U2 Sway Frame	[2011/2013] No significant findings.	
U2-L2 Vertical	[2013] No change. [2009] Paint failure and corrosion on north face near bottom.	[2013] No change. [2009] Corrosion blisters on web and inner flange, bottom end.

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
L2 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] Previous loss inside of both plates painted over – leaking rust now through paint. [2009] Active & arrested pitting, inner plate. UT 0.41" min. inner plate @ top of chord.	[2011/2013] No significant findings.
Floorbeam 2	[2013] Top flange corrosion at L2N (Photo 52). [2009] Pitting & minor S/L in bottom flange @ L2S. 8' heavy corrosion on top flange.	
U2-U1 Top Chord	[2013] No change. [2009] Large area of paint failure on south face near U1S.	[2011/2013] No significant findings.
U2-U1 Upper Lateral Bracing	[2011/2013] No significant findings.	
U1-L2 Diagonal	[2013] ¼" pack rust at L2S connection. [2009] Isolated corrosion blisters on north face near bottom. Heavier corrosion on inner surface of north flange.	[2011/2013] No significant findings.
2-1 Horizontal Bracing	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L2-L1 Bottom Chord (100 KSI steel east of field splice, 50 KSI to west)	[2011/2013] No significant findings.	[2013] No change. [2009] Corrosion & pack rust on bottom of splice near PP L2N.
L2-L1 Stringers	[2011/2013] No significant findings.	
L2-L1 Lower Lateral Bracing	[2011/2013] No significant findings.	
U1 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Freckling rust on inside surface of outer plate.	[2013] No change. [2011] There is pigeon waste in the box, most of the gusset plates have corrosion staining. [2009] Rust staining on inner plate from sway frame connection.
U1 Sway Frame	[2011/2013] No significant findings.	
U1-L1 Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.
L1 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] No change. [2009] Large area of pitting (arrested) on inner plate. UT .591" min. @ top of chord.	[2013] No change. [2011] Corrosion near bolts, west side. Interior gusset plated top east free edge bowed in $^{1}/_{8}$".

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
Floorbeam 1	[2013] Top flange section loss of 1/8" on south end (Photo 53). [2009] Corrosion & pitting in web @ bottom flange, S end. Flaking rust on top flange between deck edge and truss, both ends.	
U1/2 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] No change. [2009] Bowing <1/8" on west edge of outer plate (fit-up). West stiffening angle has cracked tack welds and partial separation from plate due to pack rust (has been repaired by bolting).	[2013] No change. [2009] Bowing < 1/8" on west edge of outer plate. Corrosion and cracked tack welds on edge stiffening angles. Bolted repairs made. The interior plate is bowed $^{1}/_{8}$" in on the west edge.
U1/2-L1/2 Vertical	[2011/2013] No significant findings.	[2011/2013] No significant findings.
U1/2-L1 Diagonal	[2013] No change. [2009] Isolated surface corrosion on outer face near bottom.	
L1/2 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] No change. [2009] 1" crack in plug weld on outer plate. Crack was ground out; slag inclusions found between plate and chord. No propagation into chord.	[2013] No change. [2009] MT detected no cracks in plug welds. Stiffening angle on inner plate detached due to pack rust, E side. UT 0.42" min. Corrosion blisters & pack rust on outer plate, E side. W side stiffening angles partly detached due to pack rust. [2007] UT detected lack of fusion in plug welds: 1 of 8 in outer plate. Outer plate has surface repair weld.
U1-L0 End Diagonal	[2011/2013] No significant findings.	[2013] No significant change. [2009] Heavy corrosion & paint failure on inner flange @ PP L0N. Isolated corrosion blisters on south face near bottom.
East Portal Brace	[2011/2013] No significant findings.	
L1-L0 Bottom Chord (100 KSI Steel)	[2013] No change. [2008] Transverse crack through fillet weld between lower edge L0S outer gusset and chord, propagated into full-pen. Flange/web weld of chord. Crack was fully ground out. Erroneously labeled as PP L0'S in 2008 reports.	[2011/2013] No significant findings.
L1-L0 Stringers	[2011/2013] No significant findings.	
L1-L0 Lower Lateral Bracing	[2013] Heavily corroded bracing on south end and cracked drain bracket (Photo 54). Hanger broke off near FB1S (Photo 55). North inside brace corrosion.	

Bridge #9090 Inspection Field Notes

Br. #9090 Span #7 (East Truss Span)		
Location	**South Truss (Upstream)**	**North Truss (Downstream)**
L0 Gusset Plates 5/8" thick (50 KSI Steel)	[2013] Top plate still warped. Outside plate leaking rust from top. [2009] Top cover plate warped 1/2"-3/4" from pack rust.	[2013] Top plate between inner and outer gusset is warped downward ½". [2009] Heavy corrosion on inner plate above chord line. UT .509" min.
Floorbeam 0	[2013] Heavy leaking rust from top flange at Pier 8.	
Pier #8 Truss Expansion Rocker Bearings	There are no records of this bearing ever being reset - take measurements to compare current position of masonry plate with pier centerline? [2013] Bearing tipped east 1 7/8" and 2 7/8" low and high tip points. [2011] Same measurements as in 2009. [2010] Bearing tipped east 1.50" and 3.25" low and high tip points. [2009] Bearing tipped E, 1.75"/3.00" tip hts.	There are no records of this bearing ever being reset - take measurements to compare current position of masonry plate with pier centerline? [2013] Bearing tipped east 1 ½ " and 3 5/14" low and high tip points. [2011] Same measurements as in 2009. [2010] Bearing tipped east 1.75" and 3" low and high tip heights. [2009] Bearing tipped E, 1.50"/3.25" tip hts.

Pier #8 (East end of East Truss Span)

Pier Notes: [2013] Previous graffiti painted over, but new graffiti is present – west face.
[2009] Minor cracking & staining W face.

Bridge #9090 Inspection Field Notes

Br #9090 East Approach Spans #8-13 (Minnesota Side)			
Location	**Detail**	**South (Eastbound) Side**	**North (Westbound) Side**
Span #8	Deck Joint #7 (Strip Seal at Pier #8)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Swivel Hinge (Joint #7)	[2013] Underdeck cracking, spalling, and efflorescence	[2013] Underdeck cracking and efflorescence.
	Deck & General	[2011/2013] No significant findings.	[2013] Overhang delamination typical in all spans (Photo 56).
	Rolled Steel Beams (64 ft. span)	[2011/2013] No significant findings.	[2013] Beam 4 leaking rust from top flange at Pier 8 (Photo 57).
	Deck Joint #8 (Finger Joint)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Expansion Hinge (Joint #8)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Bent #9	Footings	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Pinned Columns	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Span #9	Deck & General	[2013] Underdeck cracking, spalling, and efflorescence.	[2013] Overhang delamination typical in all spans (Photo 58).
	Rolled Steel Beams (64 ft. span)	[2011/2013] No significant findings.	[2013] Beams 2 & 6 bottom flange flaking rust at pin and hanger.
	Pin & Hanger (Swivel) Joint #9	[2011/2013] No significant findings.	[2013] Girder 3 has a note on the bottom pin from 5-2-01 of a crack…just a tack weld crack on the nut.
	Deck Joint #9 (Strip Seal)	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br #9090 East Approach Spans #8-13 (Minnesota Side)			
Location	**Detail**	**South (Eastbound) Side**	**North (Westbound) Side**
Bent #10	Footings	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Pinned Columns	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Span #10	Deck & General	[2013] Underdeck cracking, spalling, and efflorescence. Saturation also present (Photo 59).	[2013] Overhang delamination typical in all spans.
	Rolled Steel Beams (64 ft. span)	[2011/2013] No significant findings.	[2013] No change. [2009] Surface corrosion, bottom flange B1.
	Pin & Hanger (Swivel) Joint #10	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Deck Joint #10 (Strip Seal)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Bent #11	Footings	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Pinned Columns	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Span #11	Deck & General	[2013] Underdeck cracking, spalling, and efflorescence.	[2013] Overhang delamination typical in all spans.
	Rolled Steel Beams (64 ft. span)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Pin & Hanger (Swivel) Joint #11	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Deck Joint #11 (Strip Seal)	[2011/2013] No significant findings.	[2011/2013] No significant findings.

Bridge #9090 Inspection Field Notes

Br #9090 East Approach Spans #8-13 (Minnesota Side)			
Location	**Detail**	**South (Eastbound) Side**	**North (Westbound) Side**
Bent #12	Footings	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Pinned Columns	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Span #12	Deck & General	[2013] Underdeck cracking, spalling, and efflorescence. [2003] Roadway removed and replaced with Greenway Trail.	[2013] Overhang delamination typical in all spans – exposed rebar. [2003] Roadway removed and replaced with Greenway Trail.
	Rolled Steel Beams (64 ft. span)	[2009/2013] No change. [2003] Bottom flange impact damage, B7 & B8. MT detected no cracks.	[2011/2013] No significant findings.
	Pin & Hanger (Swivel) Joint #12	[2011/2013] No significant findings.	[2013] Beam 3 offset ¾”.
	Deck Joint #12 (Strip Seal)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Bent #12	Footings	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Pinned Columns	[2011/2013] No significant findings.	[2011/2013] No significant findings.
Span #13	Deck & General	[2013] Underdeck cracking, spalling, and efflorescence.	[2013] Overhang delamination typical in all spans – exposed rebar.
	Rolled Steel Beams (56 ft. span)	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Span #13 (6 ft. slab at abutment)	Poured joints and both ends of slab.	Poured joints and both ends of slab.

Bridge #9090 Inspection Field Notes

Br #9090 East Approach Spans #8-13 (Minnesota Side)			
Location	**Detail**	**South (Eastbound) Side**	**North (Westbound) Side**
East Abutment	Slope Protection	[2013] Graffiti painted over. [2009/2013] East slope separating from abutment at north side 2”-4”, center panels heaving.	
	Fixed Bearings	[2013] Center fixed bearing corrosion.	[2013] Center fixed bearing corrosion. North fixed bearing heavy leaking corrosion at beam end (Photo 60).
	Seat	[2011/2013] No significant findings.	[2011/2013] No significant findings.
	Wingwalls	[2011/2013] No significant findings.	[2011/2013] No significant findings.

General Notes

Deck:
[2013] 8” spall 15’ west of strip seal 9 east bound lane. Open cracks and scattered areas of delamination between strip seal 7 & 8. 47” x 28” patch east of strip seal 7 at fog line. This section of deck has the worst area of open cracks. 93” x 69” patch east of west finger joint. East end of patch is deteriorating.
[2009] Spalling w/exposed rebar in overhang, both sides, both approaches. Lesser spalling in truss spans. Heavy spalling over floorbeams @ most panel points.

Median:
[2013] Cracked, spalled, delaminated and deteriorated concrete along sides. 25’ of spalled concrete along median joint on east end of bridge (Photo 61).

Curbs:
[2013] There are minor spalls at several of the poured joints (Photo 62). There is a 15’ on the north curb (Photo 63).

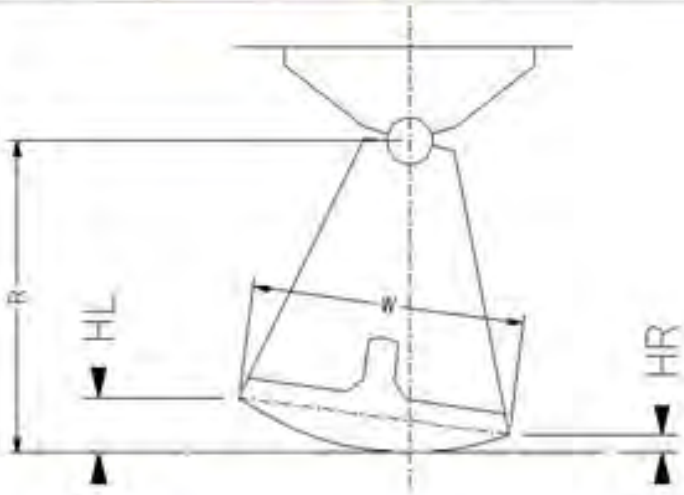
Railing:
[2013] The metal railings are in CS2. There are cracks throughout the concrete railing face (Photo 64). There is a broken rail post on the north rail on the east approach (Photo 65).

Signing:
[2013] No significant findings.

Bridge #9090 Inspection Field Notes

Measurement of Rocker Bearing Tipping Angles

Bridge	9090	**Measured By**	JLZ
Inspection Date	6/3/2013	**Calculations By**	JMF



Angle (q) = ARCSIN [(HL - HR) / W]
Displacement = (p * q * R) / 180

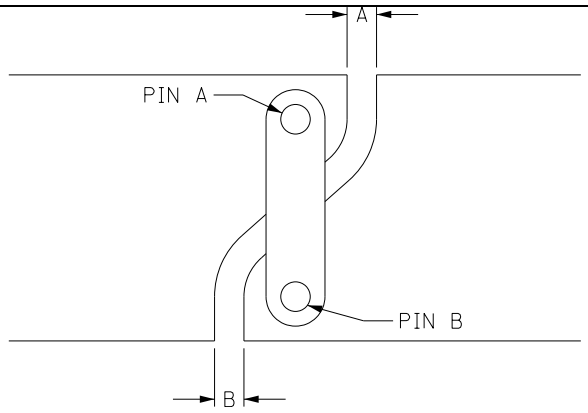
Truss	Sub.	Bearing	W (in)	R (in)	HL (in)	HR (in)	Angle	Displace-	Tip Dir.	Temp
2010	Pier 6	NW	24.00	32.00	3.75	1.13	6.3	3.5	W	83
2011	Pier 6	NW	24.00	32.00	4.75	1.00	9.0	5.0	W	70
2013	Pier 6	NW	24.00	32.00						
2010	Pier 6	SW	24.00	32.00	4.25	0.88	8.1	4.5	W	75
2011	Pier 6	SW	24.00	32.00	4.50	0.75	9.0	5.0	W	70
2013	Pier 6	SW	24.00	32.00						
2010	Pier 7	SW	24.00	32.00	2.38	1.38	2.4	1.3	E	83
2011	Pier 7	SW	24.00	32.00	2.38	1.38	2.4	1.3	E	70
2013	Pier 7	SW	24.00	32.00	3.25	1.50	4.2	2.3	E	52
2010	Pier 7	SE	24.00	32.00	2.38	1.38	2.4	1.3	E	75
2011	Pier 7	SE	24.00	32.00	2.38	1.38	2.4	1.3	E	70
2013	Pier 7	SE	24.00	32.00	3.00	2.00	2.4	1.3	E	52
2010	Pier 8	NE	24.00	32.00	3.00	1.75	3.0	1.7	E	85
2011	Pier 8	NE	24.00	32.00	3.25	1.50	4.2	2.3	E	60
2013	Pier 8	NE	24.00	32.00	3.36	1.50	4.4	2.5	E	52
2010	Pier 8	SE	24.00	32.00	3.25	1.50	4.2	2.3	E	80
2011	Pier 8	SE	24.00	32.00	3.00	1.75	3.0	1.7	E	60
2013	Pier 8	SE	24.00	32.00	2.88	1.88	2.4	1.3	E	52

Bridge #9090 Inspection Field Notes

Bent Angle Measurements							
Bent	North Bearing		Center Bearing		South Bearing		Comments
	2011	2013	2011	2013	2011	2013	
2	0°		0°		0°		
3	0°		0°		0°		
4	0°		1°W		1°W		
5	0°		0°		0°		
9	1°E	1°E	0°	1°W	0°	1°W	[2013] 61°F
10	1°E	0°	1°E	1°E	1°E	2°W	[2013] 61°F
11	2°W	0°	1.5°W	1°W	1.5°W	2°W	[2013] 61°F
12	0°	1°W	0°	1°W	0°	1°W	[2013] 61°F
13	2°W	1.5°W	2.5°W	1.5°W	3°W	2°W	[2013] 61°F

Bridge #9090 Inspection Field Notes

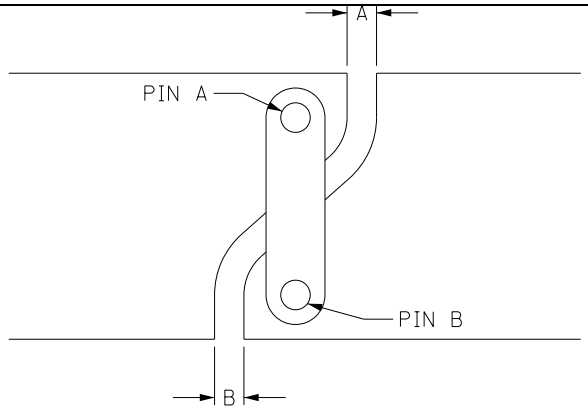
Expansion Joint Gap Measurements



Span		Girder 1		Girder 2		Girder 3		Girder 4		UT Result
		2011	2013	2011	2013	2011	2013	2011	2013	
2	A	5		5		5		5		OK
	B	2 1/2		2 1/2		2 ½		2 ¾		OK
3	A	7		6 ¾		6 ¾		6 ¾		OK
	B	5		4 5/8		4 ¼		4 5/8		OK
4	A	6 5/8		6 ¼		6 ¾		6 ¾		OK
	B	4 ½		4 ½		4 ¾		4 ¾		OK
5W	A									
	B									
8E	A									
	B									
9	A	4 ½	5 ¼	4 ½	5 ¼	5 ¾	5 ¼	5 ¼	5	OK
	B	2 ¼	3	2 5/8	2 3/4	3	2 5/8	3	2 3/4	OK
10	A	4 ½	5	5 3/8	5	5 ¼	5	5 ½	5	OK
	B	2	2 7/8	3	2 7/8	2 5/8	2 7/8	3 1/8	3 7/8	OK
11	A	5 3/8	5 1/8	5 1/8	5	4 7/8	5 ¼	4 ¾	5 ¼	OK
	B	3 1/16	2 7/8	3 1/16	2 7/8	3	3 1/8	2 ½	2 3/4	OK
12	A	2 3/8	2 5/8	4 7/16	5 1/8	5 ¼	4 7/8	5 ½	4 ¾	OK
	B	4 ¾	4 ¾	2 7/16	2 7/8	2 ½	2 7/8	3	2 3/4	OK

Bridge #9090 Inspection Field Notes

Expansion Joint Gap Measurements



Span	Girder 5		Girder 6		Girder 7		Girder 8		UT Result
	2011	2013	2011	2013	2011	2013	2011	2013	
2 A	5 ¼		5		5 ¼		5 ½		OK
B	2 ¾		2 ½		2 ½		2 ¾		OK
3 A	6 ½		6 ¾		6 ¾		7 1/8		OK
B	4 ½		4 ¾		4 7/8		4 7/8		OK
4 A	6 ½		6 ½		6 ¾		6 ½		OK
B	4 ½		4 ½		4 ¾		4 ½		OK
5W A							7 ½		OK
B							7 ½		OK
8E A									
B									
9 A	5 ¼	5 ¼	5 1/8	5 3/8	5 ½	4 ½	5 ¼	4 ¼	OK
B	2 ½	3 1/8	2 5/8	3 ¼	3	2 ½	2 ¾	2 1/8	OK
10 A	5	5 ½	5	5 1/8	5	4 ¾	5	4 ½	OK
B	2 7/8	3 1/4	2 7/8	2 5/8	2 7/8	2 ¾	2 5/8	2 1/8	OK
11 A	5	4 7/8	5 ¼	4 7/8	5	5	5 ¼	5	OK
B	2 ¾	2 5/8	3	3	2 ¾	3 1/8	2 7/8	2 ¾	OK
12 A	4 ¾	5 ½	4 5/16	5 ½	2 9/16	4 ¾	4 ¾	4 ¾	OK
B	2 7/16	3 3/8	2 7/8	2 ¾	5	2 ¼	2 9/16	2 ½	OK

Pictures

Photo 1 - West Abutment South End Spall

Photo 2 - West Abutment Parapet Delamination

Pictures

Photo 3 - West Abutment Spall at Beam 2

Photo 4 - West Abutment Spall at Beam 3

Photo 5 - West Abutment Spall at Beam 4

Photo 6 - West Abutment Spall at Beam 6

Photo 7 - West Abutment Spall at Beam 7

Photo 8 - Crack in West Slope

Photo 9 - Pier 5 Exposed Piling

Photo 10 - Span 3-4 EB Rotation Hinge CS3

Photo 11 - Pier 6 Twisted

Photo 12 - Pier 6 Twisted

Photo 13 - Pier 6 Twisted

Photo 14 - Pier 6 Cap Cracks East Face Center

Photo 15 - Pier 6 Cracks in Pier Wall and Cap

Photo 16 - Pier 6 Cracks in Pier Wall and Cap

Photo 17 - Pier 6 Cracks in Pier Wall and Cap

Photo 18 - Pier 6 Cracks in Pier Wall and Cap

Photo 19 - Pier 6 Cracks in Pier Wall and Cap

Photo 20 - Pier 6 Cracks in Pier Wall and Cap

Photo 21 - West Truss L0S Corrosion on Interior of Panel Point

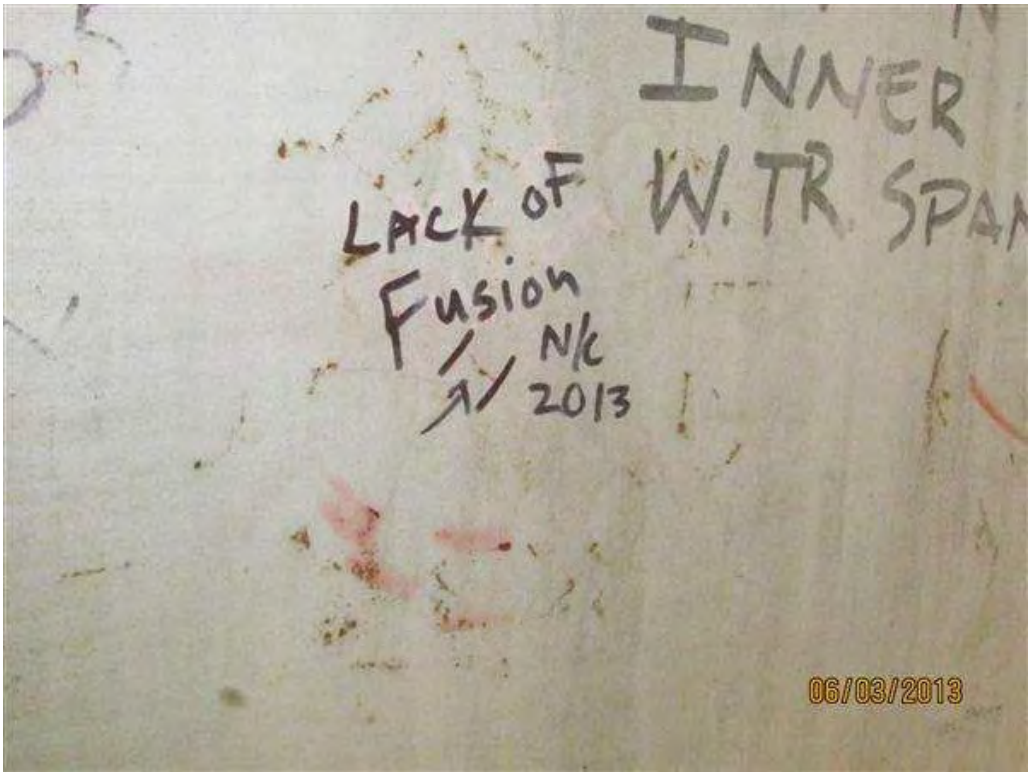


Photo 22 - L1/2N Fusion Defect No Change

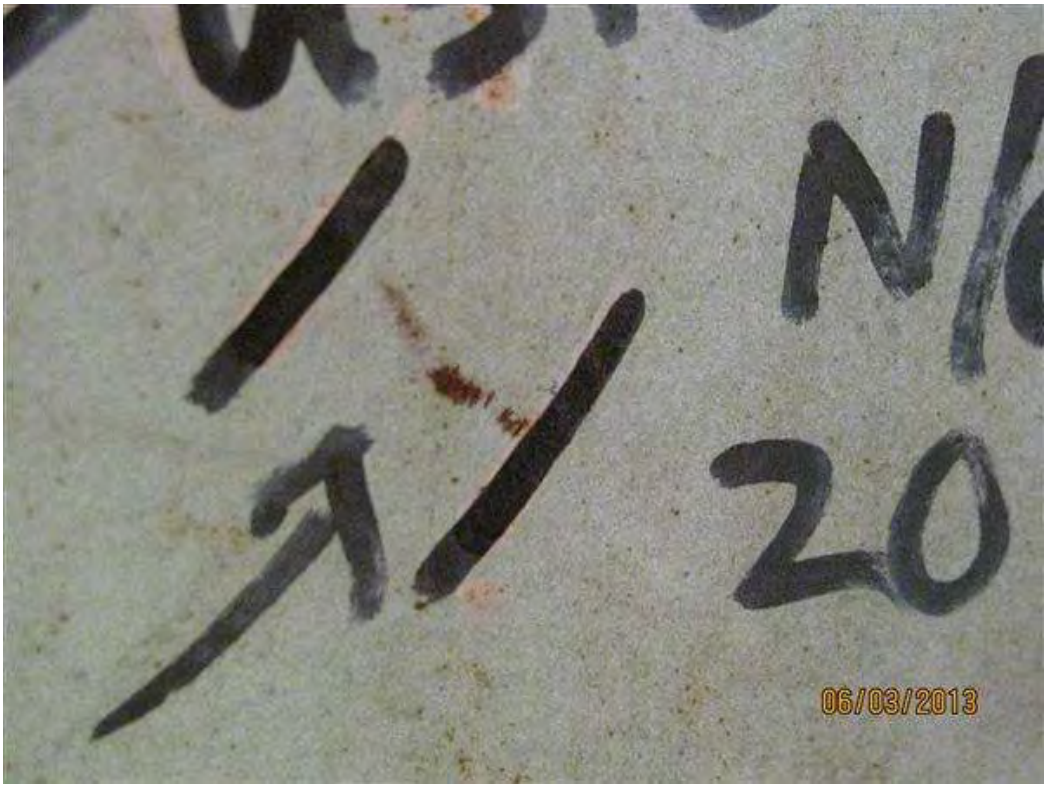


Photo 23 - L1/2N Fusion Defect No Change

Photo 24 - West Truss U1-L1S Typical Corrosion in Splash Zone

Photo 25 - West Truss L1-L2S .18 in Pit at Splice

Photo 26 - West Truss L1-L2S .18 in Pit at Splice

Photo 27 - West Truss L2S Corrosion on Interior of Panel Point

Photo 28 - West Truss FB2 - .5 in Pit on Bottom Flange North End

Photo 29 - West Truss FB3 Corrosion on Top Flange Due to Deck Leakage

Photo 30 - West Truss L3-L4N at L4 Pack Rust at Horizontal Gusset Plate

Photo 31 - West Truss FB4 CS4 at Horizontal Gusst Plate North End

Photo 32 - West Truss FB4 Corrosion on North End

Photo 33 - West Truss FB4' Pitting on South End Bottom Flange

Photo 34 - West Truss L3'N Typical Corrion in Panel Point

Photo 35 - West Truss L2'S Corrosion on Exterior Gusset Plate

Photo 36 - Pier 7 South Rocker Bearings Looking South

Photo 37 - East Truss FB0'N Top Flange Corrosion

Photo 38 - East Truss L0'S Bottom Horizontal Plate Corrosion

Photo 39 - East Truss L0'N at Pier Looking South - Corrosion

Photo 40 - East Truss L0'S Bottom Horizontal Plate Corrosion

Photo 41 - East Truss FB1'S Bottom Flange Water Ponding

Photo 42 - East Truss FB1'S Corrosion and Loss at Bottom Web One Eighth

Photo 43 - East Truss FB1'S Bottom Flange Paint Failure

Photo 44 - East Truss L2'N Floorbeam Bottom Flange Corrosion Looking South

Photo 45 - East Truss L2'N Floorbeam Top Flange Corrosion West Face

Photo 46 - East Truss L4'-L4S Bottom Chord Inside Surface Corrosion Spots

Photo 47 - East Truss L4-L4'N Bottom Chord Sporadic Corrosion

Photo 48 - East Truss FB4S Bottom Gusset Corrosion and Pack Rust

Photo 49 - East Truss L4N Floorbeam Top Flange Corrosion

Photo 50 - East Truss L3-U3N East Face Corrosion

Photo 51 - East Truss L2-L4N Bottom Chord Corrosion

Photo 52 - East Truss L2N Floorbeam Top Flange Corrosion

Photo 53 - East Truss FB1S Top Flange .12 in. Section Loss

Photo 54 - East Truss L0-L1S Drain Bracket Cracked and Corroded Sway Bracing

Photo 55 - East Truss L0-L1S Sway Brace Hanger Broken Off near FB1S

Photo 56 - South Overhang Underdeck Delamination and Spall Looking Towards Pier 8

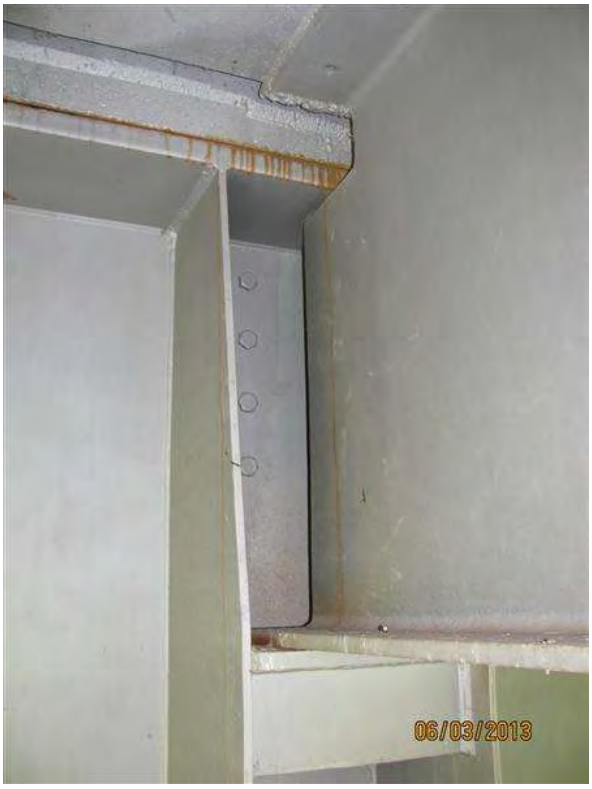


Photo 57 - Floorbeam Over Pier 8 East Face Girder 4 Leaking Rust

Photo 58 - Northeast Approach Overhang Delaminations Looking West

Photo 59 - Span 10 Underdeck Saturation and Efflorescence

Photo 60 - East Abutment North Girder Bearing Corrosion

Photo 61 - Median Spall Typical

Photo 62 - North Side Curb Spall at Joint Typical

Photo 63 - North Side Curb Spall Typical

Photo 64 - Typical Rail Cracking

Photo 65 - North Rail on East Approach

Appendix A: SI&A

MnDOT Structure Inventory Report

Bridge ID: 9090 US 2 over RED RIVER Date: 12/16/2013

GENERAL

Agency Br. No.
District District 2
Maint. Area 2B Crew
County 060 - Polk
City East Grand Forks
Township
Desc. Loc. AT N DAKOTA STATE LINE
Sect., Twp., Range 2 - 151N - 50W
Latitude Deg 47 Min 55 Sec 59.78
Longitude Deg 97 Min 2 Sec 13.84
Custodian 01 - State Highway Agency
Owner 01 - State Highway Agency
BMU Agreement
Year Built 1963
MN Year Reconstructed
FHWA Year Reconstructed
MN Temporary Status
Bridge Plan Location 1 - CENTRAL
Date Opened to Traffic 1/1/1963
On-Off System 1 - ON
Legislative District 01B

STRUCTURE

Service On 5 - Highway-pedestrian
Service Under 5 - Waterway
Main Span Type
3 - Steel 03 - High Truss
Main Span Detail A - PARKER
Appr. Span Type
3 - Steel 01 - Beam Span
Appr. Span Detail
Skew 0
Culvert Type
Barrel Length ft.
Cantilever ID S - Pin and Hanger

NUMBER OF SPANS

MAIN: 2 APPR: 11 TOTAL: 13
Main Span Length 279.0 ft.
Structure Length 1261.0 ft.
Deck Width (Out-to-Out) 65.0 ft.
Deck Material 1 - Concrete Cast-in-Place
Wear Surf Type 4 - Low Slump Concrete
Wear Surf Install Year 1984
Wear Course/Fill Depth 0.17 ft.
Deck Membrane 0 - None
Deck Rebars N - Not Applicable (no deck)
Deck Rebars Install Year
Structure Area (Out-to-Out) 81965 sq. ft.
Roadway Area (Curb-to-Curb) 70611 sq. ft.
Sidewalk Width Lt 2.50 ft. Rt 2.50 ft.
Curb Height Lt 0.75 ft. Rt 0.75 ft.
Rail Type Lt 19 Rt 19

ROADWAY

Bridge Match ID (TIS) 1
Roadway O/U Key Route On Structure
Route Sys 02 - USTH Number 2
Roadway Name or Description
US 2
Level of Service 1 - MAINLINE
Roadway Type 2 - 2-way traffic
Control Section (TH Only) 6018
Reference Point 000+00.000
Detour Length 4.0 mi
Lanes On 4 Under 0
ADT 20740 Year 2008
HCADT 1867 ADTT 9 %
Functional Class 14 - Urban - Other Principal Art

RDWY DIMENSIONS

If Divided	NB-EB		SB-WB	
Roadway Width	28.00	ft.	28.00	ft.
Vertical Clearance	19.8	ft.	19.8	ft.
Max. Vert. Clear.	19.8	ft.	19.8	ft.
Horizontal Clear.	27.9	ft.	27.9	ft.
Lateral Clearance		ft.		ft.

Appr. Surface Width 60.0 ft.
Bridge Roadway Width 56.0 ft.
Median Width On Bridge 4.00 ft.

MISC. BRIDGE DATA

Structure Flared 0 - No flare
Parallel Structure N - No parallel structure
Field Conn. ID 4 - Bolted
Abutment Foundation 1 - CONC
(Material/Type) 3 - FTG PILE
Pier Foundation 3 - STEEL
(Material/Type) 3 - FTG PILE
Historic Status 5 - Not eligible

PAINT

Year Painted 1996
Unsound Paint % 5
Painted Area 183102 sq. ft.
Primer Type 0 - Other - non 3309
Finish Type M - Urethane

BRIDGE SIGNS

Posted Load 0 - Not Required
Traffic 0 - Not Required
Horizontal 1 - Object Markers
Vertical 0 - Not Required

INSPECTION

Userkey 18
Unofficial Structurally Deficient Y
Unofficial Functionally Obsolete N
Unofficial Sufficiency Rating 49.0
Routine Inspection Date 06/06/2013
Routine Inspection Frequency 12
Inspector Name Bridge Office FC Unit
Status A - Open

NBI CONDITION RATINGS

Deck 5 - Fair Condition
Unsound Deck % 10
Superstructure 6 - Satisfactory Condition
Substructure 4 - Poor Condition
Channel 6 - Bank slump; minor damage
Culvert N - Not Applicable

NBI APPRAISAL RATINGS

Structure Evaluation 4
Deck Geometry 5
Underclearances N
Water Adequacy 6 - Occasional Overtopping
Approach Alignment 7 - Better than present minir

SAFETY FEATURES

Bridge Railing 0 - SUBSTANDARD
GR Transition N - NOT REQUIRED
Appr. Guardrail N - NOT REQUIRED
GR Termini N - NOT REQUIRED

IN DEPTH INSP.

	Y/N	Freq	Date
Frac. Critical	Y	24 mo.	06/06/2013
Underwater	Y	60 mo.	08/29/2012
Pinned Asbly.	Y	48 mo.	06/09/2011
Spec. Feat.			

WATERWAY

Drainage Area (sq. mi.)
Waterway Opening 29000 sq. ft.
Navigation Control 0 - No nav. control on waterw
Pier Protection
Nav. Clr. (ft.) Vert. ft. Horiz. ft.
Nav. Vert. Lift Bridge Clear. (ft.)
MN Scour Code L - STBL - LOW F Year 1997

CAPACITY RATINGS

Design Load 5 - HS 20
Operating Rating 1 - LF (LF) HS 26.8
Inventory Rating 1 - LF (LF) HS 16.0
Posting VEH: SEMI: DBL:
Rating Date 7/22/2008

MnDOT Permit Codes

A: 1 - No Restriction
B: 1 - No Restriction
C: 2 - Straddle 2 Lanes

Appendix B: 7 Day FC Report

7 Day Fracture Critical Report (Report Date: 06/18/2013)

Disclaimer: The condition ratings in this report are only suggested. It is the responsibility of the Bridge Owner to approve inspection data in SIMS.

Bridge #	9090
Facility Carried:	US 2
Facility Intersected:	RED RIVER
Bridge Owner:	State Highway Agency
Inspection Date(s):	06/03/2013 - 06/06/2013
Primary Inspector:	Nelson, Bill
Other Inspector(s):	Evens, Eric; Klein, Gregory; Potter, Farrell; Rand, Ken; Walker, Scott; Wilson, Pete; Zink, Jennifer
Method of Access:	Manlift, A-62, A-62
Traffic Control:	Lane closure provided by District 2.
Scope of Inspection:	Routine and Fracture Critical Inspection

Critical Structural Deficiencies	(Yes/No)	No
New Load Rating Recommended	(Yes/No)	No
Traffic Safety Hazard	(Yes/No)	No
Structural Analysis Recommended	(Yes/No)	No

NBI Condition Ratings

Item	Current	Suggested	Comments
Deck	7	5	2013 - Lower to 5. Scattered areas of saturation and efflorescence, especially in east bound lane of east truss span (Photos 1-5). Spalling and exposed rebar on the fascias throughout. Deck should be analyzed by the District via GPR to determine extent of deterioration. Rating lowered from 7 to 6 in 1993. Overlay has isolated concrete patches, with one section of delamination in Span #5 (eastbound). Underside of deck overhang has spalling (exposed rebar), particularly on the east approach spans.
Superstructure	6	6	2013 - No change. Scattered areas of active corrosion and old pitting (painted over) on the bottom chord, bottom chord gusset plates and truss verticals & diagonals located near the deck level. Any section loss is minor.
Substructure	5	4	2013 - Lower to 4 due to continued movement and cracking of Pier 6 and spalling behind the fixed bearings on the West Abutment (it appears that the bearings are being pulled out (Photos 6-14).
Channel	6	6	2013 - No change (recommended in the 2012 Underwater Report).

Suggested Element Condition Rating Changes in RED

#	Element Description	Quantity	Element Condition Ratings				
			1	2	3	4	5
022	Low Slump O/L (Concrete Deck with Uncoated Rebar)	81,185 SF	0	81,185	0	0	0
048	Low Slump O/L (Concrete Slab with Uncoated R	780 SF	0	780	0	0	0
107	Painted Steel Girder or Beam	5,600 LF	3,320	2,000	280	0	0
113	Painted Steel Stringer	4,464 LF	2,298	2,166	0	0	0
121	Painted Steel Through Truss - Bottom Chord	1,116 LF	0	780	220	116	0
126	Painted Steel Through Truss - Upper Members	1,116 LF	540	516	60	0	0
152	Painted Steel Floorbeam	1,380 LF	0	620	620	140	0
161	Pin & Hanger (or Hinge Pin) Assembly (Painted)	56 EA	0	56	0	0	0
202	Painted Steel Column	72 EA	36	18	18	0	0

Suggested Element Condition Rating Changes in RED							
#	Element Description	Quantity	Element Condition Ratings				
			1	2	3	4	5
210	Reinforced Concrete Pier Wall	210 LF	140	0	70	0	
215	Reinforced Concrete Abutment	140 LF	105	35	0	0	
220	Reinforced Concrete Footing	18 EA	18	0	0	0	
234	Reinforced Concrete Pier Cap	210 LF	140	0	70	0	
300	Strip Seal Deck Joint	700 LF	689	0	11		
301	Poured Deck Joint	1,400 LF	280	1,120	0		
303	Assembly Deck Joint (with or without seal)	1,260 LF	0	1,245	15		
311	Expansion Bearing	6 EA	0	4	2		
313	Fixed Bearing	18 EA	10	8	0		
321	Concrete Approach Slab-Concrete Wearing Surface	2 EA	1	1	0	0	
333	Masonry, Other or Combination Material Railing	8,274 LF	6,798	1,470	6		
357	Pack Rust Smart Flag	1 EA	0	1	0	0	
358	Concrete Deck Cracking Smart Flag	1 EA	0	0	1	0	
359	Underside of Concrete Deck Smart Flag	1 EA	0	0	0	1	0
360	Substructure Settlement & Movement Smart Flag	1 EA	0	0	1		
361	Scour Smart Flag	1 EA	0	1	0		
362	Traffic Impact Smart Flag	1 EA	0	1	0		
363	Section Loss Smart Flag	1 EA	0	1	0	0	
373	Steel Hinge Assembly	32 EA	0	32	0	0	0
380	Secondary Structural Elements	1 EA	0	1	0	0	
387	Reinforced Concrete Wingwall	4 EA	3	1	0	0	
411	Open Finger Deck Joint	140 LF	140	0	0		
422	Painted Beam Ends	14 EA	10	2	0	2	0
423	Gusset Plate Truss Connection - Painted Steel	80 EA	0	36	44	0	0
964	Critical Finding Smart Flag	1 EA	1	0			
966	Fracture Critical Smart Flag	1 EA	1	0	0		
967	Gusset Plate Distortion Smart Flag	1 EA	0	1	0	0	
981	Signing	1 EA	0	0	1	0	0
984	Deck & Approach Drainage	1 EA	0	1	0		
985	Slopes & Slope Protection	1 EA	0	1	0		
986	Curb & Sidewalk	1 EA	0	1	0		
988	Miscellaneous Items	1 EA	0	1	0		

Element Rating Notes:

ELEMENT #022: Cracks in overlay at west finger jt. Small spall in median concrete on bottom side @ E. abut. br/wall w/rbar exposed. Major deterioration of overhangs w/rust staining on approach spans underside. Concrete spalling at the ends of many floorbeams.There appears to be a dip at the joint above the second bent from the west.**Repaired approx 12 sq. of delam concrete at west end EBL. on 8/08. DSH
Epoxied deck cracks GK 5/2012
2013: No significant change in condition. Quantity reduced to reflect area of 6' deck slabs on each end of bridge.

ELEMENT #048: This element reflects the 6' approach spans between the abutments and the bearings at each end of the bridge; both in CS2 due to spall and cracks on the sides and undersides of the slabs.

ELEMENT #107: Rust Beginning to Form. Several of the centerline rotational pins appear to be frozen due to corrosion. There is a bend in the bottom flange of the south fascia beam, span 12, no cracks present.
2003 FC Inspection:Several of the rotational pin assemblies appear to be frozen due to corrosion. Beams 1 and 2 (from the south) in span 2 have impact damage due to a high load hit. Magnetic Particle examination of those areas revealed no cracks.
2007 FC Inspection:The approach span beams are in relatively good condition, with the exception of some

advanced corrosion of the bottom flange on some of the fascia beams.
See Notes and Pictures on file in the Engineers office. Some paink chalking and corrosion present. Joe F 6/25/09
Facia bottom flange continues to rust in approach spans at various locations. GK 5/2012
2013: No significant change.
ELEMENT #113: Rust Beginning to Form.
2003 FC Inspection:There is a square cope detail at the top of the web on all of the stringers, which is fatigue prone and should be monitored for cracking. There is scattered rust beginning to form on the stringers, but no areas of significant section loss.
2007 FC Inspection:There are isolated areas of paint failure and surface rust, but no pitting or section loss. The square cope detail shouldl still be monitored for cracks during in-depth inspections.
See Pictures and Notes on file in the engineers office.
2009 FC report, recommended changing the lin. ft. from 97 to 4464 , and quant 1 & 2 respectivly. GK 4/10
2013: No significant change.
ELEMENT #121: Pack rust is forming at the gusset plates and batten plates on the bottom of both chords. The bolting plate @ the bottom of the 4th vert chord in the NW cor of the truss is twisted down approx 1in. where cross brace ties in. The cross brace coming into this plate is also twisted. 3 cross bracing hanger rods are bent. The X-bracing on the SW cor of truss is also bent. Bott chord boxes are infested with pigeons. Several interior welds of the lower chord box members were inspected after blasting & cleaning-no defects were noted. There are some nuts missing on diag bracing hanger rods. Two broken hanger rods 1 at MN (east) Pier, 1 at ND (west) pier.
2003 FC Inspection:There are scattered areas of surface pitting on the chords. There is minor section loss with moderate pitting at the bottom panel point connections on gusset plates and truss members (<5 %) Pack rust is forming at the gusset plates and batten plates on teh bottom of the chords.
There is some rust and some minor section loss beginning to form, elpecially along the curb line and at the bottom chord connections on the verticals and diagonals. The total area affected is less than 5%.
2007 FC Inspection:There are scattered areas of surface pitting on the chords. Magnetic particle was performed on the gusset plate to lower chord welds. A crack approx. 2" long was found on the at the L0 south side of the west truss.
Consulted w/CO & was instructed to grind out crack and prime & apply Dow 888 to prevent rusting. Re-inspected in December 07 & again in March of 08. DSH.***Ground out crack was re-inspected on 11/19/07 by DSH and found to have no further propagation. No propagation on may 13th 2008.
See Pictures and Notes on the FC reports in engineers office.
2009 FC inspection: Pack rust, corrosion blisters, flaking rust and minor section loss on bottom chords and panel points. Joe F 6/25/2009
East truss, 3rd bay, no. side 1 wind bracing anchor rod snapped off.GK 5/18/10
At center pier rust and corrosion continues at diag. bracing connections. GK 5/2012.
2013: No significant change. Magnetic particle examination of the plug welds and welds on the T1 steel from L0-L1 revealed no new discontinuities.
ELEMENT #126: Rust Beginning to Form. Minimal sect loss on top chord verticals & diagonals. Pigeons nesting in upper chord box members. There are broken welds on the angle stiffeners on top of the inside gusset plates (deck height) @ the 1st panel points W of the NE & SE end posts.**Bridge Maint. added angle iron stiffeners to all (8) Lo locations to the un-supported lengths, also the in-place stiffeners were repaired / installed where needed on 5/20/2010, (4) bolts that were missing on the East truss -no. side @ Lo location were replaced. DSH.** Bridge Maint. incorporated debris drains @ all (8) Lo locations in order to adequately flush areas that were not able to clean; this was done on 6/10/2010. DSH
2003 FC Inspection:The top chords are in good condition with only minor scattered areas of isolated paint loss and surface rust (<5 %). There is some rust and some minor section loss beginning to form, especially along the curb line and at the bottom chord connections on the verticals and diagonals. The total area affected is less than 5%.
2007 FC Inspection:No Significant shange from previous inspection. Verticals and Diagonals No significant changes.
2009 FC inspection: Corrosion blisters on top chords and panel points Joe F 6/25/2009.
Br. crew added /repaired stiffeners to some bottom chord gusset platesGK 5/18/10.
North side of west truss, vert. members are within 1/4 inch of conc. sidewalk slab, south side shows 1-2 inches of clearance.GK 5/18/10.
2013: No significant change.
ELEMENT #152: Rust Beginning to Form. Top Flanges of the floor beams are rusting.
2003 FC Inspection:The floor beams are in generally good condition, with some scattered surface rust. Some of the floor beams are starting to develop pack rust at the chord connections.
2007 FC Inspection:There is minor scattered surface rust. Rust is forming on the top flanges where the deck is leaking. Pack rust continues to develop at the horizontal bracing gussets at the end of the floor beams but section

loss is minimal.
See Pictures and notes on file in the engineers office.
2009 FC inspection: Top flange corrosion on all floorbeams at ends and center joint; isolated section loss at lower flange connections to panel points Joe F 6/25/2009
Each Floorbeam has 17 stiffeners.Typical of these FB's is rust w/ minor sect. loss at the bottom of the 2 exterior stiffeners.
East truss 4 th FB from Pier 7 "center pier" has an area of corrosion with sect loss 1/8 + inches on bottom flange 4 ft. from south lower truss chord.
2013: Moved 620' into CS3 due to corrosion on the top flanges due to deck leakage.

ELEMENT #161: Rust Beginning to Form. Crack in Tack Weld on Nut to Hanger bottom pin 4th beam from north, 4th bent from the east. 2003 ultrasonic with no indications.
2003 FC Inspection:Ultrasonic straight beam examination was performed on all of the pins, utilizing a Panametrics Epoch III protable flaw detector and a 1/2" diameter 5 Mhz normal beam transducer. The pins were checked from both ends. A signal was noted on most of the pins, emanation from the shoulder area. This signal location is not from a stressed area of the pin, and is probably caused by a machined chamfer in the shoulder (see figure 1) No crack indications were noted on any of the pins.
2007 FC Inspection:No change from previous inspection.
See Notes and Pictures on file in the Engineer's office.
2009 FC inspection: Several cracked tack welds between nut and hanger channel noted Full UT inspection of pin/hanger assemblies will be done in 2011 on 4 year cycle. Joe F 6/25/2009
Span 4, 2 cotter keys in pins are 1/2 broke off, but enough remaining that nut can not come loose.GK 5/18/10
Ultrasonic examination was performed on the hanger pins during the 2011 FC inspection (no significant findings). The pin & hanger assemblies all have minor surface corrosion (condition state 2) - they are all functioning as intended.FC 6/11.
2013: No significant change. NDT not required until next FC inspection.

ELEMENT #202: 2 cotter keys missing bent 4 4 @ bent 5 &2 missing at bent 11GK 5/18/10.
There appears to be a dip at the joint above the second bent from the west. is this bent settled? Needs an evaluation.
2013: No significant change. All bents are tilted from 2-5 degrees.

ELEMENT #210: The ND Pier 6 is moving towards the river and is twisting causing cracks in Pier wall, there are cracks in the east Pier but not as severe as Pier 6. There is 6 feet of debris and soft silt at the bottom of the center pier - 2004 underwater inspection. 2009 FC inspection: Evidence of movement and twisting in west main pier (pier 6) Joe F 6/25/2009
A baseline was established using 2 eyebolts,one on no. side one on so. side of west end of pier 6 cap, a stringline streched between the 2 eyebolts show a bow of 5.5 inches in the pier wall.GK 5/18/10
Cracks continue in 2012
Bow in wall 5 5/8 inches in 2012 GK
2013: No significant change. Cracks in wall are more pronounced at Pier 6. See notes for element 234.

ELEMENT #215: Minor Hairline cracks @ W. abut. 1 sq. ft. of spall & deteriorated concrete @ S. end of W. Br/wall. Patch is coming out of conc. Bm. Water leaking through poured joint over parapets some cracking w/leaching of E. parapet. Horz. crack in S. half of W b/wall, 6 inches from the top. Small spall in top of E. br./wall w/rebar expsed.GK 5/18/10
Abut. 14, north 1/2 added an additional 8 inches to face of backwall, some wood forms still inplace. GK 5/18/10
Loose cork from cantilevered sect. working its way out. RH 5/2012
2013: There are spalls behind the fixed bearings on the West Abutment due to likely uplift from the structure. This is reflected in the current rating.

ELEMENT #220: This element should be used to describe the condition of the footing caps supporting the steel column bents. This is an each item - there are 9 bents, with a separate footing for each side (eastbound & westbound), so the total quantity should be 18 (each footing support 4 steel columns). Each footing cap is 30 ft. long, and is supported by two lower footings (each lower footing is supported by 2 steel H-piling). The lower footings should be below grade (not visible for inspection) - only the upper portions of the footing caps should be visible for inspection.FC 6/11
2013: There is minor cracking on the footings.

ELEMENT #234: Cracks in the west pier are at 1/16 inch wide.
As the bowing and cracking in Pier #6 extends through the “cap”, the ratings for this element should be the same as for element #210 (pier wall).FC 6/11
2013: No significant change. The cracks on Pier 6 range from hairline to 1/4" and extend through the pier wall and cap. The string line attached to the pier wall measures 1-3/8" on the north end and 6" at the center, indicating a

4-5/8" bow.
ELEMENT #300: First two joints from the ND end in the E.B.L. are leaking above the centering pins and are causing them to Rust and deterioration of the paint system.
2013: 1 foot of gland is cut on each joint.

ELEMENT #301: **Sealed jts./ pourable on 11/09. DSH
The poured joints located above the truss span floorbeams show evidence of leakage below (corrosion on the top flange of the floorbeams).FC 6/11
2013: No significant change.

ELEMENT #303: This element is to be used to describe the condition of the longitudinal median joint running along the full length of the bridge (installed in 1984). This is a 4” wide “cross linked ethylene vinyl acetate” joint material bonded to the concrete with epoxy. The joint has scattered areas of leakage (approximately half the length of the bridge)FC 6/11
2013: 15' in CS3 due to leakage at the transverse expansion joints.

ELEMENT #311: SW rocker is tipped all of the way toward the west. East rockers tipped slightly to the E. Base plates 3rd Beam from the N.@ E. expansion jt. & 3rd Beam from S. on N side @ W. expansion all are fractured. West Pier rockers tipped away from river @ 40 degrees Base plate on No. one beam @ W. finger jt. also fractured on inside face. Bearings and pins need cleaning and greasing. Bearing holder plate is broken loose, crack between beam and Bearing Holder east finger Jt.
2003 FC Inspection:The Rocker bearing at the SouthWest corner of the west truss span is fully expanded. The remainder of the bearings appear to be functioning properly.
2007 FC Inspection:The Southwest bearing is now only slightly in expansion. No other changes were noted.
See Notes and Pictures on file in the Engineers office.
I went to 9090 on 03/24/09 at 40 degrees to observe the bearing configurations, below are the results.
Both West Rockers have been moved from the original Positions;
The southwest Rocker has been moved twice on the base, to the west 5 3/4" for a total of 11.5 inches. and moved once on top attachment to the lower chord, to the east 7".
There is room for additional top movement four times for a total of 28" and no room for the base.
The bearing is currently out of plumb 4" to the west.
The northwest Rocker has been moved once on the base, to the west 5 3/4" and never moved on the top attachment to the lower chord.
There is room for additional top movement five times for a total of 35" and room for one move on the base for 5 3/4".
At Pier 6, SW rocker, The bearing is currently out of plumb 5" to the west.
The top movement in the South West top was done March 8th 2004, one of the bottom movements appear to have been done in 1999, it is not known when the other bottom movement was done.
The bolt pattern on the Lower chord is 7" and the pattern on the bearing is 21".
RN & MG 03/24/2009
SW rocker tilt/angle is the same as 2009 inspection.GK 5/18/10
The truss has expansion rocker bearings at Pier #6, Pier #7 (south truss only), and Pier #8. The element quantity is currently 148, which presumably includes two bearings for each steel column. The Bridge Office has never developed specific rating procedures for pinned steel columns. At this point we recommend include only the truss rocker bearings in this element, and including all components of the steel column (bottom rocker plate and upper pin) in the steel column element. All of the truss rocker bearings have surface corrosion. The south rocker bearing at Pier #6 was reset in 1972 (masonry plate moved 5” west), 1999 (masonry plate moved 6” to the west), and again in 2004 (upper rocker bearing plate moved 7” east at the bottom chord connection). The north truss rocker bearing at Pier #6 has been reset once (masonry plate moved 5-3/4” to the west).FC 6/11
SW rocker is monitered quarterly, and continues to move. In july of 2012 the bridge crew reset both rockers on Pier 6 by moving the rockers to the next set of holes on the bottom chord. The South rocker was canted 3/8" west and the North rocker 1 3/8" east after the reset. GK 2012
2013: The bearings were reset in 2012 on Pier 6.
At Pier 6 the NW Bearing is tilted 3.6 degrees and the SW Bearing is tilted 0.6 degrees to the east at 61 degrees.
At Pier 7 the SE Bearing is tilted 2.4 degrees and the SW Bearing is tilted 4.2 degrees to the east at 55 degrees.
At Pier 8 the NE Bearing is tilted 4.8 degrees and the SE Bearing is tilted 2.4 degrees to the east at 62 degrees.

ELEMENT #313: Abutment Bearings have been blasted and painted. 2009 FC inspection: Fixed bearings at west abutment are tipped toward tiver, with unsound concrete below them.
Bottom of bearings are corroding & rusting GK 5/18/10
Continuing to corrode. RH 5/2012.
2013: The fixed bearings at the West Abutment have uplift issues as the concrete around the bearings has now spalled off.

ELEMENT #321: W.B.L. has been overlaid with bituminous.
2013: The west approach slab has a 28" x 2" spall forming at the centerline joint.

ELEMENT #333: Minor spalls in rails. Minor spall in South rail @ West end w/rbar exposed. Two rail anchor castings on the same post,.NE end have been hit & are broken. Several spalls on the bottom of the railposts.
8 ft. cracked @ the south rail, mid span, west truss.GK 5/18/10
The concrete rail bases have scattered areas of cracking, delamination, and spalling (mainly on the approach spans). The north rail in the west truss span (L3'-L4') has impact damage - the upper rail casting is bent and fractured.FC 6/11
2013: Horizontal cracks with staining in the majority of the concrete rail sections. Scattered spalls in the posts and lower portion of the horizontal railing. North side rail second section from the east end, the west end post is missing. North side rail 11th section from the west end, the horizontal metal rail is pushed to the north between the 2nd and 3rd posts. The surface treatment on the metal rail sections is primarily intact. All rail sections on the bridge were intact except on the north rail on the east approach, where 1 rail post is damaged and not intact.

ELEMENT #357: Pack rust at chord splices and horizontal gussets connecting diagonal bracing on lower chord.
2013: No significant change.

ELEMENT #358: Deck is cracked with leaching, and has been epoxied in 08. DSH
2013: There is extensive unsealed cracking throughout the deck surface.

ELEMENT #359: Spalling aat numerous places in the approach spans @ CenterLine @ 2nd Floor Beam W. of E. pier. Concrete spalling out over Floor Beams @ the 3rd and 4th from W. pier with rebar exposed. Large spalls in deck soffet, north side full length.. 2009 FC inspection: spalling on deck overhang areas are less than 10% of total deck area. Joe F 6/25/2009
General appearance under deck is in very good condition other than north soffit.GK 5/18/10
As the spalling on the underside of the deck is generally confined to the fascia overhangs, it constitutes less than 10% of the total deck area, A rating of condition 3 would be appropriate FC 6/11
Bottom of deck in generally fair condition, poor condition on catilevered sections. RH 5/2012
2013: Underside of deck has numerous areas of saturation, especially in the east truss span. This warranted an NBI of 5.

ELEMENT #360: Appears south end of west main pier " P6 " has moved toward the river. Movement and twisting and continues to get worse.
There appears to be a dip at the joint above the second bent from the west.
While no change in the eastward movement of Pier #6 was observed during the 2011 inspection, the cumulative long-term movement of Pier #6 (25" to the east at the south end), probably warrants a rating of condition 3 for this smart flag.
2013: No significant change.

ELEMENT #361: The undermining of the footing at Bent #5 has increased since 2009 FC 6/11
Same in 2012
2013: No significant change.

ELEMENT #362: Bent Flange from impact on south fascia and 1st interior beams, east approach over road.
2013: No significant change.

ELEMENT #363: . ***The through trusses were blasted & painted in 1996. The % rated down is to denote existing sect loss. This problem will be evaluated at the next snooper inspection.***
2013: Areas of isolated section loss on the floorbeams and lower chords, but no significant loss of cross section.

ELEMENT #373: This element should be used to describe the condition of the hinge bearings supporting the suspended spans (in approach spans #5 & 8). There are swivel hinges at the truss ends (Piers #6 & 8), with expansion hinges at the other end of Spans #5 & 8 (4 joints, total of 32 assemblies). The hinge assemblies all have minor surface corrosion (condition state 2) - they are all functioning as intended.FC 6/11
Close inspection of swivel jts in 2012 show no signs of problems. GK / RH 5/2012
2013: No significant change.

ELEMENT #380: 2009 FC inspection: New element. Bottom gusset plates showing some corrosion and pack rust. Joe F 6/25/2009
2013: There is corrosion on the lower lateral braces and 2 hanger rods are broken.

ELEMENT #387: 2009 FC inspection: 3 ft x 4 ft washout and separation from slope protection at NE wingwall. Joe F 6/25/2009
2013: No significant change.

ELEMENT #411: Rubber diaphrams are deteriorating, there are leaks at 10 ft. and 30 ft. from the north in the NE gland that were repaired in 2009.This is not endangering any bearings. GK 5/18/10
2013: No significant change.

ELEMENT #422: There is some rust forming. 2009 FC inspection: Quantity changed to correspond to number of strip-seal and finger expansion joints.
Increased the element quantity by 2 to include the “splash zone” on the truss spans (one for north side and one for the south side). The truss splash zones should be rated as condition 4. FC 6/11
2013: No significant change.

ELEMENT #423: ** A (3) stage spot painting of gussets was completed in the week of 9/14/09. DSH
The bottom chord gusset plates have old pitting (painted over), with some active corrosion – all should be rated as condition state 3. Some of the upper chord gusset plates staining, surface corrosion, and isolated pitting (painted over).FC 6/11
2013: Quantity of 36 in CS2 due to minor corrosion starting to form on the interior surfaces of the gusset plates.

ELEMENT #964: DO NOT DELETE THIS CRITICAL FINDING SMART FLAG.
2013: No change.

ELEMENT #966: Do Not Remove. See in-depth report for location of F/C members.
2013: No change.

ELEMENT #967: 2009 FC inspection: new element. , Top plates have minor distortion due to construction fit-up. Bottom plates have distortion due to pack rust. Joe F6/25/2009
2013: No significant change.

ELEMENT #981: < none >
2013: NE delineator is bent over to the west.

ELEMENT #984: Downspouts missing on NE corner of the main span. Slope washouts @ E. River bank. Downspouts were extended from the deck drains by slipping squar tubing over the exsisting and fstening with 2 screws. Rust and corrosion is present at this connection and where fastened to the lower chord with a welded strap. The straps are rusting baddly and a few are broken, completely thru, should be repaired. GK 5/2012
2013: No significant change.

ELEMENT #985: Major transverse cracking in slope paving @ West end w/some heaving. Erosion at the 4th bent from the west. The Dike on the MN side has been removed. There are Gophers or something tunnelling in and around the abutment on the ND side.
Abut. 14, washout undermining NE br. seat. GK 5/18/10
ND built a rock flume type ditch alongside Bent 5 / pier 6 south side . GK 5/2012
2013: No significant change.

ELEMENT #986: cracks are present, with a 4 ft. area spalled out w/ rebar @ the west end of C&G.GK 5/18/10
2013: No significant change.

ELEMENT #988: Conduit seperated from light pole, E. approach on the North Rail
2009 FC inspection: Crack to to lack of fusion in a plug weld in outer gusset plate L-12, east truss, south side was discovered. Per consultation with Todd Nieman, crack was ground out. Crack penetrated full thickness of gusset plate, but did not extend into lower chord. Crack was caulked and painted after grinding.
Out of plumb measurements for the Bents are listed below: Measurement show the distance between a plumb line centered on the pin and the center of the lower bearing plate, in inches.

Bent	North	South
13	4.0	2.250
12	2.375	4.250
11	1.125	2.625
10	2.125	2.50
9	1.750	2.50
5	4.750	3.750 - 4.2 in 2012
4	1.250	1.250
3	3.750	2.125
2	3.375	3.750

In sept 2010, meas. were taken from Bent 5 to Pier 6.Bent 5 has an eyebolt protruding from the south concrete base near centerline, approx 5 ft. above ground and meas. were taken from the center of this eyebolt to the center of Pier 6,"Paintmark with an X" and to two eybolts on Pier 6 both approx 1 ft. above ground and approx. 4 ft. from the outside edge of the pier wall, both painted .
Meas. were taken with a steel chain.

So. meas.	72.15 ft.
Center	63.35 ft
North	68.95 ft.

Triang. meas. continue to remain close to orig. meas. in 2012 GK

Bents on west side /ND are tipping east,worst being bent 4 at 4.25 inches out of plumb and Mn side are tipping west worst being bent 12, 3 1/2 inches out of plumb.
Bents out of plumb meas. are on file GK 5/2012
2013: No significant change.

General Notes:

Inventory Item Notes:

58. Deck NBI:	2013 - Lower to 5. Scattered areas of saturation and efflorescence, especially in east bound lane of east truss span (Photos 1-5). Spalling and exposed rebar on the fascias throughout. Deck should be analyzed by the District via GPR to determine extent of deterioration. Rating lowered from 7 to 6 in 1993. Overlay has isolated concrete patches, with one section of delamination in Span #5 (eastbound). Underside of deck overhang has spalling (exposed rebar), particularly on the east approach spans.
36A. Brdg Railings NBI:	
36B. Transitions NBI:	
36C. Appr Guardrail NBI:	
36D. Appr Guardrail Terminal NBI:	
59. Superstructure NBI:	2013 - No change. Scattered areas of active corrosion and old pitting (painted over) on the bottom chord, bottom chord gusset plates and truss verticals & diagonals located near the deck level. Any section loss is minor.
60. Substructure NBI:	2013 - Lower to 4 due to continued movement and cracking of Pier 6 and spalling behind the fixed bearings on the West Abutment (it appears that the bearings are being pulled out (Photos 6-14).
61. Channel NBI:	2013 - No change (recommended in the 2012 Underwater Report).
62. Culvert NBI:	
71. Waterway Adeq NBI:	
72. Appr Roadway Alignment NBI:	

Appendix C: Structural Assessment Report - FC

BRIDGE STRUCTURAL ASSESSMENT REPORT

PURPOSE:

This report is a structural assessment of the structure and its ability to carry loads based on conditions identified in the attached bridge inspection report. The assessment is only a cursory review intended to provide guidance as to the relative hazards for structural conditions and deficiencies identified. This report is mandatory for all fracture critical bridges and is completed by the MnDOT Bridge Office upon receipt of the 7 Day FC Report; however, it is an OPTIONAL tool for agencies to utilize at their discretion for all other inspection types.

BRIDGE NO.: 9090

BRIDGE OWNER: State Highway Agency

DATE INSPECTED: 06/06/2013

STRUCTURE TYPE: Steel
Truss - Thru

FACILITY CARRIED: US 2

FEATURES INTERSECTED: RED RIVER

TYPE OF INSPECTION:
- ☐ ROUTINE
- ☑ FRACTURE CRITICAL
- ☐ PINNED ASSEMBLY:
- ☐ SPECIAL:
- ☐ DAMAGE:
- ☐ OTHER:

Check all that apply:

Redundancy:
- ☐ Load Path
- ☐ Structural
- ☐ Internal

Connection Type:
- ☐ Riveted
- ☑ Bolted
- ☑ Welded
- ☐ Other:

1. Was a critical finding identified during this inspection or upon structural review? ☐ Yes ☑ No

   a) If selected "Yes" above, state briefly the finding(s):

2. If a critical finding was identified, what is the current status?
   - ☐ Pending
   - ☐ Resolved
   - ☑ N/A

   a) Briefly state actions taken:

3. Does the condition of any bridge component indicate impaired function? Examples of bridge components with impaired function include elements that are: frozen or immoveable, out-of-plumb or misaligned, distorted or structurally deformed, excessively deteriorated, cracked, broken, eroded or scoured. ☐ Yes ☑ No

a) If selected "Yes" above, state briefly the component(s) and condition(s):

4. Does the overall condition of the bridge, or any of its components mentioned in Question 3, suggest the need for detailed structural analysis and/or a revised load rating? ☐ Yes ☑ No

a) If selected "Yes", state the reason for this recommendation and indicate a proposed timeframe in accordance with State of Minnesota Rule 8810.9500 (Subpart 2):

Continue monitoring the movement of rocker bearings and, if needed, relocate the bearings that have reached their design capacity.
Continue monitoring the tilting and movement of steel bents.
Continue monitoring the paint failure, pitting and surface corrosion conditions of all steel members at the approach spans and the river crossing trusses and, if needed, partially repaint the severe corroded areas.
Continue monitoring the pack rust forming at the gusset plates and chord splices, and the twisting of truss members.
Continue monitoring the cracked tack welds between the nuts and hanger channels.
Continue monitoring the pier movement and the uplifting of bearings.

5. Based on the structural assessment of these findings, recommendations include:

☑ Repair/Maintenance ☑ Monitoring Plan
☐ Other ☐ Increased Inspection Frequency

Explain recommended actions:

Continue monitoring the movement of rocker bearings and, if needed, relocate the bearings that have reached their design capacity.
Continue monitoring the tilting and movement of steel bents.
Continue monitoring the paint failure, pitting and surface corrosion conditions of all steel members at the approach spans and the river crossing trusses and, if needed, partially repaint the severe corroded areas.
Continue monitoring the pack rust forming at the gusset plates and chord splices, and the twisting of truss members.
Continue monitoring the cracked tack welds between the nuts and hanger channels.
Continue monitoring the pier movement and the uplifting of bearings.

6. Other comments:

Clean pigeon decrement before next inspection for human hazard concerns.
Repair the broken fastening straps for the downspouts.
Refill the washout at abutment and under piers.

Bridge Office Reviewer Jihshya J. LIn
06/17/2013

Appendix D: 2008 Load Rating

Truss Load Rating and Gusset Plate Review Summary Report

Bridge General Information

Bridge No. 9090 Bridge Type 303
Truss Load Rated By LHB Date 7/22/2008
Gusset Plate Reviewed by KAF Date 7/22/2008

Dead Load = Original + 2" overlay in 1984
Live Load = HS20
Section Loss Assumed at 5%
Bridge Rating Controlling Location: Truss L6U5 (L4'-U4)
Inventory HS 16.0 Operating HS 26.8
Load Post Not Required

Truss Member and Gusset Plate Ratings

Section Loss Assumed at 5%:
Controlling Upper Chord Member Operating Rating: HS 43.7
Controlling Bottom Chord Member Operating Rating: HS 37.3
Controlling Diagonals/Verticals Operating Rating: HS 26.7
Controlling Upper Joint Gusset Plates Operating Rating: HS 33.2 (U3' Flexure)

Section Loss Assumed at 15%:
Controlling Bottom Joint Gusset Plate Operating Rating: HS 29.2 (L0' Shear)

Truss Floor System Ratings

Section Loss Assumed at 5%:
Controlling FB Operating Rating: HS 27.4
Controlling Stringers Operating Rating: HS 33.2

Field Inspection

Based on 2007 FC inspection report, the top and bottom members have less than 5% section loss.

The stiffening angle was removed by inspector at L1A (L1/2). Based on the original plan and discussion of the reviewers, panel point L1a (L1/2) was set for temporary bearing during original erection and is not carrying much loads at current position. The stiffening angle is not required.

Recommendations and Actions

The bridge ratings have been lowered because a capacity reduction factor of 0.95 has been applied to truss ratings.

The areas above the bottom chord at the end of panel points need to be flush clean to prevent further corrosion.

Appendix E: 2012 Underwater Report

UNDERWATER BRIDGE INSPECTION REPORT

STRUCTURE NO. 9090

TRUNK HIGHWAY NO. 2

OVER THE

RED RIVER OF THE NORTH

DISTRICT 2 - POLK COUNTY

AUGUST 29, 2012

PREPARED FOR THE

MINNESOTA DEPARTMENT OF TRANSPORTATION

BY

AYRES ASSOCIATES & COLLINS ENGINEERS, INC.

JOB NO. 7423

MINNESOTA DEPARTMENT OF TRANSPORTATION
UNDERWATER BRIDGE INSPECTION

REPORT SUMMARY:

The substructure unit inspected at Bridge No. 9090, Pier 7, was found to be generally in good condition with no defects of structural significance observed. The concrete from the waterline to the channel bottom was smooth and sound. The extent of footing exposure at the upstream column has slightly decreased since the last inspection with maximum vertical exposure reduced from 8 inches in 2008 to no exposure in 2012. The top of the footing at the downstream column remains partially exposed with no vertical exposure. Timber debris accumulation around both columns of Pier 7 were moderate in extent and comparable to timber accumulation levels of 2008.

INSPECTION FINDINGS:

(A) Channel bottom around entire Pier 7 was typically 6-inch-diameter stone and firm clay.

(B) Most of the area of the top of the footing at the upstream column was exposed with no vertical exposure.

(C) The top of the footing at the downstream column was partially exposed along the north and east sides with no vertical exposure.

(D) Moderate accumulation of timber debris, consisting of logs and branches 1.5 feet in diameter and smaller, was observed at the upstream column extending from channel bottom up 3 feet and at the downstream column extending from channel bottom up 4 feet.

(E) Each face of the pier diaphragm wall connecting the columns exhibited five vertical hairline to 1/16 inch wide cracks. Cracks extended from waterline to 9 feet above waterline.

RECOMMENDATIONS:

(A) Monitor footing exposure and countermeasures, such as placing riprap around the exposed footing, may become warranted if further exposure continues to occur.

(B) Reinspect the submerged substructure units at the normal maximum recommended (NBIS) interval of sixty (60) months.

Respectfully submitted,

Inspection Team Leader

Ayres Associates, Inc.

Brian K. Schroeder
Registered Professional Engineer
State of Minnesota

PROFESSIONAL ENGINEER
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.

Daniel G. Stromberg

Date 6/30/14 License # 21491

COLLINS ENGINEERS, INC.

Daniel G. Stromberg
Registered Professional
Engineer, State of Minnesota

MINNESOTA DEPARTMENT OF TRANSPORTATION
UNDERWATER BRIDGE INSPECTION

1. BRIDGE DATA

Bridge Number: 9090

Feature Crossed: The Red River of the North

Feature Carried: Trunk Highway No. 2

Location: District 2 - Polk County

Bridge Description: The superstructure consists of multiple steel through truss spans supported by two reinforced concrete abutments and reinforced concrete piers. The piers are supported by timber piles. The substructure units are numbered starting from the west.

2. INSPECTION DATA

Professional Engineer/Team Leader: Brian K. Schroeder, P.E.

Dive Team: Jason A. Cook, Anthony J. Coffaro

Date: August 29, 2012

Weather Conditions: Sunny, 80° F

Underwater Visibility: None/Negligible

Waterway Velocity: 1 ft/sec

3. SUBSTRUCTURE INSPECTION DATA

Substructure Inspected: Pier 7

General Shape: The pier consists of a reinforced concrete cap supported by two multi-sided concrete columns connected by a slender concrete diaphragm wall braced with an integral horizontal strut. The pier is founded on a rectangular footing supported by timber piles.

Maximum Water Depth at Substructure Inspected: Approximately 27.1 feet.

4. WATERLINE DATUM

Water Level Reference: The top of pier cap at downstream end of Pier 7.

Water Surface: The waterline was approximately 33.5 feet below reference.
Waterline Elevation = 794.8

5. NBIS CODING INFORMATION (Minnesota specific codes are used for 92B and 113)

Item 60: Substructure: Code 7

Item 61: Channel and Channel Protection: Code 6

Item 92B: Underwater Inspection: Code B/08/12

Item 113: Scour Critical Bridges: Code L/04

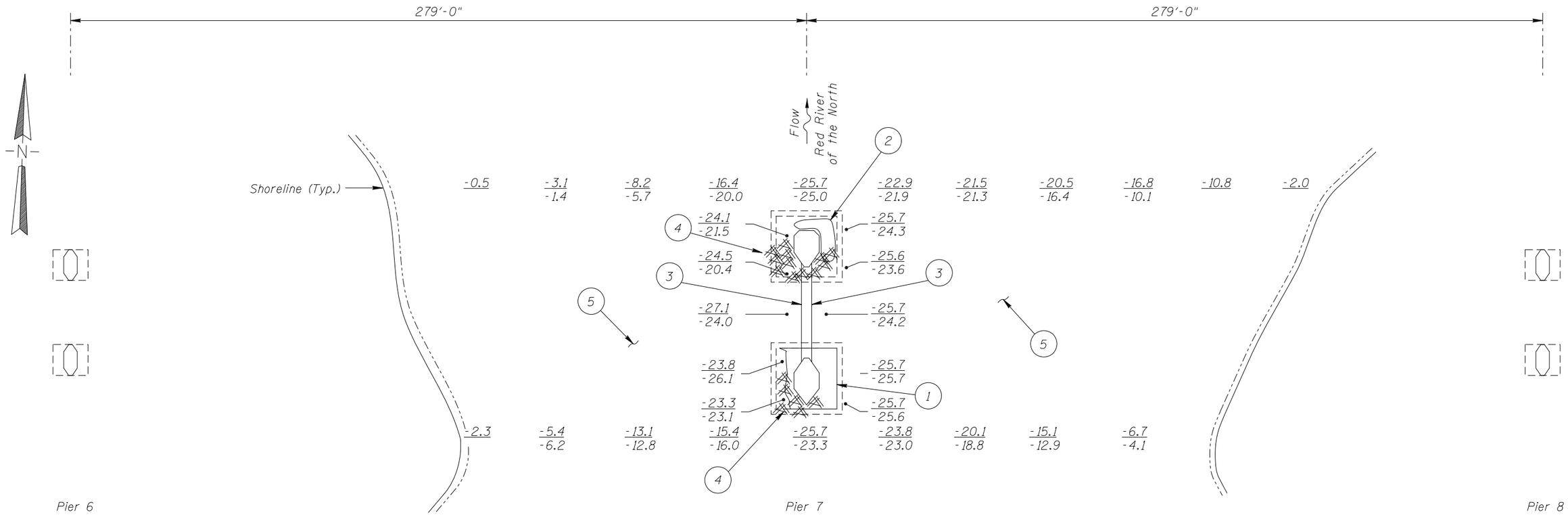
Bridge is scour critical because abutment or pier foundation is rated as unstable due to observed scour at bridge site.
_____ Yes X No

6. STRUCTURAL ELEMENT CONDITION RATING

Item #	Element Description	Quantity	Unit	Conditions				
				1	2	3	4	5
205	Reinforced Concrete Column	2	EA		2			
210	Concrete Pier Wall	70	LF	70				
220	Reinforced Concrete Footing	2	EA	2				
361	Scour	1	EA	1				
985	Slopes and Slope Protection	1	EA		1			

Photograph 1. Overall View of Structure, Looking North.

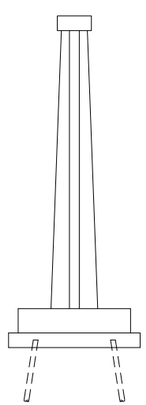
Photograph 2. Overall View of Pier 7, Looking East.



SOUNDING PLAN

GENERAL NOTES:

1. Pier 7 was inspected underwater.
2. At the time of inspection on August 29, 2012, the waterline was located approximately 33.5 feet below the top of the pier cap at the downstream end of Pier 7. This corresponds with a waterline elevation of 794.8 feet based on the report dated September 8, 2000.
3. Soundings indicate the water depth at the time of inspection and are measured in feet.
4. Soundings were taken parallel to the bridge at $\frac{1}{8}$ point intervals between the substructure units.



TYPICAL END VIEW OF PIER

INSPECTION NOTES:

1. Top of the footing at the upstream column was exposed throughout except along the west side of the footing. No vertical exposure was observed.
2. The top of the downstream footing was partially exposed at the north and east sides with no vertical exposure observed.
3. Five vertical cracks hairline to 1/16 inch wide each side of the web-wall. Cracks extended from waterline to 9 feet above.
4. Moderate accumulation of timber debris consisting of logs and branches 1.5 feet in diameter and smaller was observed at the upstream column extending from channel bottom up 3 feet and at the downstream column extending from channel bottom up 4 feet.
5. The channel bottom typically consisted of 6 inch diameter stones and firm clay.

Legend

-3.0	Sounding Depth from Waterline (8/29/12)
-2.5	Sounding Depth from Waterline (5/15/08)
	Timber Debris

Note:

All soundings based on 2012 waterline location.

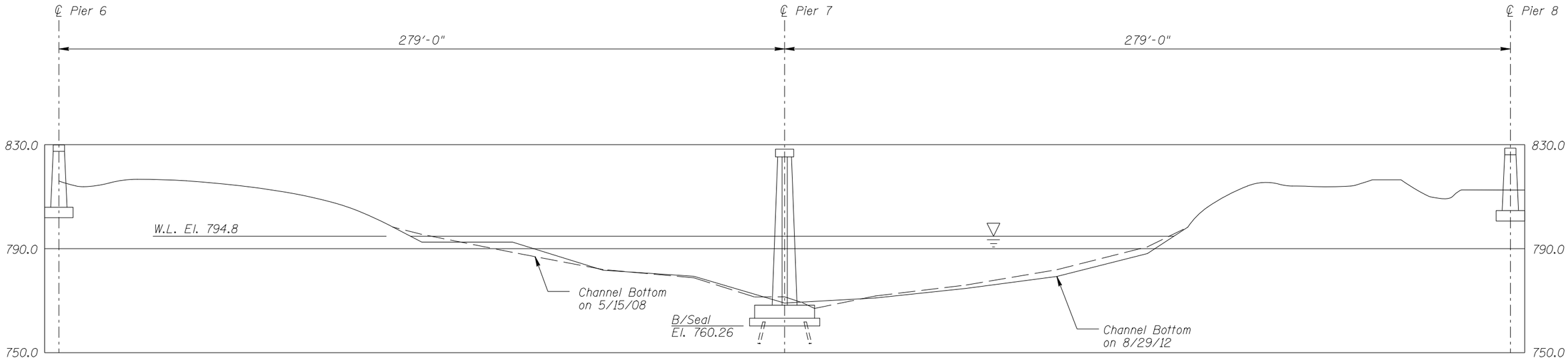
MINNESOTA
DEPARTMENT OF TRANSPORTATION
UNDERWATER BRIDGE INSPECTION

STRUCTURE NO. 9090
OVER THE RED RIVER OF THE NORTH
DISTRICT 2, POLK COUNTY

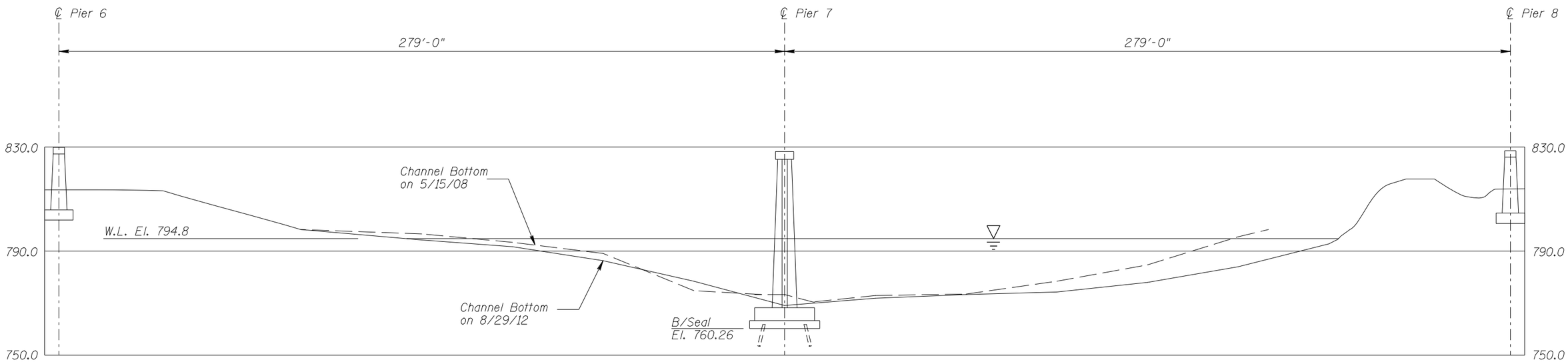
INSPECTION AND SOUNDING PLAN

COLLINS ENGINEERS INC
123 North Wacker Drive
Suite 300
Chicago, Il. 60606
(312) 704-9300
www.collinsengr.com

Drawn By: JAC	AYRES ASSOCIATES 3433 Oakwood Hills Parkway Eau Claire, WI 54701 www.AyresAssociates.com	Date: SEPT. 2012
Checked By: BKS		Scale: NTS
Code: 54739090		Figure No.: 1



UPSTREAM FASCIA PROFILE



DOWNSTREAM FASCIA PROFILE

Note:

Refer to Figure 1 for General Notes.

MINNESOTA
DEPARTMENT OF TRANSPORTATION
UNDERWATER BRIDGE INSPECTION

STRUCTURE NO. 9090
OVER THE RED RIVER OF THE NORTH
DISTRICT 2, POLK COUNTY

UPSTREAM AND DOWNSTREAM
FASCIA PROFILES

COLLINS ENGINEERS INC 123 North Wacker Drive Suite 300 Chicago, IL 60606 (312) 704-9300 www.collinsengr.com	Drawn By: JAC	Date: SEPT. 2012
	Checked By: BKS	Scale: 1"=40'
	Code: 54739090	Figure No.: 2
AYRES ASSOCIATES 3433 Oakwood Hills Parkway Eau Claire, WI 54701 www.AyresAssociates.com		

MINNESOTA DEPARTMENT OF TRANSPORTATION
OFFICE OF BRIDGES AND STRUCTURES
DAILY DIVING REPORT

INSPECTORS: Ayres Associates DATE: August 29, 2012
ON-SITE TEAM LEADER: Brian K. Schroeder, P.E.
BRIDGE NO: 9090 WEATHER: Sunny, 80° F
WATERWAY CROSSED: Red River of the North
DIVING OPERATION: _____ SCUBA X SURFACE SUPPLIED AIR
_____ OTHER _____
PERSONNEL: Jason A. Cook, Anthony J. Coffaro
EQUIPMENT: SSA, Hammer, Camera, 20-foot Boat with Jet, Sounding Rod, Probe
TIME IN WATER: 9:30 AM
TIME OUT OF WATER: 10:05 AM
WATERWAY DATA: VELOCITY 1 ft/sec
VISIBILITY None/Negligible
DEPTH 27.1 feet maximum at Pier 7
ELEMENTS INSPECTED: Pier 7
REMARKS: Overall, the concrete of the pier was smooth and sound at and below the waterline. Top face of the footing at the upstream column was exposed with no vertical exposure. The top face of the footing at the downstream column was exposed on the north and south sides with no vertical exposure. There was a moderate accumulation of timber debris at both columns of Pier 7 extending from channel bottom up 3 to 4 feet. Five vertical cracks of up to 1/16 inch in width were present at the midpoint of concrete diaphragm wall extending from waterline to 9 feet above.

FURTHER ACTION NEEDED: _____ YES X NO

Monitor footing exposure and countermeasures, such as placing riprap around the exposed footing, may become warranted if further exposure continues to occur.

Reinspect the submerged substructure units at the normal maximum recommended (NBIS) interval of sixty (60) months.

MINNESOTA DEPARTMENT OF TRANSPORTATION
OFFICE OF BRIDGES AND STRUCTURES

UNDERWATER INSPECTION CONDITION RATING FORM

BRIDGE NO. 9090
INSPECTORS Ayres Associates
ON-SITE TEAM LEADER Brian K. Schroeder, P.E.
WATERWAY CROSSED Red River of the North

INSPECTION DATE August 29, 2012

NOTE: USE ALL APPLICABLE CONDITION DEFINITIONS AS DEFINED IN THE MINNESOTA RECORDING AND CODING GUIDE INCLUDING GENERAL, SUBSTRUCTURE, CHANNEL AND PROTECTION, AND CULVERTS AND WALL DEFINITIONS TO COMPLETE THIS FORM.

CONDITION RATING

			SUBSTRUCTURE						CHANNEL					GENERAL					
UNIT REFERENCE NO.	UNIT DESCRIPTION	MAXIMUM DEPTH OF WATER	PILING	COLUMNS, SHAFTS, OR FACES*	FOOTINGS	DISPLACEMENT	OTHER	OVERALL SUBSTRUCTURE CONDITION CODE*	SCOUR	EMBANKMENT EROSION	EMBANKMENT PROTECTION	OTHER (DRIFT/DEBRIS)	OVERALL CHANNEL & PROTECTION CONDITION	CONCRETE	STEEL	TIMBER	LOSS OF SECTION	PREVIOUS REPAIR OR MAINTENANCE	OTHER
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
	Pier 7	27.1'	N	7	7	7	N	7	6	6	6	6	6	7	N	N	7	N	N

*UNDERWATER PORTION ONLY

REMARKS: Overall, the concrete of the pier was smooth and sound at and below the waterline. Top face of the footing at the upstream column was exposed with no vertical exposure. The top face of the footing at the downstream column was exposed on the north and south sides with no vertical exposure. There was a moderate accumulation of timber debris at both columns of Pier 7 extending from channel bottom up 3 to 4 feet. Five vertical cracks of up to 1/16 inch in width were present at the midpoint of concrete diaphragm wall extending from waterline to 9 feet above.

NOTES: ATTACH SKETCHES AS NEEDED, IDENTIFY REMARK BY REFERRING TO UNIT REFERENCE NO. AND REMARK NO.
USE GENERAL SECTION TO IDENTIFY OVERALL PRESENCE OF SPALLS, CRACKS, CORROSION, ETC.

Minnesota Department of Transportation
Office of Materials
DRILL DUST SUMMARY REPORT

Project No: For Info Only **Submitted By:** Greg Klein **Date Received:** 07/08/2013

Remarks: 2" Cores sampled on 6/25/13, checking in-place deck conditions

Sample ID	Field ID	Bridge No.	Source	Depth	ppmCl
CO-DD13-0001	Core #1	9090	Between B11-B12 deck location unkown WBL Rt. /center of Lane		3002
CO-DD13-0002	Core #1	9090	deck location unkown	1" - 2"	1216
CO-DD13-0003	Core #1	9090	deck location unkown	2" - 3"	1613
CO-DD13-0004	Core #1	9090	deck location unkown	3" - 4"	1412
CO-DD13-0005	Core #1	9090	deck location unkown	4" - 5"	665
CO-DD13-0006	Core #1	9090	deck location unkown	5" - 6"	445
CO-DD13-0007	Core #1	9090	deck location unkown	6" - 7"	242
CO-DD13-0008	Core #1	9090	deck location unkown	7" - 8"	462
CO-DD13-0009	Core #1	9090	deck location unkown	8" - 9"	458
CO-DD13-0010	Core #2	9090	deck location unkown 20ft. W. of P8	0" - 1"	2786
CO-DD13-0011	Core #2	9090	deck location unkown WBL Rt. /center of Lane	1" - 2"	1091
CO-DD13-0012	Core #2	9090	deck location unkown	2" - 3"	733
CO-DD13-0013	Core #2	9090	deck location unkown	3" - 3 1/2"	1243
CO-DD13-0014	Core #2	9090	deck location unkown	A?" - ?"	631
CO-DD13-0015	Core #2	9090	deck location unkown	B?" - ?"	527
CO-DD13-0016	Core #2	9090	deck location unkown	C?" - ?"	2092
CO-DD13-0017	Core #3	9090	deck location unkown Between B5-P6	0" - 1"	3134
CO-DD13-0018	Core #3	9090	deck location unkown EBL Rt. /center of Lane	1" - 2"	1965
CO-DD13-0019	Core #3	9090	deck location unkown	2" - 3"	1726
CO-DD13-0020	Core #3	9090	deck location unkown	3" - 4"	1199
CO-DD13-0021	Core #3	9090	deck location unkown	4" - 5"	830
CO-DD13-0022	Core #3	9090	deck location unkown	5" - 6"	747
CO-DD13-0023	Core #3	9090	deck location unkown	6" - 7"	801
CO-DD13-0024	Core #3	9090	deck location unkown	7" - 8"	570
CO-DD13-0025	Core #3	9090	deck location unkown	8" - 9"	599
CO-DD13-0026	Core #4	9090	deck location unkown 25ft. E. of B10	0" - 1"	4492
CO-DD13-0027	Core #4	9090	deck location unkown EBL. Rt. /center of Lane	1" - 2"	2634
CO-DD13-0028	Core #4	9090	deck location unkown	2" - 3"	948
CO-DD13-0029	Core #4	9090	deck location unkown	3" - 4"	641
CO-DD13-0030	Core #4	9090	deck location unkown	4" - 5"	582
CO-DD13-0031	Core #4	9090	deck location unkown	5" - 6"	540
CO-DD13-0032	Core #4	9090	deck location unkown	6" - 7"	331
CO-DD13-0033	Core #4	9090	deck location unkown	7" - 8"	293
CO-DD13-0034	Core #4	9090	deck location unkown	8" - 9"	902

Comments:

Copy To: Roger Hille

Report Approved By:

David L. Joerson

If you have any questions, please call: (651) 366-5550

9:00-12:00 6/25/13 GK/TP
CORES/2" diam. Drill Crew
GF TRAFFIC

MN WBL 1st. core Between B11-B12 WBL Rt Center of Lane

MN WBL 2nd. " 20' West PB " " Center of Lane

ND EBL 3 1/2 Way Bent 5- Pb Center Rt. Lane

MN EBL 4 25' E Bent 10 ℄ of Rt. Lane
Between B10-B11

CORES SENT TO MAPLEWOOD
6/26/13
GK

Appendix D:
Secretary of the Interior's Standards for Rehabilitation

The Secretary of the Interior's (SOI) Standards for the Treatment of Historic Properties were authorized by the National Historic Preservation Act of 1966. The Standards were revised in 1992 and the revisions codified in 1995 (36 CFR 68). The Standards are designed to be applied to all types of historic properties including buildings, sites, structures, and districts. They are accompanied by a set of Guidelines (Weeks and Grimmer 1995).

The Standards and Guidelines outline a hierarchy of four treatment approaches: Preservation, Rehabilitation, Restoration, and Reconstruction. The first treatment, Preservation, places a high premium on the retention of all historic fabric through conservation, maintenance, and repair. Rehabilitation, the second treatment, emphasizes the retention and repair of historic materials but more latitude is provided for adaptation to support new or expanded use, or for replacement of significantly deteriorated fabric. Restoration, the third treatment, focuses on the retention of materials from the most significant time in a property's history while permitting the removal of materials from other periods. Reconstruction, the fourth treatment, establishes limited opportunities to recreate a building, structure, object, or landscape that has disappeared.

Rehabilitation is the treatment approach most applicable to the Kennedy Bridge project.

Regardless of the treatment path, the Standards and Guidelines emphasize that the most conservative treatments – retaining, preserving, and repairing historic materials and features – are the preferred approach for preserving the historic integrity of a property. They begin with the recommendation that the property's historic features be identified, retained, preserved, protected, and maintained. If repair is necessary, the work should begin with the least intervention possible. Next in the hierarchy, if parts of an element are extensively damaged or deteriorated, limited in-kind replacement of those parts is acceptable. Removal and replacement of a historic feature that could reasonably be repaired, and thus preserved, is never recommended.

The Standards and Guidelines direct that when alterations are needed to assure continued use of a historic property, the alterations should not radically change, obscure, or destroy important materials, features, or finishes.

The Guidelines also advise that the effect of proposed changes be assessed within the overall context of the entire property, stating that "loss of [historic] character is just as often caused by the cumulative effect of a series of actions that would seem to be minor interventions" as by a single action.

SOI Standards for Rehabilitation

1. A property will be used as it was historically or be given a new use that requires minimal change to its distinctive materials, features, spaces, and spatial relationships.

2. The historic character of a property will be retained and preserved. The removal of distinctive materials or alteration of features, spaces, and spatial relationships that characterize a property will be avoided.

3. Each property will be recognized as a physical record of its time, place, and use. Changes that create a false sense of historical development, such as adding conjectural features or elements from other historic properties, will not be undertaken.

4. Changes to a property that have acquired historic significance in their own right will be retained and preserved.

5. Distinctive materials, features, finishes, and construction techniques or examples of craftsmanship that characterize a property will be preserved.

6. Deteriorated historic features will be repaired rather than replaced. Where the severity of deterioration requires replacement of a distinctive feature, the new feature will match the old in design, color, texture, and, where possible, materials. Replacement of missing features will be substantiated by documentary and physical evidence.

7. Chemical or physical treatments, if appropriate, will be undertaken using the gentlest means possible. Treatments that cause damage to historic materials will not be used.

8. Archeological resources will be protected and preserved in place. If such resources must be disturbed, mitigation measures will be undertaken.

9. New additions, exterior alterations, or related new construction will not destroy historic materials, features, and spatial relationships that characterize the property. The new work will be differentiated from the old and will be compatible with the historic materials, features, size, scale and proportion, and massing to protect the integrity of the property and its environment.

10. New additions and adjacent or related new construction will be undertaken in such a manner that, if removed in the future, the essential form and integrity of the historic property and its environment would be unimpaired.

Appendix E:
US Trunk Highway 2 Red River Crossing (Kennedy Bridge) Planning Study

Final Report

US Trunk Highway 2 Red River Crossing (Kennedy Bridge) Planning Study

Prepared for

MINNESOTA DEPARTMENT OF TRANSPORTATION

Prepared by

CH2MHILL®

April 2014

TBG011514142833MKE

Bridge Planning Study

For the Kennedy Bridge
MnDOT Bridge No. 9090/NDDOT Bridge No. 02-350.220

I hereby certify that this report, and referenced technical memoranda, were 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.

Dale R. Thomas

Dale R. Thomas

Date: 4/28/14 Reg. Number: 22143

Report Submitted to:

Mr. Derrick Dasenbrock, P.E.
Minnesota Department of Transportation
Office of Materials and Road Research
1400 Gervais Avenue
Maplewood, MN 55109

Report Prepared by:

CH2M HILL

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Appendixes

A MnDOT Draft Statement of Purpose and Need

Acronyms and Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
ADT	average daily traffic
ATR	Automatic Traffic Recorder
CE	Categorical Exclusion
DNR	Department of Natural Resources
DOT	Department of Transportation
EA	Environmental Assessment
EIS	Environmental Impact Statement
Engineering	City of Grand Forks, ND
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
GF-EGF MPO	Grand Forks-East Grand Forks Metropolitan Planning Organization
HCS	Highway Capacity Software
LOS	level of service
MnCMAT	MnDOT Crash Mapping Analysis Tool
MnDOT	Minnesota Department of Transportation
mph	miles per hour
NAVD	North American Vertical Datum
NDDOT	North Dakota Department of Transportation
NOAA	National Oceanic and Atmospheric Administration
NRHP	National Register of Historic Places
PHF	peak hour factor
PHV	peak hour volume
SAC	Study Advisory Committee
SHPO	State Historic Preservation Office
TM	technical memorandum
USACE	U.S. Army Corps of Engineers
USEPA	U.S. Environmental Protection Agency
USFWS	U.S. Fish & Wildlife Service

SECTION 1

Introduction and Report Purpose

The Minnesota Department of Transportation (MnDOT) led the U.S. Highway 2 Bridge Planning Study through 2013 to develop an approach to maintain this major crossing of the Red River of the North (Red River). The MnDOT team completed the work in consultation with the North Dakota Department of Transportation (NDDOT) and the Federal Highway Administration (FHWA). Section 1 introduces the report by referencing background, listing the supporting technical documents, and by describing the need for action and proposed action/approach for MnDOT's project S.P. 6018-02 (addressing MnDOT Bridge No. 9090; NDDOT Bridge No. 02-350.220).

The Kennedy Bridge is eligible for listing on the National Register of Historic Places (NRHP). Because of this, a proposed action to maintain the crossing could have an adverse effect on a protected historic resource. The Planning Study outlined project development alternatives and issues to be addressed, but it does not establish a preferred alternative nor finalize any determinations under Section 106 of the Historic Preservation Act.

1.1 Background and Supporting Documents

This report provides a summary of the Kennedy Bridge Planning Study, which was the first major step in maintaining the Red River crossing. The proposed action is to rehabilitate or replace the bridge, and to maintain the area's major interregional river crossing on U.S. Highway 2. The Kennedy Bridge (MnDOT Bridge No. 9090; NDDOT Bridge No. 02-358.220), which is 1,261 feet long and provides four highway lanes, also serves as a vital local connection between the cities of Grand Forks, North Dakota and East Grand Forks, Minnesota. The bridge, built in 1963 (Exhibit 1-1), presents a number of technical challenges, historic bridge engineering characteristics, and opportunities for context-sensitive improvements, as presented in Sections 2 and 3.

EXHIBIT 1-1
U.S. Highway 2 Kennedy Bridge (Looking South from Grand Forks, ND)

The Planning Study was organized to provide a series of documents used to commence and focus the overall decision-making process. That process and the results are addressed in a series of technical documents, including the following:

- **Technical Memoranda (2): Pier 6 Movement Capacity; Summary of Pier 6 Movement Records—**The two technical documents address the known issue of movement in the bridge's Pier 6 and provide detailed background data (see Section 1.4).
- **Technical Memorandum (TM): Bridge Rehabilitation Alternatives—**The study's main technical/engineering document, providing an assessment of bridge rehabilitation actions, including alternative levels of investment for rehabilitation and recommendations, which carry into the Final Report. The TM also includes appended information on geotechnical conditions, Red River hydraulics, and scour.
- **TM: Bridge Replacement Options—**Summarizes findings from a supplementary analysis of bridge replacement alternatives, completed to provide a comparison to Bridge Rehabilitation, and to identify the most promising bridge types and alignments, should bridge replacement become a course of action.

More information about the documents is available at the Kennedy Bridge website (www.mndot.gov/d2/projects/kennedybridge) or can be obtained by contacting MnDOT.

1.2 Report Objectives and Project Development Context

1.2.1 Overall Bridge Study Context and Next Steps

The Final Report provides a summary of the Kennedy Bridge Planning Study, with reference to the technical studies. Additionally, it provides contextual information and data regarding functional/transportation background and needs, and establishes the broad vision and framework for a potential bridge rehabilitation project.

The Planning Study addressed issues of project context through development of project goals for bridge rehabilitation and bridge replacement. Based on these factors, and the potential to address needs, bridge rehabilitation is considered the priority action. The bridge rehabilitation concepts identified through the study are structured to cost-effectively address needs, while avoiding or minimizing adverse impacts.

The study also looked at bridge replacement concepts, which would provide more opportunity for functional improvements and fewer maintenance issues in the coming decades compared to rehabilitation—but only with greater initial costs and more adverse environmental impacts. Because the Kennedy Bridge is eligible for listing on the NRHP, the long-term feasibility and cost-effectiveness of bridge rehabilitation must be considered. Based on the study's findings, considering costs, funding, and environmental review steps, a determination will soon be made as to whether rehabilitation of the Kennedy Bridge is confirmed as the preferred alternative.

Section 2 addresses the setting and context for development of a project, including an overview of the environmental and community setting. Section 3 addresses the primary alternatives, including choices and challenges for next steps, as the Planning Study concludes. The next steps will follow one of a few configuration and investment choices for a bridge rehabilitation or replacement project, with bridge rehabilitation screened first to de-

termine if it will perform reasonably. The preferred course of action will be identified by MnDOT and NDDOT after completion and public release of this Final Report. The likely next steps include a more detailed bridge rehabilitation design study (see Section 3).

1.2.2 Study Participants and Project Development Goals/Objectives

The planning work included a series of meetings with a Study Advisory Committee (SAC), two rounds of outreach to the general public, and other meetings and consultations with stakeholders. The invited and participating agencies, given representation on the SAC or with opportunities afforded regularly to provide input, included the following:

- MnDOT
- NDDOT
- FHWA
- City of Grand Forks, ND (Engineering)
- City of East Grand Forks, MN (Department of Public Works)
- Polk County, MN
- Grand Forks County, ND
- Grand Forks-East Grand Forks Metropolitan Planning Organization (GF-EGF MPO)
- Grand Forks Historic Preservation Commission
- U.S. Fish & Wildlife Service (USFWS) (MN & ND)
- U.S. Army Corps of Engineers (USACE) (St. Paul & Bismarck)
- U.S. Environmental Protection Agency (USEPA) (Region 5—MN, Region 8—ND)
- Minnesota Department of Natural Resources (DNR)
- Minnesota Pollution Control Agency
- North Dakota Game & Fish Department
- North Dakota Department of Health—Environmental Health Section
- Minnesota State Historic Preservation Office (SHPO)
- North Dakota SHPO

The planning process developed and refined goals and objectives for development of a transportation improvement project through technical study of the Kennedy Bridge and through input from the agencies and the local communities.[1] During the process, the following goals and objectives were discussed and refined:

- **Set priorities to maintain the U.S. Highway 2 river crossing—**From the beginning of the work, the immediate structural question concerned the known movement of the bridge's Pier 6, which presents an immediate bridge maintenance priority. A clear understanding of the potential for further adjustment in response to movement was needed (see Section 1.4). The other objective was to consider the entire bridge; to outline, prioritize, and define longer-term scenarios to maintain the river crossing—namely, bridge rehabilitation or replacement. Section 1.4 identifies the proposed components of a bridge rehabilitation project, which are also addressed in substantial detail in the Bridge Study's technical memoranda.

[1] SAC meetings/teleconferences were held on 3/6/13, 5/22/13, 7/31/13, 10/2/13, and 12/17/13. Public information meetings (held on 7/17/13, 12/16/13, and 12/17/13) were advertised in local newspapers and through the Web. Additional outreach included presentations to the Grand Forks and East Grand Forks City Councils (12/16/13 and 12/17/13, respectively) and briefings/discussions with the Metropolitan Planning Organization Executive Board on 7/17/13 and 1/15/14. All of the listed agencies were regularly provided documentation of Planning Study meetings.

- **Address community context and the environment—**The study's approach included efforts to understand the Kennedy Bridge setting and the context in which a transportation infrastructure project is being proposed. Section 2 of this report provides more depth on the important factors considered in planning the project. Based on the study team's evaluation of the bridge vicinity and input from stakeholders, the main contextual objectives for project development include the following:
  - **Minimize traffic disruptions—**Development of a project warrants advanced planning to minimize periods of closure and limited capacity at the Kennedy Bridge, as well as considerations for the area's system of river crossing bridges and roadways. This objective was the topic most frequently raised by local stakeholders (see Section 2.2.1).
  - **Accommodate bicycle and pedestrian crossings—**The immediate area includes a remarkable bicycle and pedestrian environment (the Red River Greenway) and there is proven need and community interest in accommodation for bicycles and pedestrians at the crossing. This objective was also raised often by local stakeholders (see Sections 2.3 and 3.1.2.3).
  - **Respect historic resources and other environmental values—**The Kennedy Bridge is eligible for listing on the National Register for Historic Places. Other historic resources are also found in the immediate vicinity. Additional environmental factors include the Red River Greenway, a Minnesota state park and campground, and the Red River itself, with its history of major floods, soil movement, and other natural attributes (see Sections 2.2.2 and 3.1.2.4).

1.3 U.S. Highway 2 Kennedy Bridge Need for Action

The Kennedy Bridge includes the following characteristics (Exhibits 1-2 and 1-3):

- 1,261 feet long overall; 2 steel truss main spans, each 279 feet long, with 19.8-foot vertical portal clearances above roadway; and 11 steel-beam approach spans (5 spans to west and 6 spans to east).
- Eastbound and westbound directions each have two, 12-foot-wide lanes with 3-foot outside shoulders and 1-foot inside buffers next to the median.

EXHIBIT 1-2
U.S. Highway 2 Kennedy Bridge Elevation (Looking North)

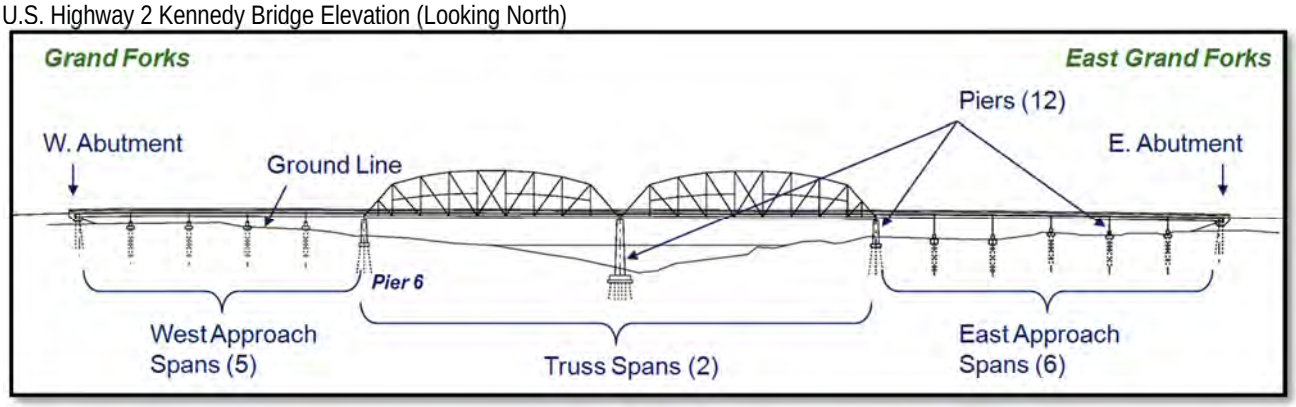
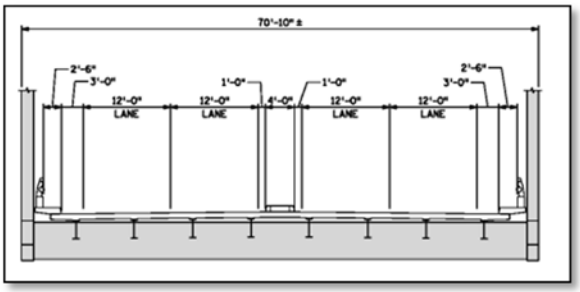


EXHIBIT 1-3
U.S. Highway 2 Kennedy Bridge Deck Cross Section and Photograph



- The bridge has no sidewalks, but each side includes a 2.5-foot-wide raised curb inside the barrier rails (pedestrians and bicycles are prohibited on the bridge, as posted westbound only at the east abutment).

Based on a June 2013 bridge inspection, the Kennedy Bridge has a sufficiency rating of 48.2 (based on a 100-point scale) and is now classified as structurally deficient. The measures do not mean that the bridge is unsafe, but the sufficiency rating provides a scale relative to other bridges through which to determine project development priorities and actions to be taken. In this case, the general condition rating of the substructures (especially Pier 6) had an important role in the structurally deficient classification.

The primary purpose of a Kennedy Bridge project is to provide a structurally sound crossing of U.S. Trunk Highway 2 over the Red River between Grand Forks and East Grand Forks. The need for action was documented in detail by MnDOT in July 2012, in a draft statement of purpose and need, which is attached to this report as Appendix A.

The Kennedy Bridge is fracture-critical in its original design. This means the bridge has a steel superstructure (the steel truss spans) with tension members, which are arranged in a manner whereby if one fails, the bridge could collapse because there is no backup or redundant structural support. The bridge's approach spans also contain pin and hanger details that are considered fracture-critical. A fracture-critical designation does not mean the bridge is unsafe. The fracture-critical features are inherent to the original design of the Kennedy Bridge and make the structure a higher priority for inspection and necessary maintenance.

Chapter 152 of the Minnesota Legislature 2008 Session Laws (Chapter 152)[2] directed MnDOT to establish a bridge improvement program with an emphasis on structurally deficient and fracture-critical bridges. The Kennedy Bridge is part of a Chapter 152 master bridge list, which identifies 172 bridges meeting the law's criteria. As such, it is to be under contract for rehabilitation or replacement by June 30, 2018. It is also anticipated that improvements to the Kennedy Bridge will be partially funded under the Chapter 152 program.

Chapter 152 and similar bridge management programs, including decision-making and risk-management systems, are often found within the context of Homeland Security and vulnerability. Such programs often place emphasis on fracture critical bridges; however, flooding and other risks are also considered. Minnesota's Office of Homeland Security and Emergency Management (2014), for example, notes MnDOT's responsibilities for 19,600 bridges, with

[2] Codified as Minn. Stat. 165.14.

4,668 bridges on trunk highways (including the Kennedy Bridge). State and federal bridge/asset management programs and advisories also provide evaluation and maintenance frameworks aimed at improving resilience and reducing vulnerability (FHWA, 2011).

The specific needs and considerations identified for the Kennedy Bridge, as explained in substantially more detail in Appendix A, are as follows:

- **Primary Need—**The required element for any alternative is to continue to provide a structurally sound crossing of the Red River of the North at this location. This primary need is the main motivation for the Kennedy Bridge Study. Near-term priority is required for Pier 6 and a long-term perspective is also needed for the entire bridge.
- **Secondary Needs—**Desirable; to be incorporated into reasonable alternatives:
  - Provide a reliable crossing for the traffic demands.
  - Improve bicycle/pedestrian access and connectivity at this location.
- **Other Considerations—**Important project planning factors related to context and anticipated project development alternatives and challenges:
  - **Regulatory requirements, including avoidance of adverse effects to historic resources—**The Kennedy Bridge itself is eligible as a historic structure. Other historic elements are also in the vicinity, as described in Section 2.
  - **Structural redundancy—**The two steel truss spans, as well as approach span details, are fracture-critical in their design (non-redundant). This factor requires sound decision-making for rehabilitation and maintenance approaches.
  - **Geotechnical conditions and river hydraulics—**All bridge design concepts should recognize the site characteristics of soil movement and floodway engineering challenges along the Red River.

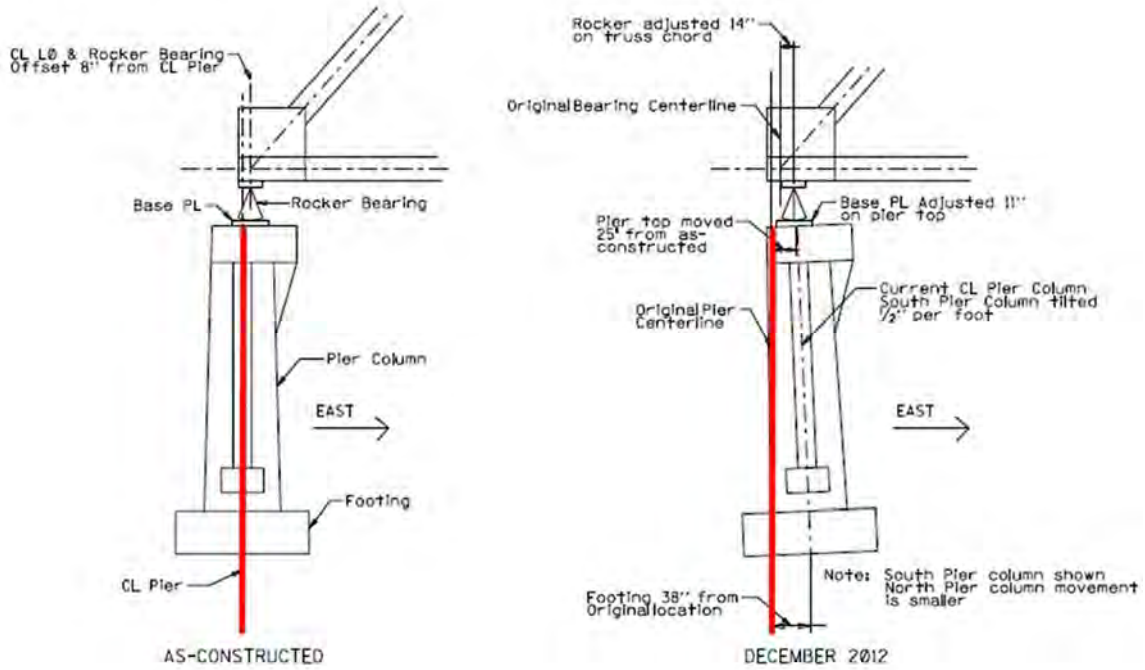
1.4 Bridge Rehabilitation and Project Development Alternatives

A variety of elements were evaluated for a potential bridge rehabilitation project, but Pier 6 (noted in Exhibit 1-2) was a special early focus. The first priority in the Planning Study was to address the movement of Pier 6, which supports the west end of the steel trusses and has gradually shifted due to Red River soil movements. While soil movement issues were anticipated in the original bridge design, the need to address the now substantially shifted position of Pier 6 was a major driver for technical work in the Planning Study (Exhibit 1-4). The key findings on Pier 6 are documented in the Technical Memorandum: *Pier 6 Movement Capacity* (MnDOT 2013a). The key findings included the following:

- The truss bearings have been adjusted periodically in response to pier movement, as anticipated in the original bridge design, cumulatively up to about 14 inches. Substantial additional adjustments could also be made.
- Additional tilt of the pier is a structural concern; therefore, actions should be taken to stabilize or replace the pier.

Monitoring of Pier 6 movement is ongoing and the Planning Study evaluated choices for pier rehabilitation/stabilization or replacement. Replacement of Pier 6 is the most promising course of action to address this most pressing need and will restore Pier 6 to the ideal location in a vertical, non-twisted position. The bearings would be reset to the original location at the ends of the trusses, allowing future adjustments for any additional soil movement.

EXHIBIT 1-4
Deflected Condition of Pier 6, South Side



Other expected bridge rehabilitation elements include blast cleaning and painting, abutment bearing reinforcement, and pier bent straightening. Recent inspection and testing has determined that chloride penetration and deterioration also makes replacement of the bridge deck a high priority (which requires replacement of the integral railings). See more information in Section 3.1 and in the Bridge Rehabilitation TM, which outline varied levels of bridge rehabilitation. Section 3 and the Bridge Replacement TM further support Planning Study conclusions by presenting possible Kennedy Bridge replacement concepts and comparing them to bridge rehabilitation.

SECTION 2

Kennedy Bridge Setting and Context

Section 2 of this Final Report provides background information and data to lay out the context for project development. This information includes the unique attributes of the Kennedy Bridge—its location, setting, role in the transportation system, and its historic and community context. Together, these factors drive the criteria for context-sensitive development of a transportation improvement project, as addressed at the conclusion of this section and in Section 3.

2.1 Kennedy Bridge Location and Area Overview

The U.S. 2 Kennedy Bridge is a border bridge connecting Grand Forks, ND and East Grand Forks, MN (Exhibit 2-1). The project location is about 300 miles northwest of the Minneapolis-St. Paul metropolitan area and about 80 miles north of the border cities of Fargo, ND and Moorhead, MN.

EXHIBIT 2-1
U.S. Highway 2 Kennedy Bridge Location Map

Combined, the greater Grand Forks/East Grand Forks metropolitan statistical area had a 2010 census population of 98,461.[1] The cities are named for and identified with the Red River of the North (Red River) and its fork with the Red Lake River, which joins the Red River just over 1 mile south (upstream) of the Kennedy Bridge. Spring flooding along the Red River has been important to the history and identity of the area, and has caused many Red River bridge closures. The best-known such event is the exceptional flood of 1997 (the worst since 1826), which caused vast and unprecedented damage throughout the region. Grand Forks and East Grand Forks were hit especially hard, with the flood causing the evacuation of most residents and contributing to significant fires in Grand Forks, in addition to the widespread flood damage.

By 2007, the communities and USACE had dedicated an innovative and adaptable flood protection system. The flood control strategy included major land use changes and dedication of the 2,200-acre Greater Grand Forks Greenway throughout the floodway. The Greenway includes a multi-use, paved recreational trail which loops more than 20 miles along both river banks. The Greenway is designated as a National Recreation Trail by the National Park Service.

The following subsections expand on these topics to provide an understanding of the Kennedy Bridge setting and context. The key subjects include traffic volumes crossing the Red River, the potential for bridge closures, the community setting and values, and the resultant criteria for development of a Kennedy Bridge improvement project.

2.2 Transportation and Environmental Setting

The Kennedy Bridge is located within a community setting that is both historic and forward-looking, bringing many related issues to the planning process. The issues identified and addressed through the Planning Study include projected traffic demand, the role of the Kennedy Bridge in the system of three Red River crossings, traffic diversions if bridges are closed (due to flooding or construction), and the surrounding area's context—its history, the community/recreational setting, and environmental features.

2.2.1 Kennedy Bridge Traffic and Other Red River Crossings

2.2.1.1 Level of Service—Performance Measure for Traffic

A traditional operational performance measure for roadways is level of service (LOS). A letter, A through F, is assigned to a roadway or intersection based on performance, with A being the best (no congestion) and F being the worst (gridlock). Because the context for traffic conditions can vary, MnDOT has not formally adopted a desirable LOS for operations of 4-lane urban arterials, such as U.S. 2 at the Kennedy Bridge. However, a mid-range LOS of C/D is often referenced as a reasonable standard, because such levels represent conditions with moderate and expected levels of congestion during peak periods, with little or no congestion the remainder of the day. Locally based transportation planning will also often reference LOS-based traffic performance goals, as does the GF-EGF MPO.[2]

[1] The metropolitan statistical area population includes 52,838 in the city of Grand Forks, ND and 8,601 in the city of East Grand Forks, MN (2010 census).

[2] For example, see: www.theforksmpo.org/Pages/2035LongRangeTranspPlan.htm and www.theforksmpo.org/PDFS/LRTPPerformanceBasedPlanning.pdf.

2.2.1.2 U.S. 2 Traffic Data

MnDOT collects traffic data along U.S. 2, east of East Grand Forks, via an Automatic Traffic Recorder (ATR). The ATR (ID 31) collects information on volumes and vehicle classification continuously to be used for Traffic Analysis. Based on a review of the MnDOT data, the peak hour volume (PHV) during the three highest hours varied from 11.5 to 13.8 percent of the average daily traffic (ADT). PHV is used in analysis of lane capacity in order to analyze for the highest volume hour of the day.

In addition to the ATR, analysis was completed by the GF-EGF MPO for the *GF/EGF Bridge Closure Management Study,* which looked at bridge closures and had peak hour volumes available relative to ADT (GF-EGF MPO 2007). With some structure closures across the Red River, the percentage of PHV compared to ADT was between 6 and 7 percent.

2.2.1.3 Kennedy Bridge Traffic Capacity and Level-of-Service

Highway Capacity Software (HCS) 2010 uses the Highway Capacity Manual as the basis for all capacity and level of service computations. This software was used to determine the existing (2011) LOS across the Kennedy Bridge and future LOS for the 2040 ADT provided by GF-EGF MPO and MnDOT.

Variables assumed and used in both existing and future traffic analysis are as follows:

- Base Free-Flow Speed = 45 miles per hour (mph)—The posted speed limit is 35 mph across the Kennedy Bridge; however, the multilane analysis in HCS does not model free-flow speeds below 45 mph. Therefore the model assumes that drivers travel at speeds over the speed limit when there is no congestion.
- Lane Width = 12 feet (11-foot lanes were not analyzed, but would result in a free flow speed reduction of 1.9 miles per hour)
- Lateral Clearance = 4 feet right, 2 feet left
- Divided roadway
- Access points per mile = 2
- Peak Hour Factor (PHF) = 0.92
- Number of lanes in each direction = 2
- Percent of ADT trucks and buses = 9 (MnDOT Structure Inventory Report)
- Level terrain
- Driver population is familiar with the location and most trips are local
- Directional traffic is split evenly (50/50) across the bridge

Using the data from the MnDOT ATR and bridge closure study, three scenarios were analyzed under each condition that had a larger effect on LOS than the assumed variables. Peak hour volumes of 6, 10, and 15 percent were analyzed to get a range of LOS for the Kennedy Bridge since no hourly volumes were available at the time of the analysis.

The 2011 ADT on the Kennedy Bridge was 22,500 vehicles per day.[3] Based on the analysis, the bridge is currently operating at LOS B to C (Table 2-1).

TABLE 2-1
Existing (2011) Kennedy Bridge Level of Service

PHV*	LOS	Flow Rate (Passenger Cars/Hour/Lane)	Density (Passenger Cars/Mile/Lane)
6%	A	383	8.5
10%	B	638	14.2
15%	C	958	21.3

*** In 2011, MnDOT measured PHV in a range from 11.5 percent to 13.8 percent of the ADT; GF-EGF MPO studies have indicated a PHV factor as low as 6 percent of ADT.**

The future (2040) ADT on the Kennedy Bridge is estimated by the GF-EGF MPO to be 29,910 vehicles per day (an increase of 33 percent from 2011). That forecast level assumes additional Red River crossings, as identified in the GF-EGF MPO's 2035 transportation plan, will not be constructed.

Based on the analysis of the forecast 2040 volumes, future traffic on the bridge would operate at LOS C to D (Table 2-2).

TABLE 2-2
Future (2040) Kennedy Bridge Level of Service

PHV*	LOS	Flow Rate (Passenger Cars/Hour/Lane)	Density (Passenger Cars/Mile/Lane)
6%	B	509	11.3
10%	C	849	18.9
15%	D	1273	28.3

*** In 2011, MnDOT measured PHV in a range from 11.5 percent to 13.8 percent of the ADT; GF-EGF MPO studies have indicated a PHV factor as low as 6 percent of ADT.**

The LOS analyses only addressed performance across the bridge. No signal timing or adjacent intersections were included in the analysis.

The LOS analysis accounted only for normal vehicular traffic scenarios and did not account for adjacent structure closings due to flooding. If vehicles are diverted from adjacent structures onto the Kennedy Bridge due to flooding, the LOS and vehicle flow rate will further decrease from the results above (see Section 2.2.1.4).

In general, the traffic analyses show that the Kennedy Bridge will continue to function well as a 4-lane roadway for more than 20 years, even with the forecast 33-percent increase in traffic volume. As another check, a sensitivity analysis was completed to determine if the above-assumed values caused a large variance in the LOS results. This checking showed that none of the assumptions, other than ADT percentage for PHV, will greatly affect the results (Tables 2-1 and 2-2 include a range of PHV percentages).

[3] MnDOT data for 2011. Based on input from the GF-EGF MPO, traffic counts by NDDOT immediately west of the bridge have often been substantially less than 22,500.

2.2.1.4 System of Local Bridges and Bridge Closure Impacts

The Kennedy Bridge is one of three local bridges across the Red River. Therefore, the local transportation context puts some emphasis on addressing the impacts of Red River bridge closures, which will occur because of floods or construction. As referenced above, the GF-EGF MPO 's *GF/EGF Bridge Closure Management Study* was a source for this Planning Study (GF-EGF MPO 2007). Based on that information and other data, MnDOT's team also completed an independent review, with emphasis on bridge closures due to floods.

The Red River sometimes rises to flood elevations, requiring bridge closures between the two cities for public safety reasons. As shown in Exhibit 2-2, there are three bridges across the Red River between the two cities (the Kennedy, Sorlie, and Point Bridges) and a fourth bridge in East Grand Forks across the Red Lake River (the Murray Bridge). The Kennedy Bridge is unique because it clears the river at a substantially higher elevation than the other bridges, and more than 8 feet higher than the Sorlie Bridge.

Order of Bridge Closings during Floods and Flood History

Exhibit 2-2 and Table 2-3 reference the order in which local bridges are closed in response to flooding and the established community action levels (river elevations and flood stages).[4] The National Oceanic and Atmospheric Administration (NOAA) has tracked historical crests for the Red River of the North at East Grand Forks for the past 132 years.

TABLE 2-3
Bridge Closure Order and River Elevation Action Levels (see also Exhibit 2-2)

Bridge (Closure Order)	**River Elevation (feet)/River Stage (feet)**
Point Bridge (1)	819.0/40.0
Sorlie Bridge (2)	822.5/43.5
Murray Bridge (3)	824.0/45.0
Kennedy Bridge (4)	831.0/52.0

Source: GF/EGF MPO (Notes: the 100-year flood recurrence = 832.4/52.7 feet elevation/stage; see also the footnote referenced to text above).

[4] There are conflicting data sources pertaining to bridge closure action levels, particularly for the Point Bridge. The GF-EGF MPO advised values are presented in Table 2-3 as conservative local action levels, with Point Bridge closure at 40.0 feet flood stage vs. 44.9 feet in other data sources. See: http://www.grandforksgov.com/gfgov/home.nsf/Pages/Flood+Fight and http://water.weather.gov/ahps2/hydrograph.php?wfo=fgf&gage=egfm5&hydro_type=2. All river elevations and flood stages noted in this report are in reference to the 1988 datum (North American Vertical Datum [NAVD] 88) and the gage location is at the Sorlie Bridge.

EXHIBIT 2-2
Red River Bridges and Flood Closing Order (see also Table 2-3)

Table 2-4 provides data on traffic demands across the Red River and historic flood data, which demonstrate the potential for bridge closure over the full period of record.

TABLE 2-4
Bridge Closure Potential Due to Floods (based on 132 years of record)

Bridge/Road Closure	**Percentage of Traffic Crossing the Red River**	**Number of Flood Closures (In 132 years of Record)**	**Percentage of Years (Average Probability in Record) ***
Point Bridge	16% (2 lanes)	29	22.0%
Sorlie Bridge	33% (2 lanes)	18	13.6%
Murray Bridge	NA (2 lanes)	14	10.6%
Kennedy Bridge	51% (4 lanes)	1	0.7%

*** Closure potential is based on river stage crests and includes years the bridges were not present to broadly compare probability (sources: http://water.weather.gov/ahps2/crests.php?wfo=fgf&gage=egfm5 and GF/EGF MPO 2007, Bridge Closure Management Study)**

The estimated probability of bridge closure in Table 2-4 is based on the long historic record of 132 years. Today's potential for bridge closure is likely greater based on review of flood stage history and considering increased urbanization and the flood controls completed in

2007 (which contain Red River flood flows at the bridges).[5] However, the relationships between local bridges will remain as shown previously.

Closures of the Kennedy Bridge and Impacts at Other Crossings

The Kennedy Bridge, with its 50-year history, is only known to have been forced closed by Red River flooding in 1997, which was an unprecedented event (see Exhibit 2-3). The 1997 flood crested at 54.35 feet flood stage—more than 4 feet higher than the next-highest floods on record (50.20 feet in 1897 and 49.86 feet in 2011). This was an extreme condition, 1.65 feet higher than the 100-year flood level (52.7 feet stage). During the peak of the 1997 flood, there were no opportunities for the traveling public to cross the Red River locally by roadway for approximately 1 week. Also, as proven by the 1997 flood, the approach roadway to the east includes a low segment still inside the new levee system (see Exhibit 2-3) that will be considered for adjustment.

EXHIBIT 2-3
U.S. 2 Kennedy Bridge During the 1997 Flood (Aerial View Looking North-Northwest—Source: Corps of Engineers)

Source: USACE (Note: the inundated approach roadway near the middle of the photo)

A Kennedy Bridge closure scenario far less extreme than the 1997 flood would be associated with major bridge construction, during which U.S. 2 traffic would be diverted to the Sorlie Bridge and partially to the Point-Murray Bridges to cross the Red River and Red Lake River. The GF-EGF MPO's *GF/EGF Bridge Closure Management Study* addressed a Kennedy Bridge closure scenario, as well as other scenarios (GF-EGF MPO 2007). With closure of only the Kennedy Bridge (for maintenance or other construction), the study found that traffic on the Sorlie Bridge would exceed the road's capacity by more than 40 percent. This scenario would also result in a high level of congestion in downtown Grand Forks/East Grand Forks, as the 2-lane local streets are not capable of carrying traffic as efficiently as the 4-lane U.S. Highway 2 urban arterial. The GF-EGF MPO's study found a substantially lower potential for traffic to divert to the Point and Murray bridges and few capacity concerns for those crossings and connections. With only the Kennedy Bridge closed, the Point Bridge would operate at about 28 percent of its potential capacity and the Murray Bridge at about 18 percent.

[5] Staff review of the Web-based data referenced in Table 2-4 noted that of the 10 highest flood stages on record (over 132 years), 6 of the 10 floods occurred during the last 20 years (1996, 1997, 2006, 2009, 2010, and 2011).

Kennedy Bridge Accommodation of Traffic with Other Bridges Closed

As previously noted, the Kennedy Bridge is the only available detour route if the two other Red River bridges are closed due to flooding. Under this scenario, traffic volumes on the Kennedy Bridge would be substantially increased. To understand potential impacts, Bridge Planning Study staff completed an analysis to further check the potential peak traffic demands and the highway capacity at the Kennedy Bridge.

Based on the referenced *Bridge Closure Management Study*, 20.5 percent of the Kennedy Bridge ADT may be present under peak hour conditions with the other bridges closed (GF-EGF MPO 2007). This is a substantially higher PHV than observed under normal network conditions, with all bridges open. MnDOT measured PHV in a range from 11.5 to 13.8 percent of ADT in 2011, and MPO studies have indicated a PHV factor as low as 6 percent of ADT. For this extreme-case analysis, 20.5 percent was applied to updated and future ADT values. HCS 2010 was used for the capacity and LOS analysis.

The 2011 ADT on the Kennedy Bridge was 22,500 vehicles per day. Based on 20.5 percent PHV, the PHV is 4,612 vehicles in both directions. The capacity analysis showed this volume would operate at a LOS D with 1,309 passenger cars/hour/lane and 29.1 passenger cars/mile/lane.

The 2040 ADT on the Kennedy Bridge is forecast to be 29,910 vehicles per day, assuming no major Red River bridge improvement actions or additions in the area. Based on 20.5 percent of that value, the PHV is 6,132 vehicles per hour in both directions. The capacity analysis showed this volume would operate at a LOS E with 1,741 passenger cars/hour/lane and 40.2 passenger cars/mile/lane.

Because running all of the area's river-crossing traffic on the Kennedy Bridge would be a temporary scenario, expected only under extreme conditions, a level of service of D to E is considered acceptable. The forecasted moderate to heavy congestion on the Kennedy Bridge during peak periods is within the range of expected and tolerable congestion levels, with less congestion the remainder of the day.

The capacity analyses completed for this study did not address congestion at surrounding signals and intersections. The metropolitan area's traffic management planning efforts indicate that conditions would also be optimized through signal retiming, other temporary intersection traffic control modifications, and traveler information efforts. Therefore, while running all river-crossing traffic on the Kennedy Bridge is not ideal, traffic can still move reasonably between the two cities with all four lanes open on U.S. Highway 2 and the Kennedy Bridge open.

Long-term Transportation Plans and Possible New Bridge Locations

The GF-EGF MPO has established long-term plans for locations of two additional Red River bridges. The growth trend in the metropolitan area is toward the south and the new bridge crossings identified in long-term plans are at 32nd Avenue South and Merrifield Road (see Exhibit 2-4). These new bridges are not funded projects and, therefore, are not part of the committed future transportation system and have not been factored into the traffic forecasting or traffic impact issues previously discussed.

EXHIBIT 2-4
Area Map Showing Red River Bridge Locations—Existing and Potential Long-term Additions (base map: Google)

2.2.1.5 Crash History and Highway Safety Observations

Crash data was pulled from MnDOT's Crash Mapping Analysis Tool (MnCMAT) and ArcGIS data from the NDDOT. Crash data was reviewed for a 5-year period between 2008 and 2012. The results from 2008 through 2012 showed a total of 19 crashes in the segment along U.S. 2, from 1st Street North (Grand Forks) and 4th Street Northwest (East Grand Forks), a distance of 0.42 mile, including the Kennedy Bridge. Of these, there were no fatal crashes, but two crashes were severe, with incapacitating injuries. There were also 3 injury, 2 possible injury, and 12 property damage crashes in the 5-year period. The crash rate for this segment is 1.1 crashes per million vehicle miles traveled.

The two severe crashes were classified as a rear end and a sideswipe same direction crash, respectively. The severe crash rate for this segment is 0.12 crash per million vehicle miles traveled. Minnesota publishes average crash rates throughout the state based on road types in "Section Green Sheets." In the 2011 update of the Green Sheets, the statewide average crash rate for a similar urban, 4-lane divided roadway was 3.4 crashes per million vehicle miles traveled (including along the roadway and at intersections). Comparing the crash rates along the Kennedy Bridge crossing (1.1) to the statewide average for similar roadways (3.4), there is no evidence of a particular crash problem in the Kennedy Bridge segment.

Stakeholder input on safety obtained during the Bridge Planning Study often highlighted the following safety issues or local preferences:

- **Median on the Kennedy Bridge—**There were favorable comments on the role of a center median on the bridge, specifically that the median has had a role in preventing crashes during icy winter conditions.
- **Ramps Connecting to 4th Street Northwest in East Grand Forks—**Stakeholders sometimes referenced safety concerns associated with the ramp connections, located just east of the Kennedy Bridge. Logically, the west-bound on ramp is most likely to present safety concerns because of the merge it presents with U.S. Highway 2.
- **Bicycles and Pedestrians—**The Bridge Planning Study has addressed accommodations on the Kennedy Bridge for bicycles and pedestrians, with considerably more information provided in the following subsections and in Section 3. Stakeholders have often cited safety as an important factor to consider for bikes/pedestrians—both as justification for proposed improvements and for inclusion in project evaluation criteria.

2.2.2 Environmental and Historic Features

Exhibit 2-5 is an environmental overview map for the vicinity of the Kennedy Bridge. The principal environmental features of this area are summarized in the following subsections.

2.2.2.1 Red River and Floodway/Greenway

The Red River, which flows from south to north through the area, creates a dynamic and challenging environment for maintenance of structures. The Red River itself is the main remaining drainage across the flat lakebed of the enormous, ancient Lake Agassiz, a glacial lake, which drained about 9,500 years ago. Therefore, the Red River is located within a vast and mainly flat basin covering more than 111,000 square miles. These parameters and natural conditions result in a wide floodplain and contribute to the risks from periodic major floods. Additionally, the soils along the Red River are known to creep (move slowly, imperceptibly), generally from the river banks toward the middle of the river (see the reference to Pier 6 movement in Section 1.4). In fact, the west riverbank area (including the area around Pier 6 and to the south) was identified by USACE as a "landslide risk area" to prevent any inappropriate construction activities or designs, temporary or permanent. The City of Grand Forks has also noted a storm sewer outfall in this area (immediately southwest of the bridge), which includes a membrane and riprap covering as bank stabilization features.[6]

[6] City of Grand Forks Engineering, input on the Kennedy Bridge Planning Study (re. landslide risk and the storm sewer).

Locally, the Red River floodway was engineered in the aftermath of the 1997 flood. Flood-damaged homes throughout entire neighborhoods were demolished after the flood, as evident through comparison of the Exhibit 2-3 photograph and the open space shown on Exhibit 2-5 (note also the legacy parcel boundaries on Exhibit 2-5). Floodway areas inside the flood protection system (levees and walls) are now publicly owned open space, designated as the Red River Greenway.

The Greenway, completed in 2007, provides about 2,200 acres of open space and more than 20 miles of multi-purpose paved trails within the floodway on both sides of the Red River. The Greenway is also designated as a National Recreation Trail by the National Park Service. In addition to trails, the recreational features along the Greenway include two golf courses, three disc golf courses, shore fishing sites, and a Minnesota State Park Campground—the Sherlock Park Campground, as shown on Exhibit 2-5.

The Sherlock Park Campground, which is configured around the legacy residential streets and lots, is part of the Minnesota Red River State Recreation Area—essentially, the Greenway space on the Minnesota side of the Red River. In addition to the developed and full-service camping area located south of U.S. 2, the State Campground provides for primitive camping just north of U.S. 2.

2.2.2.2 Section 4(f) Applicability and Section 106 Historic Resources

The publicly owned recreation lands around the Kennedy Bridge (comprising the Greenway) are noteworthy for project development context in that this land will likely meet the definition of a Section 4(f) resource. Section 4(f) of the 1966 Department of Transportation (DOT) Act (49 USC 303, 23 USC 138)[7] provides protection for publicly owned parks, recreation areas, historic sites (public or private), and wildlife refuges from conversion to a transportation use. Section 4(f) applies only if the following criteria are met:

- Federal transportation funds are anticipated or an action is being taken that requires approvals by a federal transportation agency.
- The property is publicly owned and open for public recreation, or meets historic property criteria if privately or publicly owned.

The Section 4(f) evaluation process requires that any impacts from direct use of a publicly owned park, recreation area, historic site, wildlife, or waterfowl refuge for highway purposes be evaluated in context with the proposed highway construction/reconstruction activity. Finally, any such use of the eligible resource can only be allowed if there is no feasible and prudent alternative.

Historic properties are also protected by Section 106 of the National Historic Preservation Act of 1966 as amended (16 USC 470). Both Section 4(f) and Section 106 define "historic" properties as those listed on, or eligible for, the NRHP.[9] Section 4(f) generally defers to the Section 106 review process for identifying historic properties and assessing the potential effect of an undertaking on the properties.

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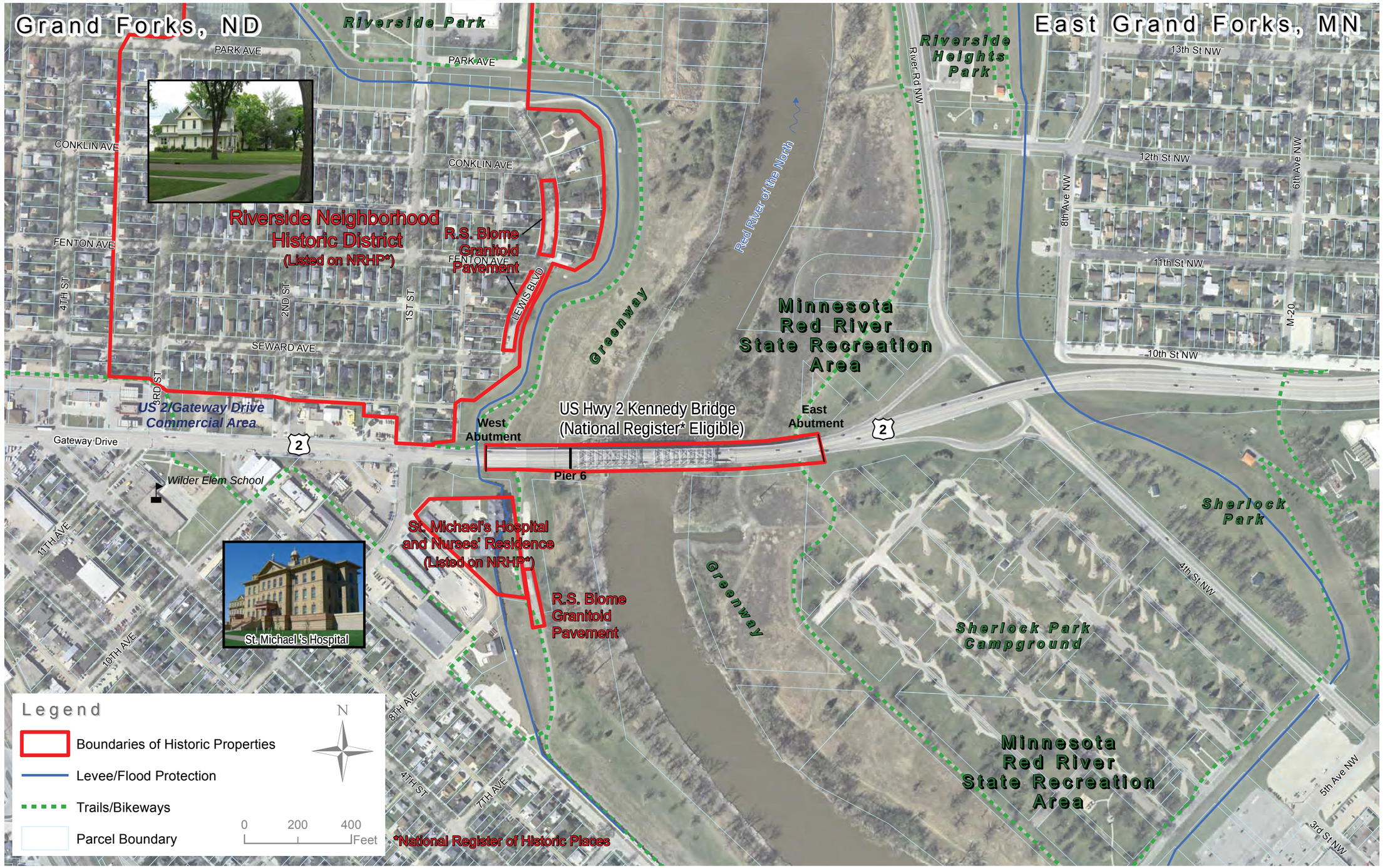
[7] In January 1983, as part of an overall reorganization of the DOT Act, Section 4(f) was amended and codified in 49 U.S.C, Section 303. However, the regulation is more commonly known as "Section 4(f)."

[9] To be eligible for the NRHP, properties must typically be at least 50 years old and must also satisfy at least one of four criteria associated with prehistory or history.



Kennedy Bridge Planning Study
East Grand Forks, MN | Grand Forks, ND

TBG011514142833MKE 2-5_KBPS_Environmental_Overview_v2 04.21.14 sls

Exhibit 2-5
Environmental Overview Map

The Kennedy Bridge area includes several historic resources, identified on Exhibits 2-5 and 2-6. The Kennedy Bridge itself, built in 1963, is eligible for listing on the NRHP based on its significance to engineering (the two, 279-foot-long steel Parker truss spans) and transportation (the river crossing's role in economic development). Other historic properties in the vicinity of the bridge include the following:

- St. Michael's Hospital and Nurses' Residence (now adapted for residential use as Riverside Manor), located south of the bridge approach in Grand Forks
- The Riverside Neighborhood Historic District, a residential area located north of the Grand Forks bridge approach
- Historic "granitoid" pavement, present in some locations around the Grand Forks bridge approach area (see Exhibits 2-5 and 2-6)

Additional information on Kennedy Bridge character-defining features and the significance of the other historic resources is included in Section 3 of the TM: *Bridge Rehabilitation Alternatives*.

2.2.2.3 Other Environmental Resources and Regulations

The Red River floodway, the river's natural environment, and other characteristics must also be taken into account in planning to maintain the river crossing at the Kennedy Bridge. Some of the key applicable environmental regulations include the following:

- **Presidential Executive Order 11988, Floodplain Management (and related)—**The applicability of this Executive Order, and closely related federal, state, and local rules, will depend on the extent to which the proposed project would encroach on the Red River floodway. Any actions taken within the 100-year floodplain can be considered encroachments. But actions considered most challenging to completing project reviews and approvals would be those in the floodplain that involve the addition of new structures, addition of fill, or the replacement of existing structures. Related technical considerations include any adverse effects on natural and beneficial floodplain values, any increased risk of flooding, and the development of any features considered incompatible with the floodplain. In addition, any modifications to the area's engineered flood protection system require approval by USACE and both cities. Because the system is built to protect against a flood event greater than 100-year frequency, hydraulic impacts greater than a 100-year event need to be evaluated. At a minimum, the hydraulic effects at the 250- and 500-year events, which are both below the top of levee elevations, should be evaluated in addition to the 100-year event. Any negative impact to the current level of flood protection would need to be mitigated.[10]

[10] City of Grand Forks Engineering, input on the Kennedy Bridge Planning Study (re. modification approval requirements).

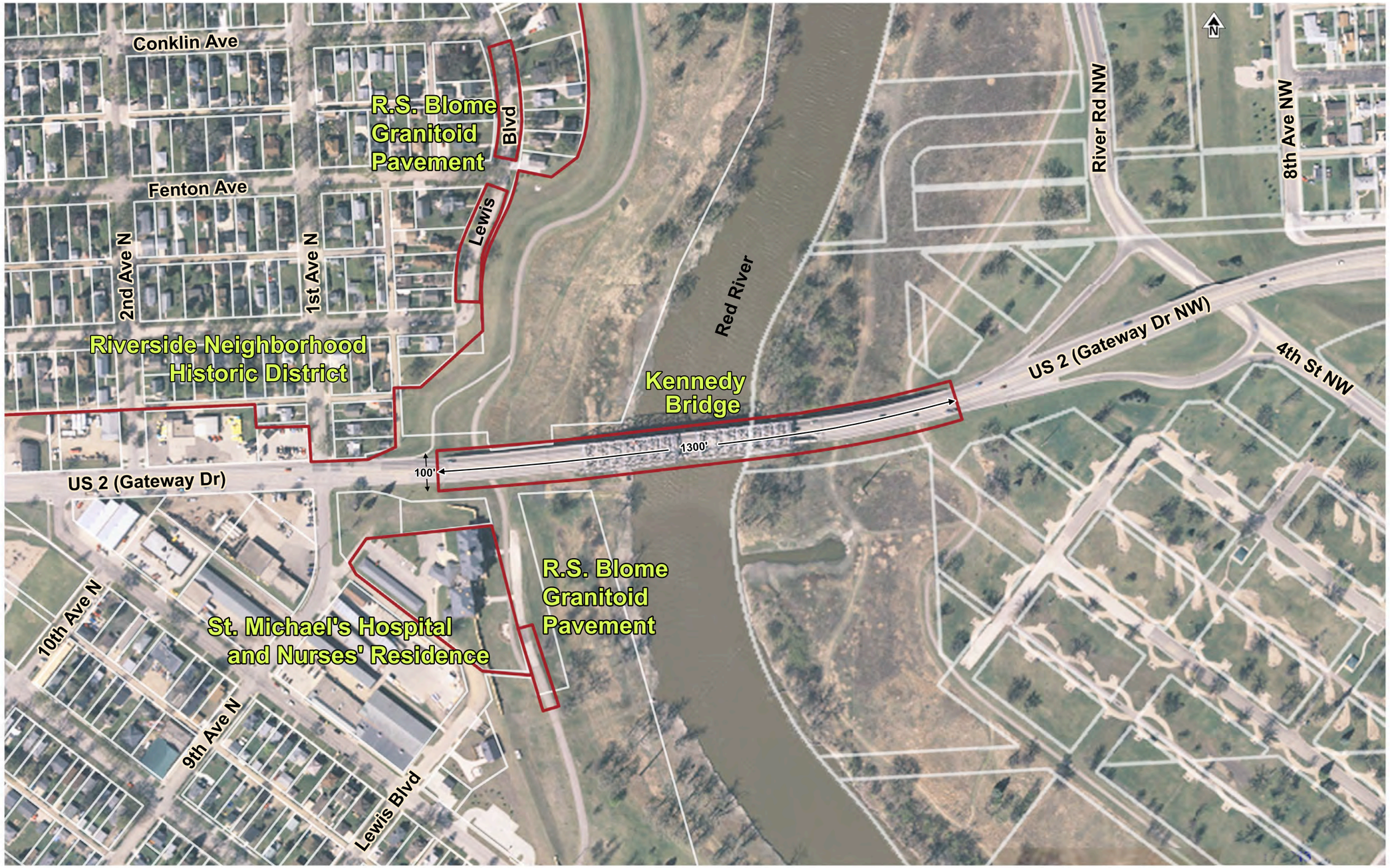


Exhibit 2-6
Historic Properties

- **Federal Clean Water Act (CWA), 33 U.S.C. 1344, Section 404 and 404(b)(1) guidelines and CWA Section 401 Water Quality Certification—**The referenced regulation and sections concern water quality and the regulation and protection of wetlands and aquatic resources. While the Planning Study did not include wetland delineation or other detailed environmental data reviews, wetland characteristics are often observed along the Red River. Similar to the floodplain management issues, greater potential for impacts would be associated with actions that involve addition of new structures, addition of fill, or the replacement of existing structures.
- **Fish and Wildlife Coordination Act (16 U.S.C. 661-666)—**This regulation applies to actions that may involve impoundment of water (surface area of 10 acres or more), channel diversion, channel deepening, or other control or modification of a stream or other body of water. Fisheries may also need to be addressed if the work to be done will impact the river.

The Bridge Planning Study has outlined issues of project need, context, and the potential regulatory issues. Many of these factors are discussed in the previous subsections, with general goals and objectives listed in Section 1.2. Based on these factors, and the ability to cost-effectively address needs, bridge rehabilitation is considered the priority action. In general, the bridge rehabilitation concepts identified through the study were found to sufficiently address needs, while avoiding or minimizing adverse impacts. However, additional design studies and regulatory review will be required to render a project development decision.

More information about project development choices is provided in Section 3, including comparisons between bridge rehabilitation and bridge replacement.

2.3 Bicycle and Pedestrian Considerations

A summary of context for the Kennedy Bridge Study would be incomplete without special attention to bicycle and pedestrian movements. As noted in Section 1.3, the Kennedy Bridge does not accommodate pedestrians and bicycles and, in fact, includes a posted prohibition (westbound only at the east abutment). But the importance of the bridge as a Red River crossing, the popularity of the Greenway trails, and observed demand raised the need to seriously consider improvements as part of any bridge rehabilitation project. Minnesota legislation (passed 2010) requires that all bridge projects funded under Chapter 152 in fiscal year 2012 or later include bicycle and pedestrian accommodations if both sides of the bridge are located within a municipality or the bridge links a pedestrian way, shared-use path, trail, or scenic bikeway. As context for a potential Kennedy Bridge improvement, the comprehensive multi-use trail system can be seen in Exhibit 2-5 (note the presence of the Greenway trails and other multi-use trails).[11] Federal legislation (23 USC Section 217e) provides a similar requirement, for any "bridge deck being replaced or rehabilitated with Federal financial participation and on a highway on which bicycles are permitted to operate at each end." The cited section advises that if (the FHWA) "determines that the safe accommodation of bicycles can be provided at reasonable cost as part of such replacement or rehabilitation, then such bridge shall be so replaced or rehabilitated as to provide such safe accommodations."

[11] The main Greenway trails are typically 14 feet wide.

Because the Kennedy Bridge deck width is constrained by the steel truss, the scope for the Bridge Planning Study originally emphasized the idea of attaching a new structure to the outside of the truss spans. The intent was to check the structural feasibility of this concept. The Study has found that, while this concept is technically feasible, it would also add substantially to rehabilitation project costs and the structure's complexity. Section 3 provides more detailed discussion of these factors, which include the issues of bridge inspection/maintenance and whether an external structure would adversely affect the historic character of the Kennedy Bridge.

The Bridge Planning Study also had the benefit of working with the SAC, and obtaining input from the general public (see Section 1). These efforts confirmed the importance of addressing "bike/ped" accommodations on the Kennedy Bridge. Local planning staff also completed efforts to observe bike/ped demands during the summer of 2013. On portions of June 18, 19, and 22, 2013 (Tuesday, Wednesday, and Saturday), GF-EGF MPO staff observed activity on a Greenway trail under the Kennedy Bridge approach spans and the use of the Kennedy Bridge itself for crossing the Red River. The relevant data are summarized in Table 2-5 and Greenway trail summer usage is shown in Exhibit 2-7.

TABLE 2-5
Limited-Time Observations of Bicycle and Pedestrian Activity at the Kennedy Bridge

	Tuesday, June 18*		**Wednesday, June 19***		**Saturday, June 22***	
Location	**Walking, Running, Blading**	**Biking**	**Walking, Running, Blading**	**Biking**	**Walking, Running, Blading**	**Biking**
Greenway Trail Under Bridge	66	128	72	94	24	70
Kennedy Bridge Crossings	2	3	6	1	0	1

*** The observation periods for each date and location lasted 2 hours.**

EXHIBIT 2-7
Red River Greenway Trails at the Kennedy Bridge

During the reported limited time periods on the three dates (2-hour periods; not full days), a total of more than 450 trail users, including 292 bicyclists, were observed passing under the approach spans. During similar partial-day surveys on the same 3 dates, a total of 13 users

were observed crossing the river on the Kennedy Bridge (5 bicyclists and 8 pedestrians). These observations show that the Kennedy Bridge is located in the midst of a popular multi-use trail system (along the Greenway) and that non-motorized travelers will cross the Kennedy Bridge, even without adequate accommodation and against the posted westbound prohibition (at the east abutment).

Based on these planning inputs and concerns identified for attaching a new trail structure external to the truss, the team also developed concepts to adjust the roadway cross section on the existing bridge, constrained by the width available inside the steel truss spans. Section 3 shows and compares the options for accommodation of bicycles and pedestrians on the Kennedy Bridge, both internal and external to the truss.

2.4 Issues and Criteria for Project Development

The background and context for the Kennedy Bridge, as laid out in Sections 1 and 2, allowed the Planning Study team to understand the key issues and the evaluation criteria for making project development decisions. Section 3 presents the framework for development of a transportation infrastructure improvement project—namely, the options of bridge rehabilitation or bridge replacement (each of which also present a range of choices, or variations).

The objectives for the Planning Study included development of context-sensitive solutions—approaches to address needs while packaging improvements to fit well into the community. Section 3 identifies and compares the principal project development options, which are ultimately evaluated according to the following context-sensitive criteria:

- **Bridge Capital Cost and Project Funding—**Considers project construction costs and funding levels to implement various bridge rehabilitation and bridge replacement concepts.
- **Structural Performance and Maintenance—**This criterion considers the ability of the structure to perform over short-term and long-term periods, up to several decades into the future. It also considers the ability to easily maintain the bridge.
- **Construction Period and Traffic Impacts—**As outlined in Section 2.2.1, keeping the system of bridges to cross the Red River open is important to maintaining reasonable traffic movement. This is especially true for the Kennedy Bridge, which typically carries more than 50 percent of the area's total traffic demand across the river. Therefore, construction staging and duration are important evaluation issues for the Kennedy Bridge, even if complete bridge closure is not required.

- **Historic Preservation (Section 106) Review—**Any reconstruction of the Kennedy Bridge, whether rehabilitation or replacement, would require reviews to determine if the proposed modifications would cause adverse effects to the historic bridge or to other historic resources. In general, greater levels of modification will require more rigorous reviews and may present risks for unavoidable adverse effects (for example, complete bridge replacement, including removal of the existing bridge, would result in a Section 106 adverse effect).[12]

- **Long-Term Traffic and Bike/Ped Function—**This criterion considers the ability of Kennedy Bridge design options to safely and effectively serve vehicular traffic and accommodate bicycles and pedestrians. Other considerations for Kennedy Bridge improvements include:

  — **System of Shared Use Trails—**The Red River Greenway and other trails in the area are typically shared-use paved paths that are 10-14 feet wide.

  — **Trail Connection along the South Side of the Kennedy Bridge—**Local input received during the Planning Study, and previous local plans, most often support a shared-use path along the south side of a new or rehabilitated Kennedy Bridge. The main justification for the south side of the bridge is to create an "inner loop" in the trail system, looking toward the area's central business district to the south. As the Planning Study concludes, it must be noted that this perceived preference is not a design recommendation or decision. Any additional design studies to address trail connections across the Kennedy Bridge will consider options along both the north and south sides (see also Section 3.1.2.3).

- **Project Development Risks and Approval Process—**Finally, the various design options would require different levels of design and environmental review in the project development process. These reviews would focus in particular on design and construction feasibility and risks and any adverse environmental impacts.

As noted previously, rehabilitation of the Kennedy Bridge is considered the priority action. Section 3 concludes this report by providing substantially more information about project development choices and next steps.

[12] The Section 106 review process is an important factor behind this Planning Study, with background in the *Management Plan for Historic Bridges in Minnesota* (MnDOT 2006). That plan provided information and guidance on the management and long-term preservation of historic bridges in Minnesota. It also helped delineate project review procedures, principally a detailed review process in which professional engineers are teamed professional historians. This Planning Study has outlined anticipated actions and Section 106 review issues for the Kennedy Bridge; however, additional design studies and regulatory reviews will be required to render a project development decision.

SECTION 3

Project Development and Study Findings

This concluding section of the Final Report summarizes the engineering concepts considered for long-term maintenance of the U.S. Highway 2 Red River crossing. More detailed engineering information is included in the two Technical Memoranda: *Bridge Rehabilitation Alternatives* and *Bridge Replacement Options* (MnDOT 2013b and MnDOT 2013c). The engineering concepts provide a range of options for bridge rehabilitation and bridge replacement. Therefore, the Planning Study has refined the options to determine the most promising choices available for project development.

3.1 Project Development Alternatives

The Planning Study decision-making framework emphasized the following objectives:

- **Address Pier 6 movement and position**—Determine the need/ability to further adjust Pier 6 for movement; consider monitoring for more movement and longer-term risks/mitigations
- **Address overall bridge condition and secondary needs**—Address overall bridge maintenance, considering all of the other bridge components and secondary needs

The principal choices, or project development alternatives, at the study's conclusion include the following:

- No Action (baseline alternative)
- Bridge Rehabilitation (the priority action)
- Bridge Replacement (as comparison to bridge rehabilitation)

Each alternative is summarized in the subsections below and compared in Section 3.2. As previously noted, rehabilitation of the Kennedy Bridge is considered the priority action.

3.1.1 No Action Alternative

3.1.1.1 Description

The No Action (or no build) Alternative would involve no substantial bridge rehabilitation actions to maintain the bridge long-term. However, this scenario would include continued inspection, monitoring, and minor ongoing maintenance of the Kennedy Bridge. The No Action Alternative is a required consideration for environmental reviews and approvals; but it is typically considered the baseline, for comparison to the action/build alternatives. The characteristics and performance of this alternative include the following:

- Cost—Lowest, includes only routine maintenance
- Pier 6—Potential for more movement and would not be addressed proactively
- Overall Bridge Condition—The deck and other elements would continue to deteriorate, requiring increased maintenance efforts

- Bicycle/Pedestrian Function—Would remain as it is today, discouraging these uses and lacking in safety
- Risks—The No Action Alternative could result in future load restrictions and partial or complete bridge closures

3.1.1.2 Project Development Process

The No Action Alternative does not require any further technical reviews or approvals. However, it would fail to address the identified needs and would not be compliant with Minnesota's Chapter 152 legislation (Minnesota Statute 165.14), which calls for having the bridge under contract for rehabilitation or replacement by June 30, 2018 (or sooner). See also Section 1.3 and Appendix A.

3.1.2 Bridge Rehabilitation Alternative, Recommendations, and Options

3.1.2.1 Description

A detailed description of the Bridge Rehabilitation Alternative, including sub-alternatives, is provided in the TM: *Bridge Rehabilitation Alternatives* (MnDOT 2013b). The bridge rehabilitation elements considered in the TM are noted in Exhibit 3-1. A complete bridge rehabilitation project includes measures to address the major issue of displacement and tilt in Pier 6 (described in Section 1.4) as well other important elements identified and detailed in the Bridge Rehabilitation TM and noted in Exhibit 3-1.

EXHIBIT 3-1
Kennedy Bridge Rehabilitation Elements

3.1.2.2 Bridge Rehabilitation Sub-Alternatives and Findings/Recommendations

The Bridge Rehabilitation TM identifies four bridge rehabilitation sub-alternatives, which are structured to compare varied levels of investment. The rehabilitation sub-alternatives are described and compared in Table 3-1. The TM also includes more detailed discussion supporting the evaluation in Table 3-1.[1]

TABLE 3-1
Description and Evaluation of Bridge Rehabilitation Sub-Alternatives

		Alt. 1	Alt. 2A	Alt. 2B	Alt. 2C
	Component	**Minimal Rehab (1)**	**Moderate Rehab (2)**	**Moderate Rehab - Add Shared-Use Path (3)**	
Description	Underpin Pier 6 (stabilize foundation)	X			
	Replace Pier 6 (replace foundation)		X	X	X
	Protect Trusses from Corrosion	X	X	X	X
	Adjust Approach Span Bent Columns	X	X	X	X
	Reinforce Abutment Bearings	X	X	X	X
	Monitor Pins and Hangers	X			
	Replace Pins and Hangers		X	X	X
	Maintain Deck and Railings	X			
	Replace Deck and Railings		X	X	X
	Add Shared-Use Path (External to Truss)			X	
	Add Shared-Use Path (Internal to Truss)				X
Evaluation	Primary Needs Addressed	Poor	Good	Moderate	Good
	Level of Service for Traffic	Good	Good	Good	Good
	Bike/Pedestrian Accommodation & Safety	Poor	Poor	Very Good	Good
	Construction Impact on Traffic	Low	Moderate	Moderate	Moderate
	Future Maintenance & Inspection	Very Poor	Good	Poor	Good
	Risk of Section 106 Adverse Effect	Low	Moderate	High	Moderate
	Construction Cost (bridge elements only)	$3.8M	$13.4M	$16.4-$17.4M	$13.5M

Notes:

1. Deck replacement is needed to address the primary long-term need, to maintain the vehicular river crossing.

2. Moderate bridge rehabilitation is superior to serve the primary long-term need. A major bridge rehabilitation, involving replacement or addition of steel truss members, is not considered necessary.

3. Sub-alt. 2C is technically the same as 2A except 2C adds the bike/pedestrian accommodations internal to the truss.

[1] The alternatives developed for the Bridge Planning Study were structured to illustrate the possible range of rehabilitation actions to be taken. The Planning Study does not identify all details to develop a preferred alternative, nor does it finalize any regulatory determinations. Final project decisions and determinations will require support from additional detailed design studies.

The Kennedy Bridge rehabilitation sub-alternatives provide two main choices for project development, with options provided in sub-alternative 2 for accommodation of bikes and pedestrians:

1. **Minimal Bridge Rehabilitation—**The Bridge Study team identified a minimal level of bridge rehabilitation as a sub-alternative for comparison to more complete bridge rehabilitation schemes. Most importantly, this level of rehabilitation does not include deck replacement, which results in substantial short-term costs savings. However, the poor condition of the existing original bridge deck is a high priority, based on recent inspections and testing, which found evidence of chloride penetration and deterioration, including under-deck delamination of concrete (with concrete pieces sometimes falling from the underside of the deck). A modern conventional deck has a projected life of 50 years before significant rehabilitation is expected to be necessary. The existing deck, built to lesser 1960 standards, was repaired with an overlay to extend service; however, it now is reaching the end of its practical service life.

2. **Moderate Bridge Rehabilitation—**The moderate bridge rehabilitation approach developed in this study is far superior to serve the primary long-term needs. As shown in Table 3-1, this approach includes the replacement of Pier 6 (including the foundation), the pin and hanger assemblies in the approach spans, and the deck (with integral railing).[2] The variations on the moderate rehabilitation package include the following sub-alternatives:

   - **2A—**Baseline for moderate bridge rehabilitation, including only the long-term rehabilitation elements and no shared-use path for bicycles/pedestrians.
   - **2B—**The moderate bridge rehabilitation elements with a shared-use path, provided external to the truss spans, on a separate structure attached to the truss spans.
   - **2C—**The moderate bridge rehabilitation elements with a shared-use path, provided internal to the truss spans, through adjustment of the roadway cross section.

From among the choices, the design team recommends serious consideration of 2B and 2C, the moderate rehabilitation choices that provide a shared-use path. Sub-alternative 1, minimal rehabilitation, would stabilize Pier 6 in its displaced position and would not include replacement of the deteriorating deck. Because of these characteristics, the minimal rehabilitation package would fail to address the primary long-term need of providing a structurally sound river crossing. Considerable delays in implementing a deck replacement, as implied by sub-alternative 1, should only be considered if funds are highly constrained.

Sub-alternative 2A provides all of the bridge rehabilitation elements required to address the primary need for action. The added elements include complete replacement of Pier 6 and replacement of the deck, which also serves to strengthen truss spans. [3] However, the 2A bridge rehabilitation package by itself would not address the important secondary need

[2] As clarification, a "major" bridge rehabilitation is not considered necessary. This would involve replacement or addition of steel truss members, which are in good condition for the Kennedy Bridge and do not warrant replacement. Additionally, the bridge engineering team determined that adding members to provide structural redundancy is not a prudent level of action.

[3] The replacement of Pier 6 is feasible with minimal additional cost compared to underpinning and stabilizing the pier in its current displaced and tilted position, with little or no difference in traffic impacts during construction.

of providing for bicycles and pedestrians. Furthermore, the Bridge Study's development of sub-alternative 2C showed that bicycle/pedestrian improvements can be provided with no significant difference in technical approach and cost (see next section). Therefore, sub-alternatives 2B and 2C (both providing for a shared-use path on the bridge) are recommended for further comparison at the conclusion of the Bridge Planning Study. The sub-alternatives help outline MnDOT's current priorities for development of a bridge rehabilitation project. Additional design studies and a formal environmental review of a proposed project may bring changes to these/other alternatives to determine a preferred alternative.

3.1.2.3 Design Options for a Bicycle/Pedestrian Trail on the Kennedy Bridge

The Bridge Planning Study was scoped to focus on providing a Bicycle/Pedestrian trail improvement as a modification to the Kennedy Bridge (U.S. Highway 2). While a completely separate bicycle/pedestrian bridge nearby could be a viable project, it was outside the scope of the Bridge Study to address location and structure options for a second, separate, bridge. The options considered in the Planning Study, and the background for developing and evaluating the options, are discussed in the following subsections.

New Bike/Pedestrian Structure (*External* Shared-Use Path)—Because the Kennedy Bridge deck width is constrained by the steel truss, the first concept evaluated was to attach a new structure to the outside of the truss spans. The new structure would continue immediately next to the approach spans and the profile would be set at/above the Kennedy Bridge low-beam clearance profile.[4] While this concept is technically feasible, it would add substantially to rehabilitation project costs and the structure's complexity. Some important considerations would be whether bridge inspections and maintenance can be completed effectively with the added structure and whether it would adversely affect the historic character of the Kennedy Bridge, or other historic properties (see more in Sections 3.1.2.4 and Section 3.2). Exhibits 3-2 through 3-4 provide cross sectional and 3D perspective views of the design concept for an external path or trail.

Exhibit 3-5 provides a plan view of the external structure, shown along the south side of the Kennedy Bridge, per local preference; however, additional design studies must not preclude addressing options along the north side per Section 2.4. See Section 3.2 for a comparison of this design with other project development alternatives.

Adjusted Roadway Cross Section Inside the Truss (*Internal* Shared-Use Path)—Given the above-referenced challenges to adding a new structure for a shared-use path, the design team also developed concepts to adjust the roadway cross section on the existing bridge, constrained by the width inside the steel truss spans (67 feet–4 inches). With a posted speed limit of 35 mph, the traffic engineering has the potential to be adjusted to accommodate bicycles and pedestrians without widening. The relatively low posted speed, and traffic volumes that are often well below capacity, allow bicycles and pedestrians to be accommodated adjacent to vehicular traffic without physical separation. Overall, this approach can provide a reasonable environment for bicyclists and pedestrians, within norms based on similar urban arterials and bridges.

[4] Supporting the separate path structure with Kennedy Bridge approach spans is not feasible. This warrants completely separate new bridge spans next to the approaches, with spacing provided for inspection/maintenance.

EXHIBIT 3-2
Cross Section of Separated Bicycle/Pedestrian Path (Trail) at Approach Spans

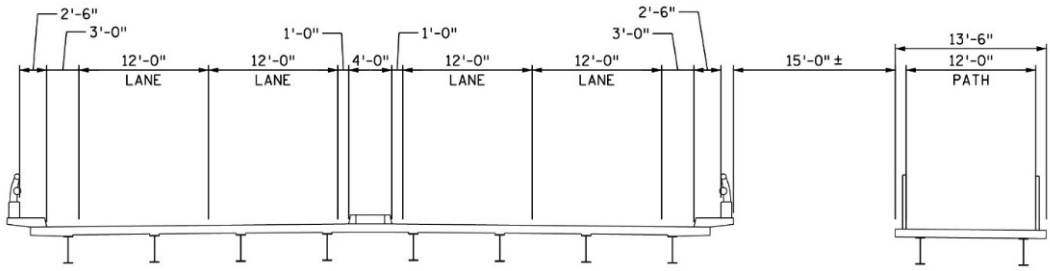


EXHIBIT 3-3
Cross Section of Separated and Attached Bicycle/Pedestrian Path (Trail) at Truss Spans

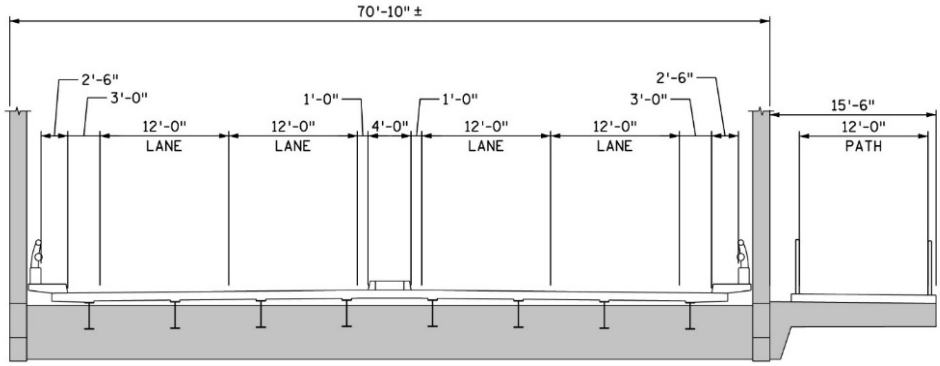
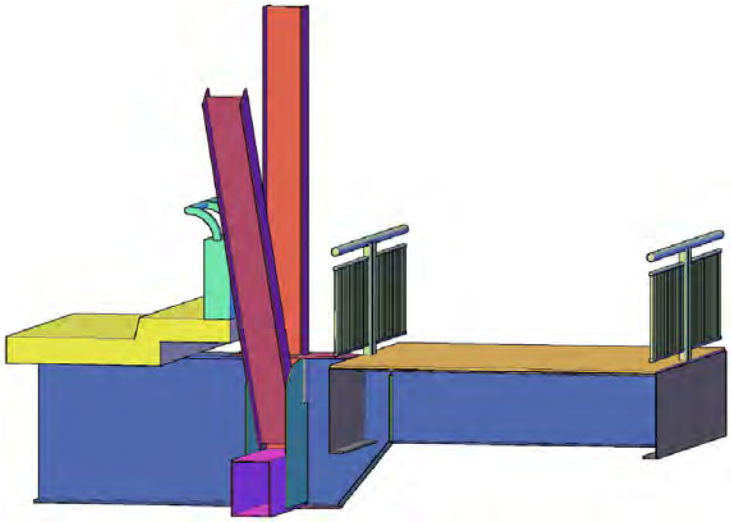


EXHIBIT 3-4
3D Model Perspective of Bicycle/Pedestrian Structure Attached to Truss Spans



Several roadway cross sections were considered. In developing the adjustments, designers considered the desirability of a center median and the preference for 12-foot-wide lanes (full width, as they are today). But with limited width available, some compromises had to be considered. New design standards allow consideration of 11-foot-wide traffic lanes (or

even 10-foot-wide lanes). Other considerations include clearance width to curb and gutter or barrier (curb reaction width is more flexible now than in the past), and the desire for a raised median. In addition, there is a joint along the centerline of the existing bridge approach spans. The longitudinal joints are integral to the unique original design of the Kennedy Bridge approach structures—that is, they provide a nearly invisible separation of the parallel approach structures, allowing the spans to move with the dynamic soils and hydraulics of the Red River. The joints will ideally remain along the centerline of the roadway because the deck condition would deteriorate faster if the joints are repeatedly driven over. Alternatively, the joints could be considered for elimination or adjustment with a new roadway cross section. However, the evaluations below of the adjusted roadway cross section concepts all consider the priority of retaining the approach span joints.

In recent design practice, 11-foot lanes have been considered a feasible and acceptable option and studied for safety effects. When existing 10- and 11-foot urban and suburban lane widths were studied, the lane width effects in the analyses were generally either not statistically significant or indicated that narrower lanes were associated with lower, rather than higher, crash frequencies.[5] This research also showed that 11-foot lanes can slightly reduce speeds and lane capacity (travel speeds are reduced by approximately 1.9 mph). For this study, 11-foot-wide lanes were considered potentially acceptable, given the constrained space inside the truss spans. Additionally, the limited space warranted consideration of wide curb lanes or shared lanes for bicyclists in the roadway. The cited research and experience in urban environments shows no significant difference in safety for bicyclists between in-roadway options and options providing a curb-separated, shared-use path.

The team also received and factored in local preferences, based on input received at meetings. These included preferences to maintain a center median for roadway safety in slippery conditions, a preference for physical separation of bicycles from the roadway, and identification of one shared-use path connected to the area's trails along the south side of the bridge. Local comments also referenced "wide load" movements across the bridge, including occasional crossings by agricultural machinery that can reportedly block the use of two lanes in a given direction by general traffic, eastbound or westbound. Some stakeholders, including GF-EGF MPO Executive Board members, expressed concerns about the wide loads as related to bicycle/pedestrian safety on the bridge, for shared-use path concepts inside the truss spans. The GF-EGF MPO Board also officially recorded a preference for the external/separate trail structure versus the cross section adjustments inside the truss spans.[6]

[5] Ports et al., Relationship of Lane Width to Safety for Urban and Suburban Arterials, TRB 2007 Annual Meeting. See also the MnDOT design memo at: http://dotapp7.dot.state.mn.us/edms/download?docId=1378703 (Traveled Lane Width Standards for State Highways, 2013)

[6] See GF-EGF MPO Board minutes of December 18, 2013 and January 15, 2014 (http://www.theforksmpo.org/Pages/Minutes.htm)

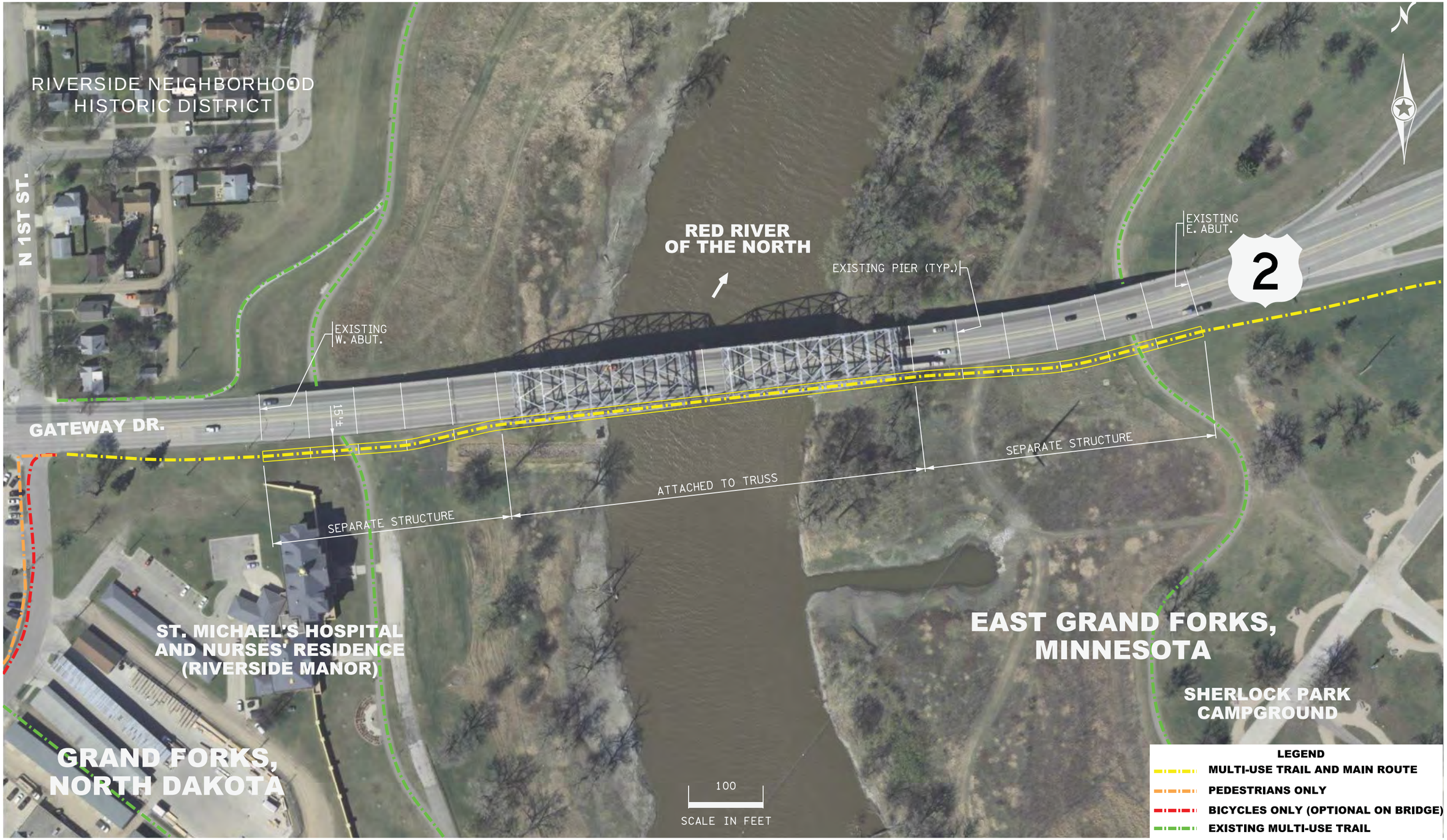


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Exhibit 3-5
Preliminary External/Separated Trail Layout

Based on reviewing more than a dozen cross sections to accommodate bicycles and pedestrians internal to the existing steel trusses, Exhibit 3-6 includes the five roadway cross section options initially considered most feasible.

Option A has no bicycle or pedestrian facilities and replaces the deck in kind with the existing lane configuration and widths, without a raised curb. The advantages and disadvantages of Option A are as follows:

Option A Advantages

- Four, 12-foot lanes with median
- Minimum shoulder and buffer requirements met
- Approach span joint protected by median
- Good cross section continuity with roadway approaches

Option A Disadvantages

- No pedestrian accommodation
- No bicycle accommodation
- No buffer from traffic if pedestrians use the paved shoulder (similar to observed use on the existing 2-foot-wide curb buffer)

Option B accommodates bicyclists using shared-use bicycle lanes in the roadway and pedestrians using a 6-foot-wide raised sidewalks. Under this option, bicyclists ride in a widened outside lane with other motorists across the bridge and the use of pavement markings encourage motorists to leave enough space for bicyclists. The advantages and disadvantages are as follows:

Option B Advantages

- Four, 12-foot lanes
- Raised sidewalk on both sides of bridge
- Bicycle access on both sides of bridge
- Approach span joint is centered, and not within lane lines

Option B Disadvantages

- No median
- Shared bicycle lane with traffic in outside lanes (not supported locally)
- Sidewalks are insufficient width for a shared-use path (some bikes would use)
- No approach span joint protection

Option C provides a non-uniform cross section with a raised pedestrian sidewalk along one side of the bridge (the south side is shown based on local preference). As with Option B, bicyclists are accommodated through shared-use lanes with motorists. The advantages and disadvantages are as follows:

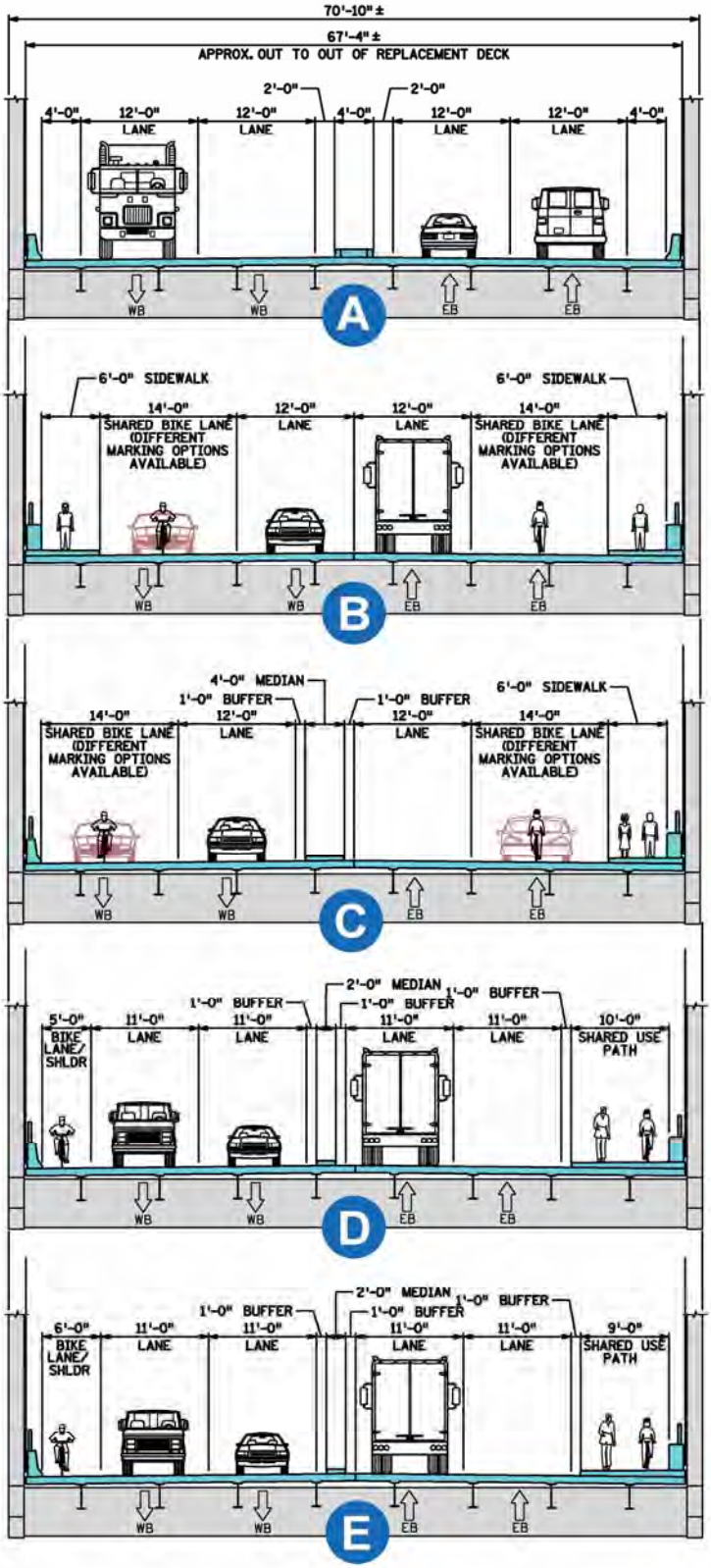
Option C Advantages

- Four, 12-foot lanes with median
- Bicycle and pedestrian facilities provided with a roadway cross section similar to existing, including median width
- Good cross section continuity with roadway approaches (better than Option B)

Option C Disadvantages

- Shared bicycle lane with traffic in outside lanes (not supported locally)
- Buffers to median are 1 foot
- Sidewalk is insufficient width for a shared-use path (some bikes would use)
- Approach span joint not protected by the median (without further adjustment)

EXHIBIT 3-6
Baseline and Adjusted Roadway Cross Section Options Internal to the Truss Spans



Options D and E were among the last cross section concepts developed and provide a raised shared-use path for bicycles and pedestrians along one side of the structure (south side shown) and a 5- or 6-foot shoulder, optionally marked as bike lane, on the other side. Option E removes 1 foot from the shared use path in order to create a wider (6-foot) shoulder and to better position the approach span joint. The advantages and disadvantages of Options D and E are as follows:

Options D & E Advantages

- Four travel lanes with a median
- Separate raised shared-use path with sufficient width (9 to 10 feet) for both bicycles and pedestrians
- Flexibility provided to position the approach span joint outside travel lane
- Shoulder with option for a marked bike lane opposite the shared-use path

Options D & E Disadvantages

- 11-foot-wide lanes (minimal effect on traffic; might reduce speed slightly)
- Narrow median problematic for maintenance (damage is more likely than with a wider median)
- Buffers to shared-use path and median are 1 foot
- Approach span joint not protected by the median (without further adjustment)

In conclusion, given the constraints of the cross sections inside the steel trusses, Option D is recommended by MnDOT's Bicycle and Pedestrian Section as the "preferred" cross section layout for a bicycle/pedestrian accommodation internal to the trusses. This preliminary recommendation was arrived at with the understanding that MnDOT's functional design leaders used maximum design flexibility to accommodate the many demands within the constrained corridor. More background on related design guidance is provided in the Bridge Rehabilitation TM, with reference to the *2012 AASHTO Guide for the Development of Bicycle Facilities* and the *MnDOT Road Design Manual* (AASHTO 2012, MnDOT 2012). Other remaining considerations for design concepts inside the truss, which would require more detailed design study, include safety factors, such as checking sight distance for motorists approaching the bridge (the Kennedy Bridge approaches include both vertical and horizontal roadway curvature that affect sight distance).

This Bridge Planning Study does not conclude with a final decision among the main sub-alternatives outlined herein for Kennedy Bridge rehabilitation, including accommodations for bicycles and pedestrians. However, it does support the Bridge Rehabilitation Alternative as the priority action and it provides background to support that priority and to support development of a project to include bicycle/pedestrian accommodations. Therefore, the comparison of project development alternatives in Section 3.2 includes the major choices for bridge rehabilitation with a shared-use path, *internal* or *external* to the trusses.

Another consideration as project development continues should be the development of connections between any shared-use path on the bridge and the greater community roadway and trail network. Exhibit 3-7 provides a sketch level concept only, which is based primarily on providing connections from a shared-use path along the south side of the Kennedy Bridge.

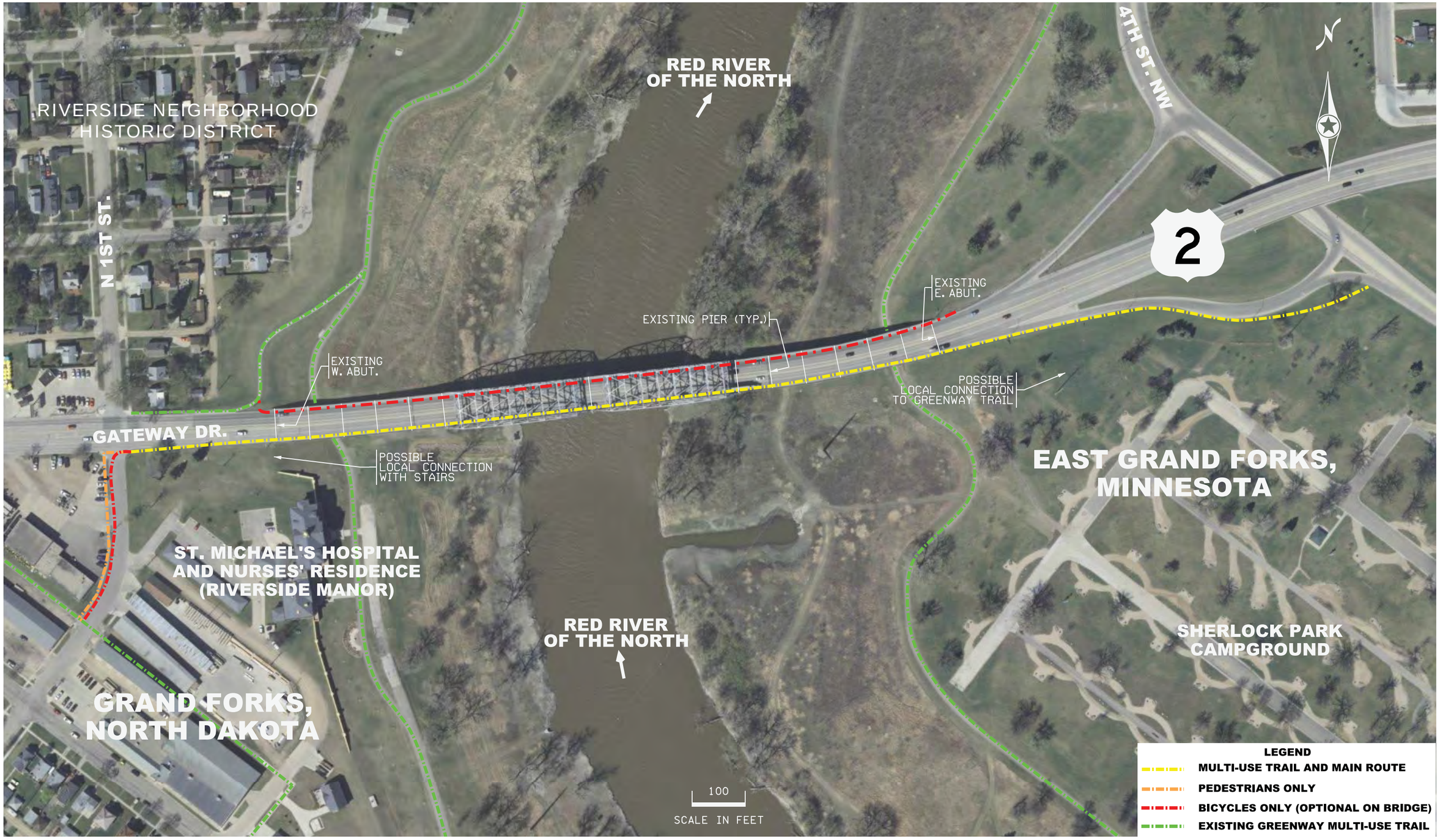


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Exhibit 3-7
Trail Connection Concept
Trail Layout

While additional connections to local/Greenway trails are highly desirable and would be addressed as noted, trail connection designs would require detailing in a project development process with cooperation and participation from local governments and other land-management entities (for example, the Minnesota State Park). Similarly, snow removal from a multi-use path typically requires cooperative agreements and understandings with local governments or other owning entities. MnDOT and NDDOT do not clear snow from paths as part of winter highway maintenance (including multi-use path areas on bridges).

3.1.2.4 Bridge Rehabilitation Project Development Process

The Bridge Rehabilitation Alternative would require project development to move forward into a substantially higher level of design investigation to refine the rehabilitation concepts. Because the Kennedy Bridge is eligible for the NRHP, the design process for all alternatives must comply with Section 106 of the National Historic Preservation Act. To avoid an adverse effect under Section 106, the bridge rehabilitation and its design details must meet the Secretary of the Interior Standards for the Treatment of Historic Properties. Avoiding an adverse effect under Section 106 is important to avoid invoking Section 4(f) of the 1966 Transportation Act, which does not allow federally-funded or -licensed projects to "use" a historic property unless there is no prudent and feasible alternative, and which requires that such use include all possible measures to minimize harm to historic properties.

Considering the rehabilitation elements identified for the Kennedy Bridge, the following issues would be addressed:

- **Pier 6, Other Substructure Elements, and Steel Members—**The replacement of Pier 6 is feasible to complete with an in-kind architectural style, which should greatly reduce the potential for a Section 106 Adverse Effect based on this one element.
- **Deck and Railing Maintenance or Replacement—**Replacement of the deck and integral railings may also provide opportunities to build elements that are similar in style to the existing. However, railings identical to the existing may not be reasonable based on modern crash protection and strength objectives, including protection of the historic truss elements. Nevertheless, the Bridge Rehabilitation TM identifies potential replacement railings, which are modern in their performance specifications, but similar in appearance to the existing. Therefore, there appears to be some potential to reach a finding of No Adverse Effect even with replacement railings.
- **Addition of a Shared-Use Path—**The most important comparison of impacts for bridge rehabilitation is the choice between designs providing a shared-use path internal to the steel trusses and the concept with an external/separate structure attached to the truss spans. The internal path designs provide a relatively low risk for a Section 106 Adverse Effect. In contrast, the external path structure may prove challenging to justify in project development based on additional costs, potential for a Section 106 Adverse Effect, and conflicts with the primary need of maintaining a structurally sound river crossing. More discussion of the issues anticipated for the external structure is incorporated in the following subsections.

The project development process, and environmental documentation, for a Kennedy Bridge rehabilitation project could vary in complexity depending on details to be included, including whether the external shared-use path is included in the project. As noted in Section 2.4,

the review process would include teamwork among professional engineers and professional historians. While this Planning Study has outlined anticipated actions and Section 106 review issues, additional detailed design studies and regulatory reviews will be required to complete the Section 106 review and reach final project development decisions.

As an example of detailed design review, the addition of a new external bicycle/pedestrian structure presents challenges as related to need, additional costs, bridge design, and environmental impact. For example, while the steel truss spans can provide sufficient capacity to support an external structure, the addition of a cantilevered trail presents several other challenges. Principally, the design team observed that attaching to the truss spans is in conflict with the primary need of maintaining the trusses, because it would create additional load and hinder inspection and maintenance—for example, by blocking or impeding favorable inspection and maintenance access to gusset plates and lower chord areas. Therefore, while the external structure design approach is technically feasible, the historically eligible fracture-critical trusses (to which an external shared-use path structure would be attached) have raised concerns about how desirable the design approach would be. As previously noted, these and other observations drove the effort to explore bicycle/pedestrian options internal to the trusses.

Other potential issues associated with the external shared-use path structure may include the following:

- **Multiple Section 4(f) Issues and Process Complexity; Potential Delays—**Adding the external structure would raise the potential for a Section 106 adverse effect based on the Kennedy Bridge alone. But the external structure, complete with its piers in floodway and other physical elements, may also present Section 4(f) conflicts with the numerous state and local park/Greenway jurisdictions and with the viewshed for the historic St. Michael's Hospital and Nurses' Residence. Comprehensive evaluations of the issues would add substantially to the project development schedule and, considering the ground rules for Section 106 and Section 4(f), could still result in findings and recommendations opposed to the external structure.

- **Geotechnical and Hydraulic Considerations—**Because the external structure would require new foundations and piers for approach spans, the concept raises technical and regulatory complexities. The complexities include design and maintenance of a new structure, partially on new foundations and partially attached to the existing bridge, in the setting's highly dynamic soils. Therefore, as with the Kennedy Bridge, the potential for movement and adjustments must be designed into the separate/attached elements of the new structure. Building new piers in the floodplain, and presumably next to existing bridge piers, brings other complexities and risks for geotechnical and construction engineering and in addressing Red River hydraulics.

For a Kennedy Bridge rehabilitation project, the likely choices for environmental documentation are:

- **Categorical Exclusion (CE)—**Bridge rehabilitation, including deck replacement, is included in lists of actions normally found to have no significant social, economic, and environmental effects. Such actions could be approved environmentally using an effi-

cient CE checklist approach, per 23 CFR 771.117.[7] This approach is possible, but is not a given, for a Kennedy Bridge rehabilitation project. The main questions about using a CE would be whether replacement of Pier 6, and of the deck/railing elements, would cause a Section 106 adverse effect. A design and project development process that does not include the external shared-used path structure would also be easier to develop than a project that does include the added structure. An Environmental Assessment (EA) level of review is more typical in cases where potential adverse impacts warrant extra review for significance.

- **EA—** A more detailed level of environmental study would be appropriate to address possible adverse effects. As noted above, an EA process is fitting for a more complex bridge rehabilitation project because the EA's objective is to determine if project effects can be managed to result in no significant adverse impacts. For example, evaluations of the issues associated with the external structure would add complexity to the project development process and would seem to warrant an EA. In this case, the issues and impacts (e.g., historic, hydraulic, geotechnical) would likely be compared in more detail to the concepts for providing a shared-use trail internal to the truss.

In summary, an EA process for a Kennedy Bridge rehabilitation project with the external shared-use path structure would be more complex and would likely take longer to complete than a baseline bridge rehabilitation project not addressing the external structure. This is based on additional design and environmental complexities and reviews—including studies of many alternatives. Several issues are previously identified in this report, including the potential for the external structure to create conflicts with the primary need for action. And again, the ground rules for Section 106 and Section 4(f) could still result in findings and recommendations opposed to the external structure.

3.1.2.5 Alternative Rehabilitation Scenario: Possible Later Bicycle/Pedestrian Structure Project

Another project development scenario for bridge rehabilitation is to completely separate a new shared-use path structure from the Kennedy Bridge rehabilitation project. The "separation" could be either physical, as a new trail bridge not immediately adjacent to U.S. Highway 2, and/or administrative, as a separate future project. For example, a new and separate project development process might be justified in the future if bicycle and pedestrian usage demonstrates a need for more capacity. This scenario illustrates that a short-term decision to exclude the external structure from project development would not preclude later consideration of a similar approach, as well as other physically separated bicycle/pedestrian structures.

Studies of completely separate trail structures, without attachment to the Kennedy Bridge, were beyond the scope of the Planning Study. Completing such studies, particularly for any new trail structure(s) located entirely outside the U.S. Highway 2 right-of-way, would involve alternative location and alignment comparisons. Development of such a project would

[7] Bridge rehabilitation, including deck replacement, is included in the list of actions which may be processed and approved through a CE in the Programmatic Categorical Exclusion Agreement Between FHWA and MnDOT 1998 (see document at this Web link: http://dotapp7.dot.state.mn.us/edms/download?docId=620464). *NOTE*: The cited document highlights bridge rehabilitation as one of a few eligible actions "with higher potential for environmental impacts." Additional detailed consideration must be given to potential bridge rehabilitation project impacts before a CE could be selected for environmental documentation.

also require studies for many of the same environmental impact issues identified herein for the external path structure, and possibly more issues.

3.1.3 Bridge Replacement Alternative

3.1.3.1 Description of Alternative and Options Evaluated

A Bridge Replacement Alternative was identified in the Bridge Planning Study; but bridge replacement would only be pursued if bridge rehabilitation cannot provide a feasible and prudent solution, meeting the primary need of maintaining a structurally sound crossing at this location. At the conclusion of the Planning Study, there was no evidence that the Bridge Rehabilitation Alternative is infeasible or imprudent. That preliminary finding was reached, in part, based on comparison of bridge rehabilitation to bridge replacement concepts (see Section 3.2 for more information about the comparison of alternatives).

The Bridge Replacement Alternative was explored primarily to provide a comparison to Bridge Rehabilitation and to identify the most promising bridge types and alignments, should replacement become a course of action in future project development.[8] To develop the bridge replacement concepts, design parameters were generated, which set the primary requirements for bridge replacement. These were developed by coordinating with MnDOT, NDDOT, and the other agencies/stakeholders included in the Planning Study process.

The main considerations for the bridge replacement study were design parameters, which led to two "families" of replacement bridge types—deck type and though type. The replacement bridge types were also considered in reference to potential alignment choices, as briefly summarized in the following subsections (see the Replacement Bridge TM for more detailed information).

Replacement Bridge Design Parameters

The Replacement bridge design parameters included requirements for addressing known soil movement issues, structural redundancy, river hydraulics, and geometric requirements to meet projected traffic demands and satisfy engineering parameters for roadway alignment and profile. Some of the key parameters were:

- **Fixed Pier Near the Center of the Red River**—Because soil movement appears to be minimal near the center of the river channel, it is desirable to locate a main river pier at that location (same as the existing Kennedy Bridge).
- **Horizontal Earth Movement**—The bridge superstructure, substructure, foundation, and bearings must be designed to accommodate earth movement, or have features to allow structural adjustments to be made in the field by maintenance crews.
- **Structural Redundancy**—A replacement bridge must have a structurally redundant superstructure; any replacement bridge should not be fracture critical like the existing Kennedy Bridge.

[8] The bridge replacement alternatives addressed in the Bridge Planning Study were developed to illustrate a possible range of costs and impacts, primarily for comparison to bridge rehabilitation. A complete evaluation of bridge replacement, to develop a project, would require more detailed engineering design and regulatory reviews—including accounting for more alternatives. For example, bridge replacement alternatives could be considered which retain the existing Kennedy Bridge repurposed for bicycle/pedestrian use.

- **Bridge Type**—The reasonable range of replacement bridge types was determined primarily based on types considered cost effective for the setting. For example, a signature-type cable bridge or bridge with spans exceeding 300 feet is not in the reasonable range because longer spans are not necessary for the setting and would result in unnecessarily high costs.
- **River Hydraulics and Bridge Profile**—Given the periodic floods of the Red River, a replacement bridge would need to meet the requirements of the Federal Emergency Management Agency (FEMA), MnDOT, NDDOT, both states' DNRs, and USACE. Design parameters for a new bridge profile would not only include clearance above anticipated 100-year recurrence floods, but also hydraulic design to ensure no worsening of floods.
- **Construction Staging and Traffic**—Considering the need to maintain the crossing and public input, two lanes of traffic in each direction should be maintained throughout the majority of bridge construction and demolition, lasting approximately 2 years. Less capacity could also be provided for short durations to transition traffic through stages of construction.
- **Approach Roadway and Bridge Alignment**—The Planning Study considered a range of alignments for a replacement bridge, all on or near the existing Kennedy Bridge alignment. The two most promising alignments were found to be north of the existing bridge, thereby creating the potential to build a new bridge while keeping the existing bridge open. Other alignments, to the south or along the existing bridge, were considered less promising because of greater potential for adverse impacts.

Representative Replacement Bridge Types

The bridge types considered ranged from steel girder to arch/truss type bridges, and these were evaluated along the north alignments to generate a range of costs. The Bridge Replacement TM discusses the full range of viable options for replacement bridge types along the two alignments and Exhibit 3-8 summarizes the alignments and bridge types for the projected lowest and highest probable construction costs. With Reference to Exhibit 3-8, the bridge type families considered were:

- **Deck Type Bridge Superstructures**—Deck bridges with straight beams are common, employing below-deck beams to span a range of distances. The family of deck types considered in the Planning Study included steel I-girders, steel tub girders, concrete girders, and concrete boxes (Exhibit 3-8). To best illustrate likely solutions, the steel bridge types in this family were selected for concept layout and cost estimating. The concrete beam bridge types are not considered as viable because they would not accommodate the pier movement and adjustments required for a Red River bridge. The steel girder bridge types can accommodate movement and adjustment well, as well as roadway curvature; therefore, they could be used with either alignment in Exhibit 3-8. The range of estimated costs for the selected steel deck type bridges was about $23 million to $30 million (bridge construction only).
- **Through Type Bridge Superstructures**—Through bridges are less common than deck bridges, but familiar, like the 1963 Kennedy Bridge—employing a superstructure above the deck to provide longer spans and/or to flatten roadway profile through less beam

depth below deck. The family of through types considered in the Planning Study included steel tied arch or modern truss types, with longer river spans than a deck bridge, yet still with a middle pier (Exhibit 3-8). The through bridge types do not accommodate curvature well and, therefore, may be more compatible with a straighter alignment as shown in the bottom layout on Exhibit 3-8. The range of estimated costs for the selected steel through type bridges was about $33 million to $39 million (bridge construction only).

Replacement Bridge Performance and Tradeoffs

A replacement for the Kennedy Bridge would perform well, certainly addressing the primary need to maintain a sound river crossing far into the future. A new bridge, with modern engineering technology, would provide a structurally redundant system (it would not be fracture critical). A new bridge would also provide superior performance for all modes (vehicular, bicycle, and pedestrian) and for bridge inspection and maintenance.

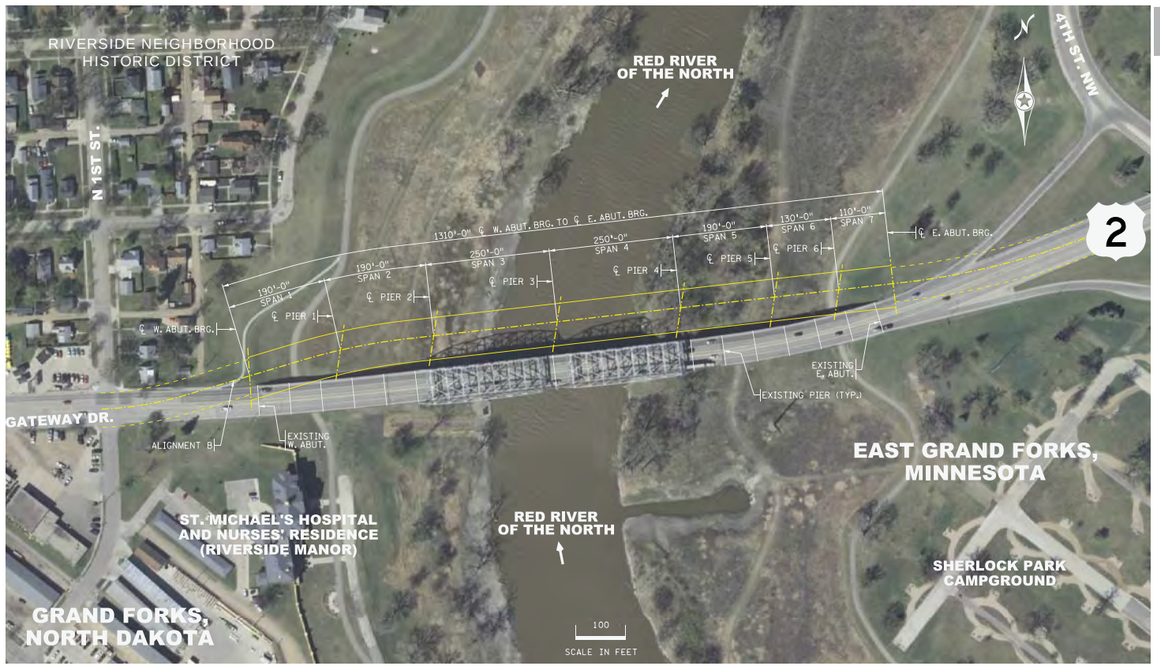
The tradeoffs for the Replacement Bridge Alternative are primarily matters of cost and feasibility, given the good potential performance of a Bridge Rehabilitation Alternative, with its lower costs and fewer adverse impacts. A Bridge Replacement, while offering superior performance, is evidently not required to address either primary or secondary needs. Therefore, its higher level of performance could be judged as marginal compared to the benefit/cost tradeoffs of bridge rehabilitation (see Section 3.2).

3.1.3.2 Bridge Replacement Project Development Process

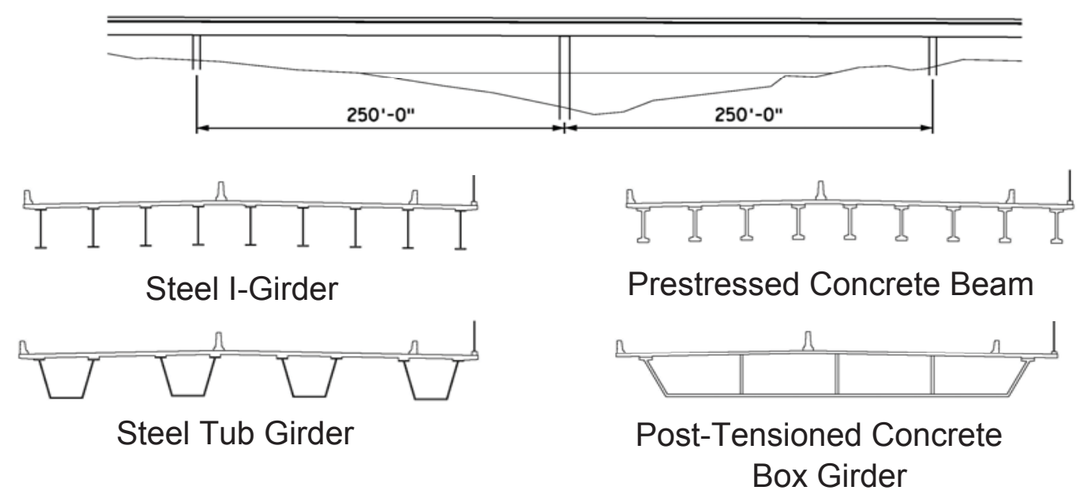
A replacement bridge would involve additional environmental impacts, including the Section 106 Adverse Effect of removal of the existing historic Kennedy Bridge. As noted in Section 2, there are additional historic and recreational resources that would be impacted by a replacement bridge project. This would include much greater potential for adverse effects on the Riverside Neighborhood Historic District, other historic structures, and on state and local parklands.

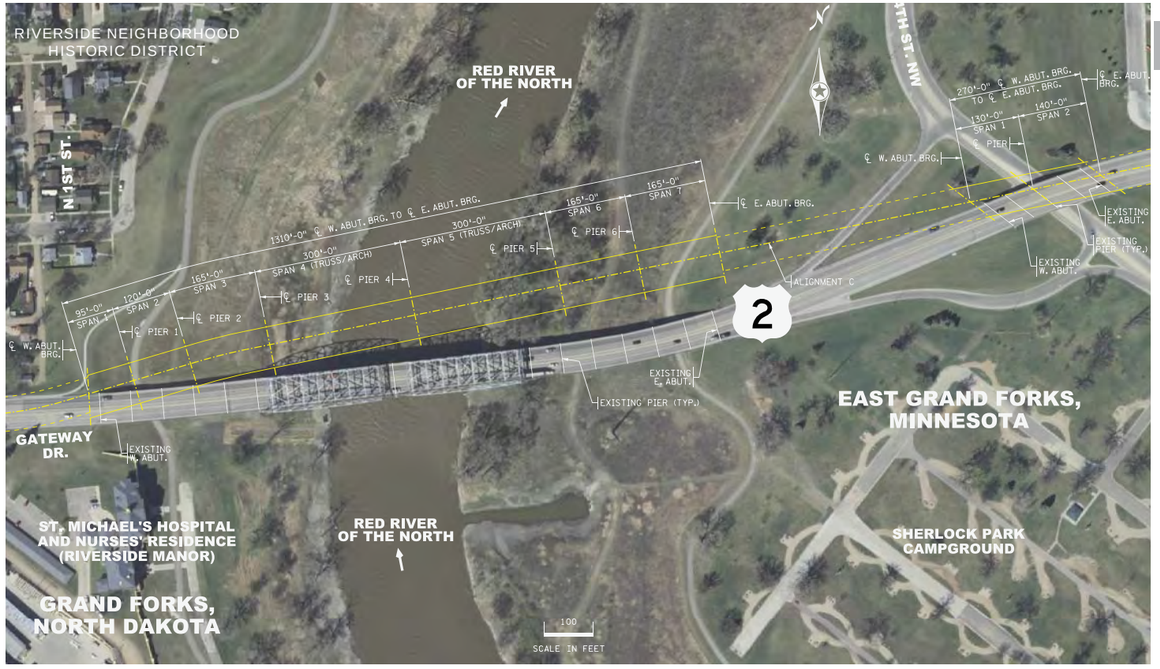
In addition, a replacement bridge would involve new geotechnical and hydraulic designs. Therefore, the environmental documentation process for a bridge replacement project would be much more involved than that required for rehabilitation of the existing bridge in the same footprint. The process would also be more complex because it would require studies of many more alternatives, including studies to support decisions to not move forward with rehabilitation (a difficult case to make for this bridge, given its adequate current capacity and the comparison of alternatives—see Section 3.2). The level of environmental documentation would be at least an EA and could warrant an Environmental Impact Statement (EIS) if many adverse impacts are anticipated and if there is controversy.

In summary, development of a Kennedy Bridge replacement project can be expected to take substantially longer than the baseline bridge rehabilitation package (without an external shared-use path structure). Many potential issues for such a project have been identified, including the apparent difficulty of developing a defensible case for bridge replacement.



Deck Type Bridge Superstructures





Through Type Bridge Superstructures

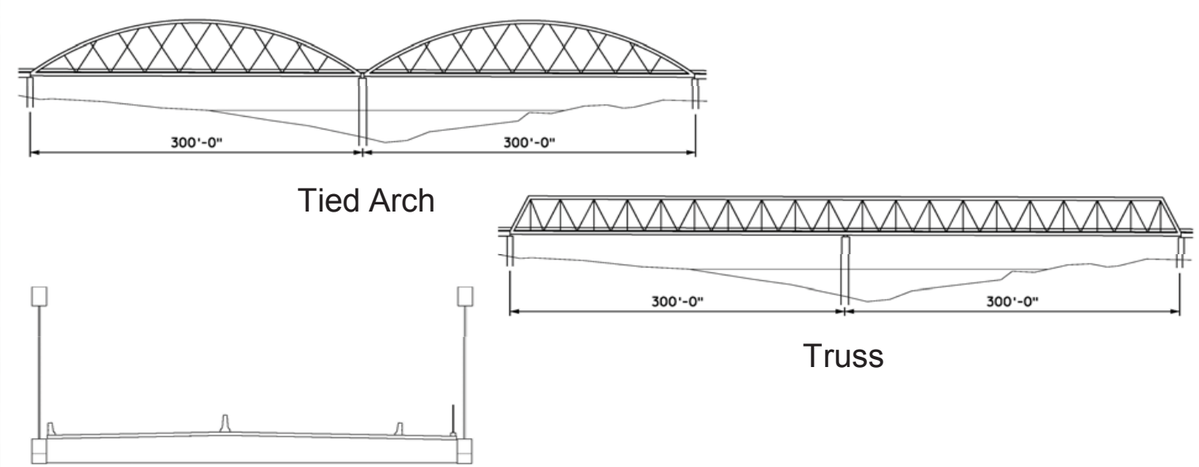


Exhibit 3-8
Range of New Bridge Concepts

3.2 Comparison of Project Development Alternatives

3.2.1 Matrix Comparison of Representative Alternatives

The next steps, following the Bridge Planning Study, will include recommendations for project development based on the concepts/alternatives addressed to date. Table 3-2 provides a matrix comparison of the three representative project development alternatives:

- **Bridge Rehabilitation A (bicycle/pedestrian accommodation *internal* to truss)—**Pier 6 replacement, deck replacement/reconfiguration, painting, other structural adjustments, and replacement railings on both truss spans and approach spans to meet vehicle and bike/ped standards.
- **Bridge Rehabilitation B (bike/ped trail on separate *external* structure, attached to truss spans)—**Pier 6 replacement, deck replacement/reconfiguration, painting, other structural adjustments, construction of separate/external trail structure, and replacement railings on both truss spans and approach spans to meet vehicle standards.
- **Bridge Replacement—**New wider bridge adjacent to existing with required tie-ins at west and east ends, possible improvements to ramps at 4th Street, and possible replacement of bridges over 4th Street.

The three alternatives are compared to illustrate the wide range of alternatives and sub-alternatives, to outline MnDOT's current priorities for development of a bridge rehabilitation project. Additional design studies and a formal environmental review of a proposed project may bring changes to these/other alternatives to determine a preferred alternative.

The project development choices in Table 3-2 outline the possible next steps and issues to be considered at the completion of the Kennedy Bridge Planning Study. In this manner, the information above provides a concluding summary of the many issues and design concepts discussed throughout this Final Report, in the Planning Study's technical memoranda, and with stakeholders throughout the process.

3.2.2 Relationships to Other Anticipated Projects

Section 2.2 discusses relationships between the Kennedy Bridge and other local Red River bridges. A complete project development process will continue to demand consideration of these relationships—especially in reference to the historic Sorlie Bridge, as it is also now being evaluated for rehabilitation or replacement. Therefore, as part of project development, relationships to potential modifications to the Sorlie Bridge should be addressed, both physically and in scheduling for construction. Avoidance of adverse traffic impacts from concurrent construction activities should especially be considered and avoided.[9] Similarly, coordination with any other local transportation projects should be considered along with any other relevant community project plans and actions.

[9] In current planning and programming, Kennedy Bridge construction is targeted for 2016 and Sorlie Bridge construction for 2018. Both bridges are listed on the Chapter 152 master bridge list, which identifies 172 bridges meeting the law's criteria (see Section 1.3). As such, the bridges are to be under contract for rehabilitation or replacement by June 30, 2018.

TABLE 3-2
US Highway 2 Kennedy Bridge Project Development Alternatives

Evaluation Criteria	Bridge Rehabilitation A (Internal Bike/Ped)	Bridge Rehabilitation B (External Bike/Ped)	Bridge Replacement
Bridge Capital Cost	$13.5 Million *	$16.4–$17.4 million *	$23–$39 million *
Structural Performance and Maintenance	Addresses primary need; good long-term service life with regular maintenance; inspection issues similar to the existing bridge	Addresses primary need, with conflicts; good long-term service with regular maintenance; however, inspection of the original truss members around the external structure and maintenance access would be blocked or limited	Addresses primary need and adds structural redundancy; longest service life; superior inspection and maintenance access provided through design of any replacement bridge type
Construction Period and Traffic Impacts	About 1 year of temporary traffic impacts and capacity restrictions	About 1 year (similar to Rehabilitation A)	About 2 years of temporary traffic impacts with periodic capacity restrictions
Historic Preservation (Section 106) - Risk of Adverse Effect	Low to moderate potential for adverse effects	Much higher potential for adverse effects	Removal of historic bridge is an adverse effect; potential additional impacts to the Riverside Neighborhood Historic District
Long-Term Traffic and Bike/Ped Function	4 roadway lanes; median is feasible Ped/bike on bridge deck Good performance for all modes; shared-use path and bike shoulder options	4 vehicle lanes with median Separate ped/bike path on external structure Superior performance for all modes; one exclusive shared-use path	4 vehicle lanes with median Separate ped/bike path included in wider bridge deck Superior performance for all modes; one exclusive shared-use path
Project Development Risks and Approval Process	Lowest-risk and least potential for adverse impacts, including Section 106 adverse effects; smallest footprint; prompt reviews and approvals possible through a CE or an EA prepared in parallel with detailed design	Substantially higher project development risks (Section 106, hydraulic, and other impacts); increased footprint; review and approvals more complex, through an EA; potential to add months or years to the design/approval process	Highest project development risks (many adverse impacts, including Section 106, with largest footprint); review and approvals most complex, through an EA or EIS; potential to add years to the design/approval process

* The preliminary cost estimates are for bridge and required roadway construction only and are subject to additional reviews (no roadway work is assumed for the rehabilitation alternatives). Additional costs, not addressed in the estimates, would include design engineering, approach roadway/ramp reconstruction, drainage features, off-bridge trail adjustments, and right-of-way if needed.

3.2.3 Concluding Remarks and Next Steps

The tradeoffs conveyed in Table 3-2, including costs, long-term performance, bike/pedestrian function, and project development process, capture the essence of the Kennedy Bridge context and the need for action with a timely project design and construction process. The following key points will also be considered as MnDOT and its partners select and begin the Kennedy Bridge project development process:

- Bridge rehabilitation is the priority action; the Planning Study has confirmed that bridge replacement would be much more costly, would bring other adverse impacts, and is not

likely to be supported by relevant data and regulatory process reviews. Considering structural analysis and inspection, the Planning Study found the steel truss spans to be in satisfactory condition, capable of accommodating further movements and adjustments. But the bridge deck is in poor condition, requiring replacement, and the integral railings are substandard (the railings are in need of replacement integrally with the bridge decks on both truss spans and approach spans).

- Replacement of Pier 6 is the most pressing bridge rehabilitation component; the next steps into project development should avoid any undue delay in design and construction of the pier replacement. Monitoring of Pier 6 should also continue, including after pier replacement, to observe performance of the improvement.
- The Bridge Planning Study has presented the important tradeoffs for the choices and issues to be addressed as project development begins. MnDOT and its partners will collaborate on the results of the Bridge Planning Study and in taking the next steps.
- The project development process will move forward based on the final reports and will continue to seek and reflect stakeholder input. As previously noted, the detailed design and project review/approval process would include teamwork among professional engineers and historians to address Section 106 issues and reach final decisions.

SECTION 4

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Appendix A
MnDOT Draft Statement of Purpose and Need

Minnesota Department of Transportation

District 2
3920 Highway 2 West
Bemidji, MN 56601

Red River of the North – Kennedy Bridge
US TH 2 – S.P. 6018-02, MnDOT Bridge #9090
US TH 2, ND/DOT Bridge #02-350.220

Introduction: Transportation System Background and Context

Mn/DOT Bridge #9090 (ND/DOT Bridge #02-358.220) is a Minnesota/North Dakota border bridge located over the Red River of the North on U.S. Trunk Highway 2 (US TH 2). It is partially located within the corporate limits of East Grand Forks, Minnesota and Grand Forks, North Dakota. This route serves as a connection between North Dakota and Minnesota, as well as access from US TH 2 in Minnesota to Interstate Highway 29 (I-29) in North Dakota. I-29 bisects the United States in a north/south direction. US TH 2 bisects the United States in an east/west direction between Maine and Idaho.

US TH 2 is a 4-lane roadway with 12’ driving lanes and 4’ shoulders. The posted speed limit in East Grand Forks is 35 mph. US TH 2 is rated as a ten-ton route.

The average daily traffic (AADT) is 20,800, with a heavy commercial average daily traffic (HCADT) of 1,770 with the posted speed limit of 35 mph. The next closest crossing of the Red River between Minnesota and North Dakota, on the Trunk Highway System is located on US TH 2B, approximately one mile south in the urban area of Grand Forks/East Grand Forks (Sorlie Bridge) with the next closest crossing located approximately 20 miles north in Oslo, Minnesota.

Anticipated Project Funding:
The Minnesota Department of Transportation (MnDOT) anticipates utilizing Federal and State funding relative to the bridge and associated roadway work.
The North Dakota Department of Transportation (NDDOT) anticipates utilizing Federal and State funding relative to the bridge and associated roadway work.

SUMMARY OF PURPOSE AND NEED

The primary purpose of this project is to provide a structurally sound crossing of U.S. TH 2 (TH 2) over the Red River of the North at East Grand Forks, Minnesota. Section A below describes the bridge structural conditions that need to be addressed by the project.

There are also secondary needs to consider during the development and evaluation of alternatives for this project. These needs (summarized in Section B below) include maximizing maintenance of traffic during construction; possibilities for addressing additional existing bridge operational deficiencies; and providing improved accommodations for pedestrian/bicycle traffic. There are also other factors that should be considered in the development and evaluation of alternatives, as described in the Other Considerations section below.

A. PRIMARY NEED: A STRUCTURALLY SOUND BRIDGE

The primary reason for undertaking this project is to address the condition of the bridge structure (described in detail below) to continue to provide a crossing of the Red River..

Bridge Background:
This bridge was constructed in 1963 and is a 13 span structure consisting of two Parker Truss style high truss main spans and 11 steel multi-beam approach spans. The trusses are constructed of steel members assembled with a combination of welds and high strength bolts. The roadway width is 28 feet, with a maximum vertical clearance of 19.8 feet. The total length of the structure is 1,261 feet.

MnDOT Assessment of the Condition of Bridge #9090:
The original deck (1963) is a 7" monolithic cast in place deck and has non-coated reinforcing steel. The typical service life for this type of deck is approximately 50 years. The deck was scarified and a low slump overlay was placed in 1984 to extend the life of the bridge deck for another 20 years and protect the black bars in the deck. The overlay is reaching the end of its service life. The expansion joints were also rehabilitated in 1984. The deck is currently in satisfactory condition with a National Bridge Inventory (NBI) condition code rating of 6.

<u>Approaches</u>
The bridge railing is substandard in height and structural capacity.

The superstructure is in good condition. Rust is beginning to form on the approach spans. There is some active corrosion in the bottom flange of the fascia beams.

Pin and hangers located along the bottom chord are not fracture critical, but do require special inspection.

The steel columns at Bents 2 to 5 and 9 to 13 exhibit some surface rust. The embankment on the west side of the river appears to be unstable and erosion of the soil is evident. The fixed hinges that support the stringers on the top of the steel pier bents are securely attached to the top of the bent. When soil movement occurs, bents are adjusted back to plumb.

There are pedestrian trails under the bridge structure on the North Dakota approach (parallel to the river) near where the District Bridge Maintenance crew has performed concrete deck delamination removal with the overhangs appearing to be in the worst condition. The falling pieces of the deteriorating concrete deck present a hazard to pedestrians, thus requiring delamination removal.

Main Spans

The bridge railing on the truss will need to be analyzed to determine its crashworthiness upon impact which may lead to modification.

The superstructure is in satisfactory condition with a NBI condition rating of 6. The current structural condition of the bridge can support legal loads, however, 'permitted' overweight Type C loads are restricted to traveling down the center of the two lanes.

There appears to be more rust exhibited at either end of each trust (L0 and L0') and at the splice connections than at other splice locations. At all four corners of each truss (L0 and L0') there is an enclosed area on the top of the bottom chord that traps water and debris. This water and debris are causing corrosion to the top of the bottom chord and the inner surface of the vertical gusset plates.

AASHTO M270 Grade 100 (ASTM A514/A517) steel, more commonly known asT-1[1] steel is located on portions of the bottom chord. In 2007 and 2008, cracks were found on T1 steel at the L0 south side of the west truss. The cracks are re-inspected annually. Other defects that have been found in this welding are lack of fusion, lack of fill, and undercut.

At panel points L1A, L1A', L2' and L2, the vertical gusset plates are stiffened with angles along the unsupported length. These angles are only tack welded to the gusset plate and the space between the welds has allowed moisture and rust to develop between the angle and the gusset plate. This rust has caused the tack welds to crack in many locations. At L1A' on the north truss, stiffening angles have become detached from the gusset plate. They have been reattached by bolting. Tack welds at stiffening angles are monitored during inspection and repaired as necessary.

It should also be noted that at random floor-beam locations, pack rust has developed between the horizontal connection plate and the bottom of the floor-beam. This pack rust is a maximum ¼" at random locations and has slightly distorted the plates. At L3N, west span, the bottom connection plate appears to have been bent upward during original construction to enable the connection of the bottom flange of the floor-beam which appears to be slightly higher than the bottom of the lower chord.

[1] T1 steel is an alloy steel composed of several elements. T1 Steel is composed of 18 percent tungsten (higher than normal to promote durability) or molybdenum, which was used as a replacement for Tungsten after 1940. Chromium makes up only 3 to 4 percent of T1 grade machine steel, with cobalt being altogether absent from its alloy. These elements are then treated at high heat in order to form high speed steel.

A major consideration along the Red River of the North is the instability of the subsurface conditions, due to deep subsoil movement. Other bridges across the Red River of the North in this area experience substructure movements, and this bridge is no exception. Substructure movements on this bridge have occurred in the last 20 years and are irregular and unpredictable. The substructure is in fair condition with a NBI condition code of 5.

Pier 6 has diagonal cracks on both faces of the pier. The cracks go in the opposite direction of each other and could indicate torsion or twisting of the pier. Pier 6 has exhibited substantial movement due to subsurface instability, and is nearing the expansion adjustments provided. The rockers have shifted to the west, the south side of the pier has moved to the east, and the north side of the pier has moved further north. There are two remaining 7" adjustments that can be made at Pier 6 without major modifications to the pier or truss bearings/gussets. North Dakota and Minnesota have jointly agreed that a plan to accommodate additional pier movement will be developed when only one adjustment remains. Continued movement will necessitate modifications or replacement of the pier and/or rocker bearing and gusset plate modifications. Movement at the Pier 6 bearing is monitored on a six-month basis. The total movement has been measured 26 inches since September 1997. The last three measurements taken in on March 7, 2012, March 22, 2012, and May 14, 2012 indicate that the pier have moved approximately 1 inch. The last significant movement took place in 2003/2004 and measured approximately 8 inches. Movement of the pier has occurred at each measurement and is anticipated to occur at future measurements.

At Pier 7, the rocker bearings on the south truss have shifted in opposite directions. There are sliding plate bearing in Spans 5 and 8. The keeper plates for several of these bearings have cracked due to pack rust.

General

The bridge was last painted in 1996, however, large areas of the paint are failing and will need substantial repair.

Inspections of this bridge have indicated the following conditions which require monitoring and/or responsive action:

- Pack rust at various connections due to active corrosion, including distortion of members. Required to be cleaned and painted routinely.
- Cracked tack welds that vary in severity. Required to be ground out and possibly repaired.
- Section loss up to 32% on gusset plates. This will continue to lower the structural capacity without action.
- Delamination of concrete from the bottom of the deck exposing reinforcing steel.
- Pigeon issues, including corrosion due to pigeon droppings. (This structure is a pigeon "haven" of which presents an inspection hazard as well as has created corrosion of which the bridge inspectors were required to remove an excess of 2.0' of pigeon droppings by cutting into the gusset plates to flush out the pigeon droppings where the upper and lower chords meet.)

B. SECONDARY NEEDS

Maintenance of Traffic:
The communities of East Grand Forks, Minnesota, and Grand Forks, North Dakota, have a need for a continued reliable river crossing and connection to the US Trunk Highway system and the nearby Interstate highway system at this location. The United States Air Force Military Base, University of North Dakota, agricultural community, manufacturing enterprises, emergency services, commuters, and interstate commercial traffic share this need. The Grand Forks/East Grand Forks area is an economic hub for the region that consists of approximately 150 miles in all directions, including Canada. The lack of a continued dependable river crossing at this location, or the temporary closure of the bridge would have an adverse effect upon the communities and the movement of interstate traffic. US TH 2 is a Principal Arterial route with an AADT of 20,800 that connects two communities in two States. The detour route for unrestricted trunk highway traffic is nearly 50 miles; therefore, provisions for maintenance of traffic during construction will be a consideration. The closest river crossing without clearance or weight restrictions is location approximately 50 miles south at Halstad, Minnesota (MnDOT Bridge #54004). This crossing is also a border bridge, jointly owned and maintained by MnDOT and NDDOT.

The MnDOT Bridge #4700 (Sorlie Bridge), one mile to the south, accommodates an existing volume of approximately 13,000 AADT with the HCADT being approximately 1,200 with the posted speed limit of 30 mph. TH 2B is a ten-ton route. Although, the Sorlie Bridge on its own accord is able to accommodate an AADT of 13,000, the timing of the signal lights and geometrics of the roadways located in the downtown areas of East Grand Forks, Minnesota/Grand Forks, North Dakota cannot accommodate an increase in traffic (e.g., if traffic were detoured from TH 2/Kennedy bridge to the Sorlie bridge during construction), without operational issues.

During annual spring flooding, the Kennedy Bridge is one of four river crossings that typically remain open between Moorhead, Minnesota and the Canadian border. Historically, this bridge has been the last bridge to close during extreme flood events. If the elevations of the flood waters became high enough, this bridge would likely be the last river crossing to remain open between Moorhead, Minnesota and Canada. The west approach touches down within the protected area of the North Dakota flood area and on the east side of the river, the approach fill area floods; however, emergency levies and other measures have proven successful in keeping this river crossing open during flood events.

Pedestrian Background and Needs:
The East Grand Forks Metropolitan Planning Organization has identified a desire for improved pedestrian access, mobility and connectivity to the existing river crossing[2]. East Grand Forks and Grand Forks currently has 46 miles of paved pedestrian/bicycle trails that traverse both Cities and Greenway areas. There is an existing multi-use trail underpass under US TH 2 just to the west of MnDOT Bridge #9090. Construction of a 10' multi-use trail to connect the existing trailhead at 12th St. NW to the US TH 2 multi-use trail underpass will be constructed during the

[2] East Grand Forks Northwest Street Network Study. (Draft Report October 28, 2011.) Prepared by Alliant Engineering, Inc. The purpose of the study is to provide a comprehensive look at the future needs of specific intersection(s), provide enhanced north-south connectivity while considering multi-modal needs (transit, pedestrian and bicycle) of the area.

2012 construction season. This construction project will utilize funding received from a Transportation Enhancement (TE) Grant. An additional 18 miles are currently planned. It is anticipated that a portion of the 18 mile multi-use trail will also be constructed during the 2012 construction season.

There is a need to consider opportunities to provide bicycle/pedestrian accommodations on the TH 2/Kennedy bridge to provide better access to the accommodations provided on the approaches and the Park located in the vicinity of the bridge on the Minnesota side of the Red River of the North.

C. ADDITIONAL CONSIDERATIONS

Regulatory Requirements: Historical Resources:
The existing bridge was determined by the Minnesota State Historic Preservation Office (SHPO) to be eligible for listing on the National Register of Historic Places as a significant example of major river crossings in Minnesota. This bridge is significant for its exceptional main span length of 279 feet for the Parker steel thru-truss under *Criteria C* (Engineering) at the State level of significance. This bridge is also eligible under *Criteria A* (Transportation) at the State level of significance because the bridge represents an initiative to improve and expand transportation networks in the region, opening new areas in Minnesota to economic development.

Structural Redundancy:
Minnesota Bridge #9090 is a fracture critical bridge with non-redundant structural design (main spans only). Chapter 152 of the Minnesota Legislature 2008 Session Laws (Chapter 152) directs MnDOT to establish a bridge improvement program with an emphasis on structurally deficient and fracture critical bridges. It is anticipated that this bridge will be partially funded under the Chapter 152 program which includes a requirement that if the bridge is repaired but not replaced, an explanation of the reasons for the repair instead of replacement is required to be submitted to the Minnesota State Legislature.

River Hydraulics:
Hydraulics along the Red River of the North is a consideration, with respect to any bridge structure improvement work that may be contemplated. The hydraulics is very complicated and any change must be carefully analyzed to determine impacts both upstream and downstream of this site. Due to frequent flooding, minor erosion around substructure units occurs.

Derrick Dasenbrock P.E.
Project Manager
MnDOT Office of Materials and Road Research
1400 Gervais Ave
Maplewood MN 55109
651.366.5597
derrick.dasenbrock@state.mn.us

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Appendix F:
Technical Memorandum:
Kennedy Bridge – Summary of Pier 6 Movement Records

TECHNICAL MEMORANDUM

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Kennedy Bridge - Summary of Pier 6 Movement Records

PREPARED FOR: MnDOT

COPY TO: Dale Thomas / CH2M HILL
File

PREPARED BY: John Hinman / CH2M HILL

DATE: May 16 2013

PROJECT NUMBER: 469027

Introduction

Bridge No. 9090, known as the Kennedy Bridge, carries U.S. Trunk Highway 2 across the Red River of the North between East Grand Forks, Minnesota and Grand Forks, North Dakota. This technical memorandum summarizes the records of the movements of Pier 6 of the Kennedy Bridge, as provided by the Minnesota Department of Transportation (MnDOT). It also summarizes the movement of Pier 6 relative to the steel truss superstructure. The location of Pier 6 is shown in Figure 1. An overview of the structure at Pier 6 is shown in Figure 2, with key components labelled.

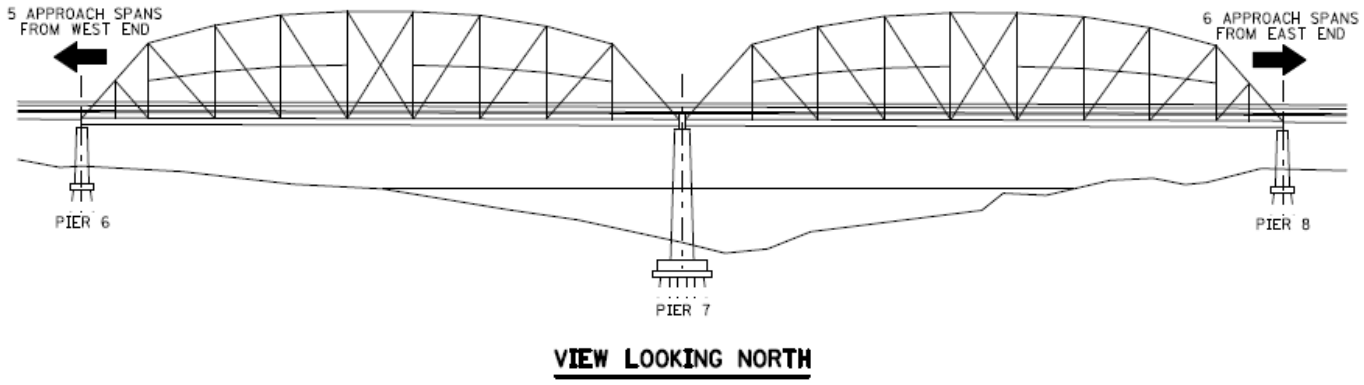


Figure 1 – Location of Pier 6

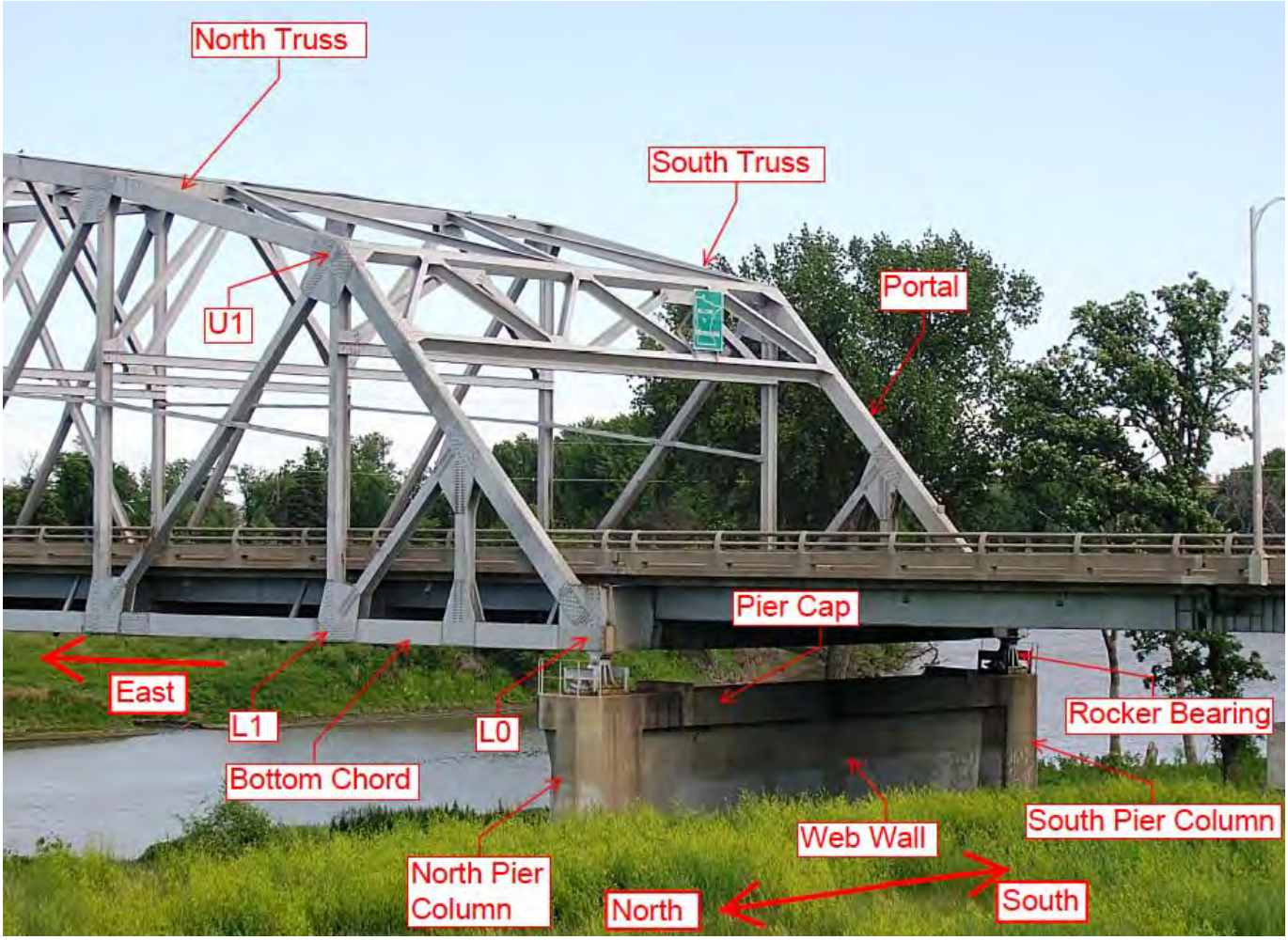


Figure 2 – Structure and Pier 6 overview. Photo taken from ND side of river, looking southeast

Movement Records

Movement records available from MnDOT addresses of three separate movements (provided in the Appendices). These documents include various field measurements related to bearing, pier and ground movement:

Log of Bearing Tilt. (See Appendix A) This document includes a log of the tilt of the bearings out of plumb that translates into top of pier movement. The measurements begin 10 September 1997 and extend through 24 October 2012. Adjustments to the rocker bases and the attachment of the bearings to the truss chord are noted along with the tilt measurements.

MnDOT records identify the bearing movement as “tilt.” The value recorded is the horizontal offset of the top of the rocker from a plumb position, rather than an angular measurement.

In addition to the rocker tilt measurements, this information contains distances from Bent 5 to Pier 6 recorded periodically between 23 September 2010 and 17 July 2012, and measurements of the bow in the wall that connects the north and south pier columns.

Pier 6 Measurements. (See Appendix B) This consists of sketches of the existing configuration of Pier 6 from field measurements as of 16 July 2012. Measurements noted include tilt of the pier, location of rocker bases on the pier cap, sketch of the rocker bearing heel measurements, log of rocker bearing tilt through the July measurements, and photos.

Slope Inclinometer Results. (See Appendix C) This document contains summaries of readings from two different slope inclinometers placed close to the south end of Pier 6. Readings cover the time periods from 3 June 2004 to 23 November 2009 (Inclinometer #1) and from 12 July 2010 through 2 May 2012 (Inclinometer #3). No readings were obtained from Inclinometer #2.

Slope inclinometer readings provide incremental and cumulative movements in two axes. Readings extend to a depth of 60 feet below the existing ground surface.

Movements not documented. All structure measurements document movement parallel to the centerline of the truss spans. No structure measurements address movement normal to the bridge. No measurements document vertical movement of Pier 6.

Discussion of Pier Movement

Initial Conditions

The initial conditions are those at the date of construction. The design drawings do not identify the location of the Pier 6 rocker bearing with respect to the pier cap and the columns that make up Pier 6. MnDOT field observations and measurements (See Figure 6) indicate that the centers of the bearings were constructed 8 inches east (toward the river) of the center of Pier 6.

Design drawings of the pier are reproduced in Figure 3. These drawings show a 5-foot-wide pier cap centered above the pier columns and the pier footing. A 2-foot-wide corbel is shown in the east side (towards the river); this corbel is believed to be provided to allow working room for jacking the truss and adjusting the bearings.

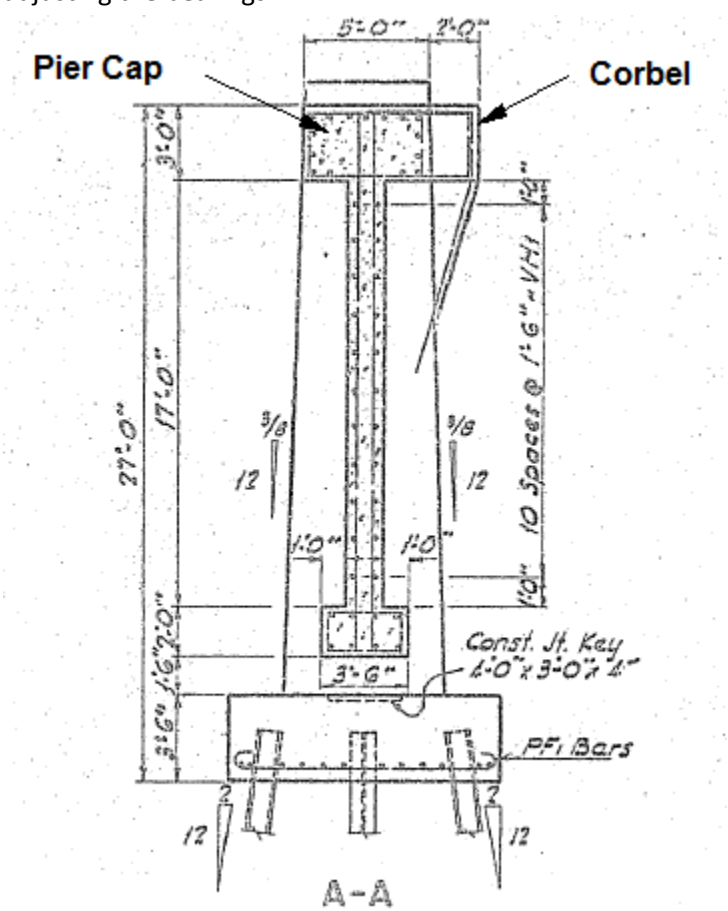


Figure 3 – Section Through Pier 6

The rocker bearing is a fabricated steel element with a cylindrical bottom surface that bears on a 5 ¾ inch thick by 36 inch wide by 60 inch long plate. A series of four pintles attached to the plate engage holes in the bottom of the rocker, preventing the rocker from translating longitudinally or transversely relative to the plate. The rocker and plate drawings are reproduced in Figure 4.

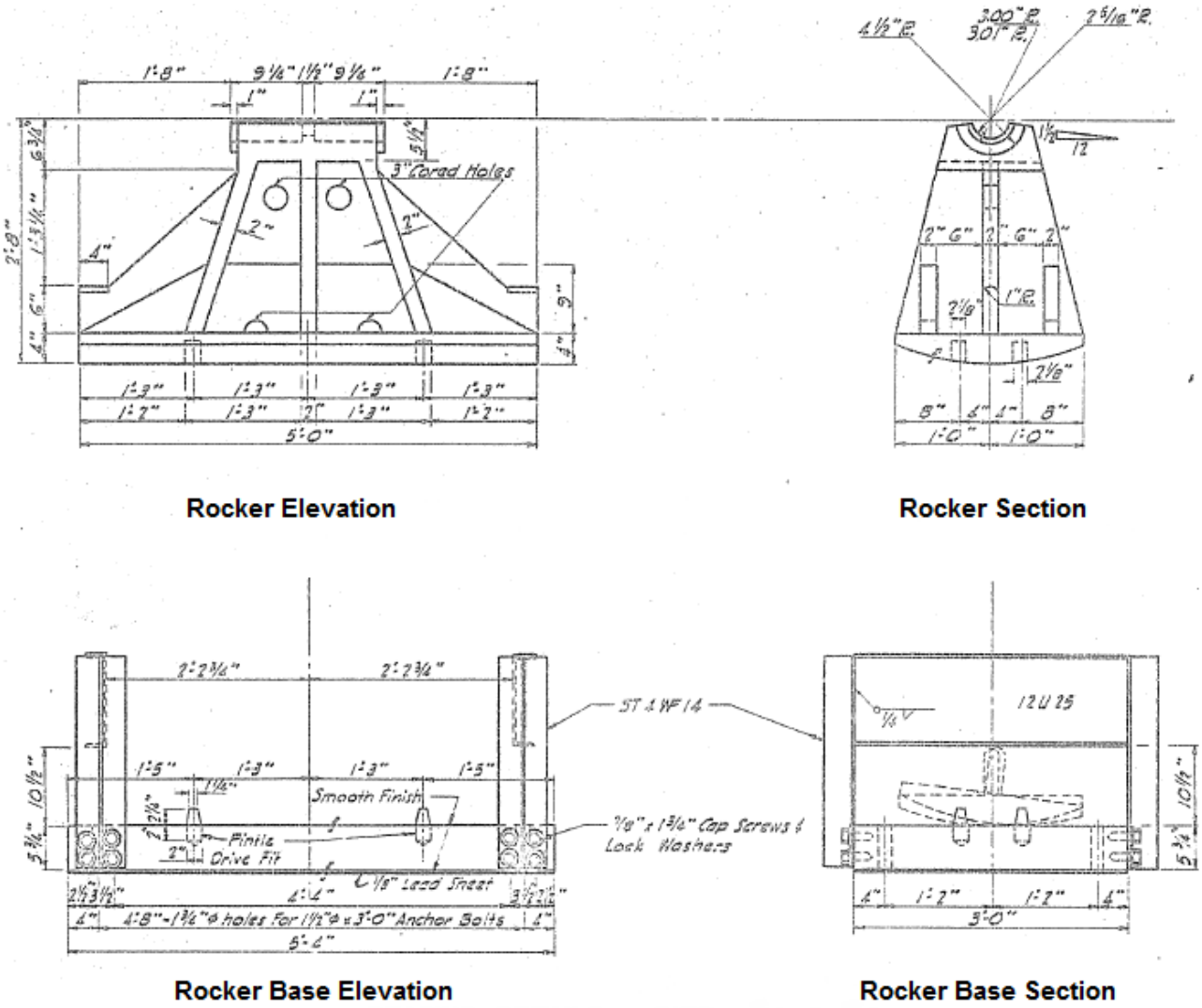


Figure 4 – Rocker and Rocker Base Detail

Bearing Movement

Variability of Bearing Movement Records

Examination of the records of the tilt of the north rocker bearing and the south rocker bearing at Pier 6 discloses that the tilt both increases and decreases between consecutive measurements. This can result from seasonal temperature variations that affect the length of the steel truss. A change in the temperature of the truss of 50 degrees F would result in a bearing movement of just over 1 inch.

The air temperature was recorded with each movement record. As it takes time for the bridge to respond to changes in air temperature, however, it is necessary to consider not only the recorded temperature but also the season in which the measurement was made. This allows judgment of whether the bridge may have been consistently warm or consistently cool long enough to have responded to the air temperature.

Variations in soil conditions that allow the pier to move both towards and away from the Red River may also affect the rocker tilt, although the slope inclinometer readings do not support movements of the soil mass away from the river.

Given the variability of the readings, it is prudent to evaluate trends in rocker bearing tilt rather than relying on specific measurements. Trends may be more revealing than individual year-to-year measurements.

First Bearing Movement Records

The first movement record consists of measurement of tilt of the rocker bearings on 10 September 1997. A note is included stating that a rocker base had been moved 5 inches prior to 1997. This note is reproduced in Figure 5; the pertinent statement is highlighted in a red box. The direction of the move was not noted, but is assumed to be west (away from the river) and from field measurements of 2012 (Figure 5) this adjustment was made to both bearing plates.

	A	B	C	D	E	F	G	H	I	J	K	L	M
1	Bridge # 9090 Rocker Measurements												
2	Tilt of Rocker from Plumb												
3													
4			Location										
5	Date	Temp, ^{0}F	NE	Dir	SE	Dir	NW	Dir	SW	Dir	Total Movement	Increment	
6												Prior move approx 5 inches.	
7	9/10/1997	80	3.13	E	2.13	E	3.63	W	7.50	W	0.00		
8	6/10/1998	61	2.25	E	1.94	E	2.81	W	7.50	W	0.00		
9	5/24/1999	55	2.25	E	1.63	E	2.75	W	3.75	W	9.75	moved baseplate 6 inches	
10	7/25/2000	80	2.75	E	1.88	E	3.38	W	4.38	W	10.38		

Figure 5 – Note identifying rocker base move.

A second bearing adjustment was recorded on 24 May 1999. This adjustment consisted of relocating the rocker base 6 inches. Based on the field measurements of 2012 (Figure 5), this is understood to refer only to the bearing plate for the south side of the bridge.

In June of 1998, the south rocker bearing was tilted 7 ½ inches, with the top of the bearing west of the bottom of the bearing. After relocating the south bearing plate in 1999, the south rocker bearing remained tilted 3 ¾”. This implies that the south side of the pier moved eastward relative to the truss approximately 2 1/4” between 1998 and 1999.

Later Bearing Movement Records

After the bearing plate relocation in 1999, the tilt of the south rocker bearing continued to increase. The tilt of the north rocker bearing remained essentially constant.

The top of the south rocker bearing was relocated eastward 7 inches along the truss bottom chord in July of 2004. This resulted in a tilt of the rocker bearing of 1 3/8 inches, oriented with the top of the rocker west of the bottom of the rocker. This implies that the top of the pier moved relative to the steel truss approximately 2 inches between the May 2003 measurement and the July 2004 bearing adjustment.

Latest Bearing Movement Records

After the bearing relocation in July of 2004, the tilt of the south rocker bearing continued to increase. The tilt of the north rocker appears to have increased by a much smaller amount than did the tilt of the north rocker.

The top of both the north rocker and the south rocker were moved eastward 7 inches along the truss chord in July of 2012. After this adjustment, the north rocker bearing had been adjusted a total of 7 inches and the south rocker bearing had been adjusted a total of 14 inches.

After the July 2012 bearing adjustments, the south rocker bearing was essentially plumb. The north rocker bearing was tilted with the top of the bearing east of the bottom of the bearing.

The south rocker bearing was tilted 6 ¾ inches to the west two months prior to the adjustment. The 7 inch adjustment brought the rocker close to plumb.

The north rocker bearing was 3 inches to 4 inches out of plumb prior to the bearing adjustment, and was 1 $^{3}/_{8}$ inches out of plumb in the opposite direction after the bearing adjustment.

Current Bearing Conditions

The most recent bearing measurements were obtained in October of 2012. Those measurements show the north rocker bearing tilted 2 inches to the east and the south rocker bearing as plumb. These readings are within the operating range of the bearings.

Figure 6 shows the field measurements locating the rocker bases on the pier cap. This diagram is copied from the MnDOT field notes. Figure 7 contains a photograph of the south rocker bearing, with the rocker base movement noted.

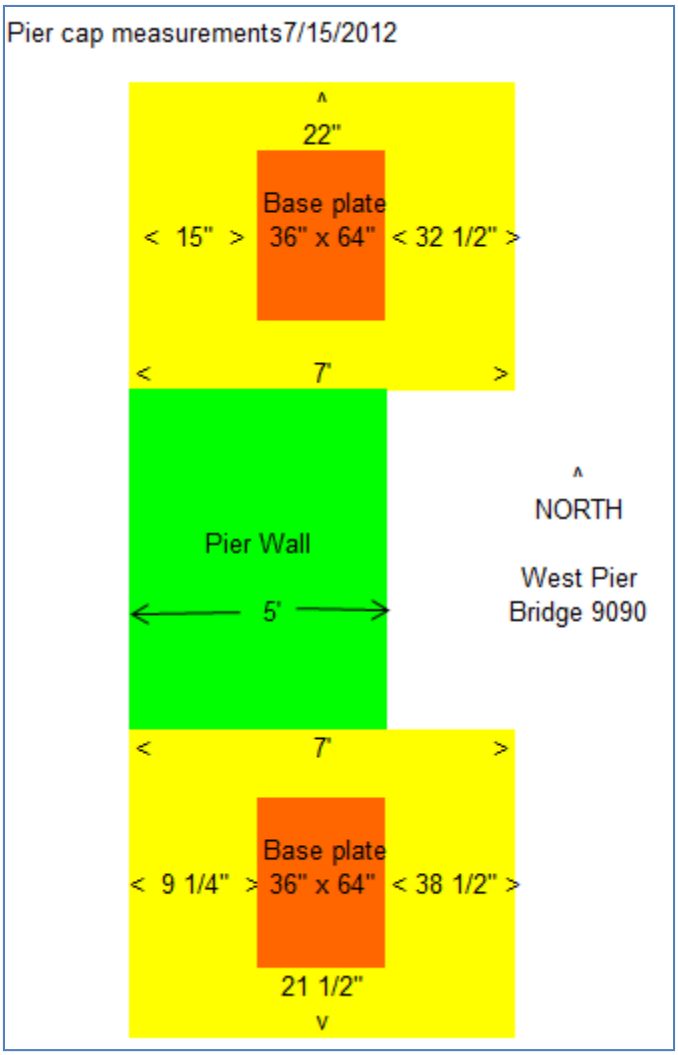


Figure 6 – Field Measurements of Pier Cap

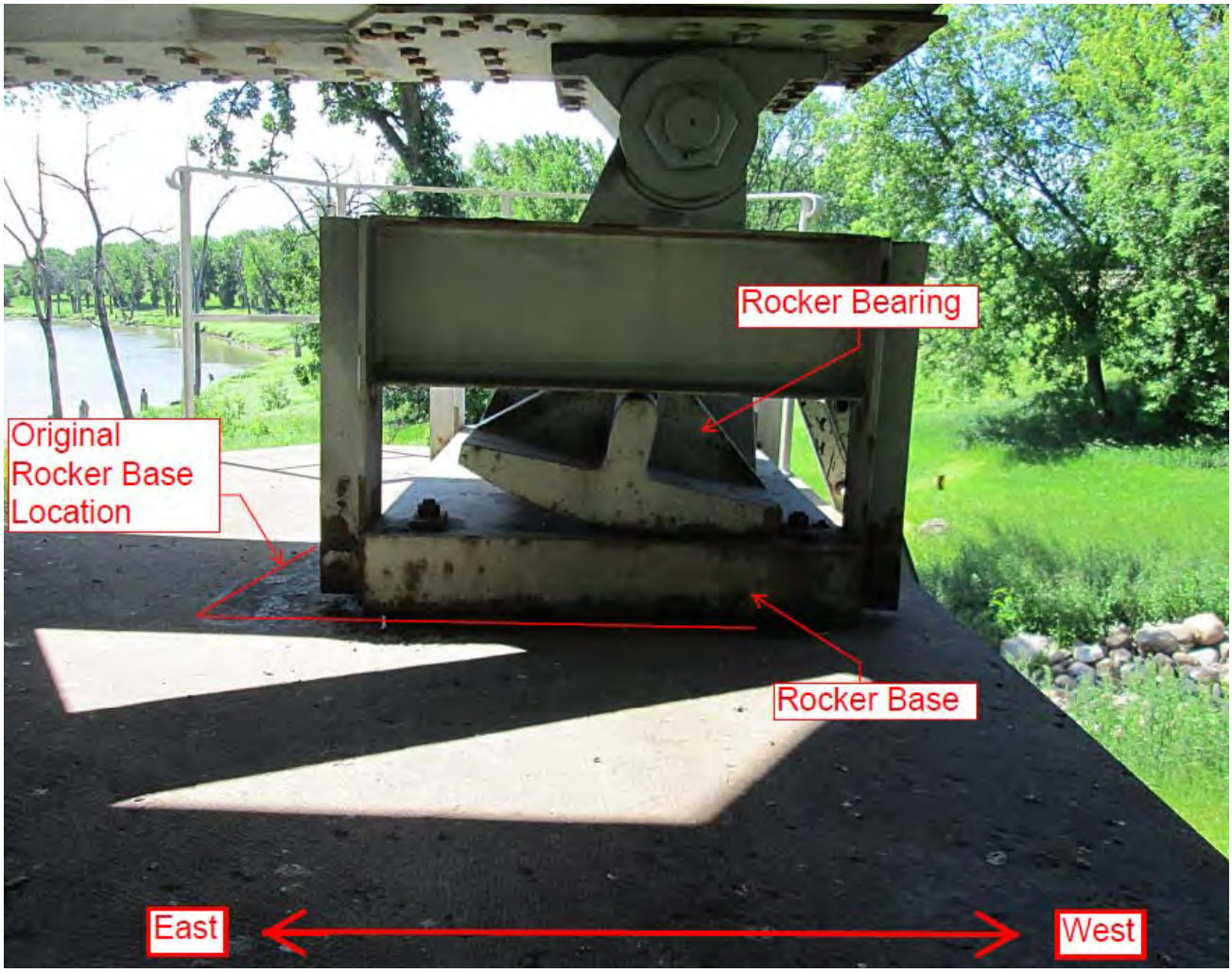


Figure 7 – South Rocker Bearing, Pier 6, looking south along top of pier

Rate of Bearing Movement

The rate of movement of the rocker bearing is relatively uniform over the time in which MnDOT has been monitoring the movement of the bearing. Bearing adjustments have been made prior to 1997, in 1999, in 2004, and in 2012. Adjustment of the bearings has been infrequent and irregular, and there is insufficient information to determine a trend in bearing adjustments.

Bearing Movement Diagram

As an aid to interpreting the movement of the rockers and the rocker bases over time, a schematic diagram of the rockers, the rocker bases, and the center of Pier 6 relative to the end of the truss is presented in Figure 8. For each of five separate dates, an elevation diagram of each bearing is shown. A plan diagram of the two bearings is also shown.

The information is presented as stick diagrams for clarity, as a scaled image of the structure would make it difficult to see the movement of the various elements. Elements are labelled in one elevation diagram and in one plan diagram.

The first diagram shows the bridge as constructed. The rockers are plumb, and the workpoint at the end of the truss is centered over the rocker bases. The rocker bases are aligned with the center of bearing on the concrete pier. The center of the pier is aligned with the end of the truss.

The second diagram represents the first bearing measurement, dated 9/10/1997. The rocker bases at both north and south bearings have been moved approximately 5 inches west on the top of the pier. The north rocker is inclined such that the top of the rocker is 3.63 inches west of the center of the rocker base. The south rocker is inclined with the top of the rocker 7.5 inches west of the center of the rocker base. The plan diagram shows the top of the pier twisted about a vertical axis.

The third diagram represents the bearings on 5/24/1999. The rocker base at both north bearing is still 5 inches west on the top of the pier, but the south rocker base has been moved an additional 6 inches westward for a total movement of 11 inches. The north rocker is inclined such that the top of the rocker is 2.75 inches west of the center of the rocker base. The south rocker is inclined with the top of the rocker 3.75 inches west of the center of the rocker base. The plan diagram shows the top of the pier twisted further about a vertical axis.

The fourth diagram represents the bearings on 7/7/2004. The rocker base at both north bearing is still 5 inches west on the top of the pier and the south rocker base is still 11 inches westward. The north rocker is inclined such that the top of the rocker is 3.25 inches west of the center of the rocker base. The south rocker has now been moved 7 inches along the bottom chord of the truss, and is inclined with the top of the rocker 1.38 inches west of the center of the rocker base. The plan diagram shows the top of the pier twisted further about a vertical axis.

The last diagram in the series represents the bearings on 7/17/2012. The rocker base at both north bearing is still 5 inches west on the top of the pier and the south rocker base is still 11 inches westward. The north rocker has been moved 7 inches along the top chord, and is inclined such that the top of the rocker is 1.38 inches east of the center of the rocker base. The south rocker has now been moved an additional 7 inches along the bottom chord of the truss for a total of 14 inches, and is inclined with the top of the rocker 0.18 inches west of the center of the rocker base. The plan diagram shows the top of the pier twisted further about a vertical axis.

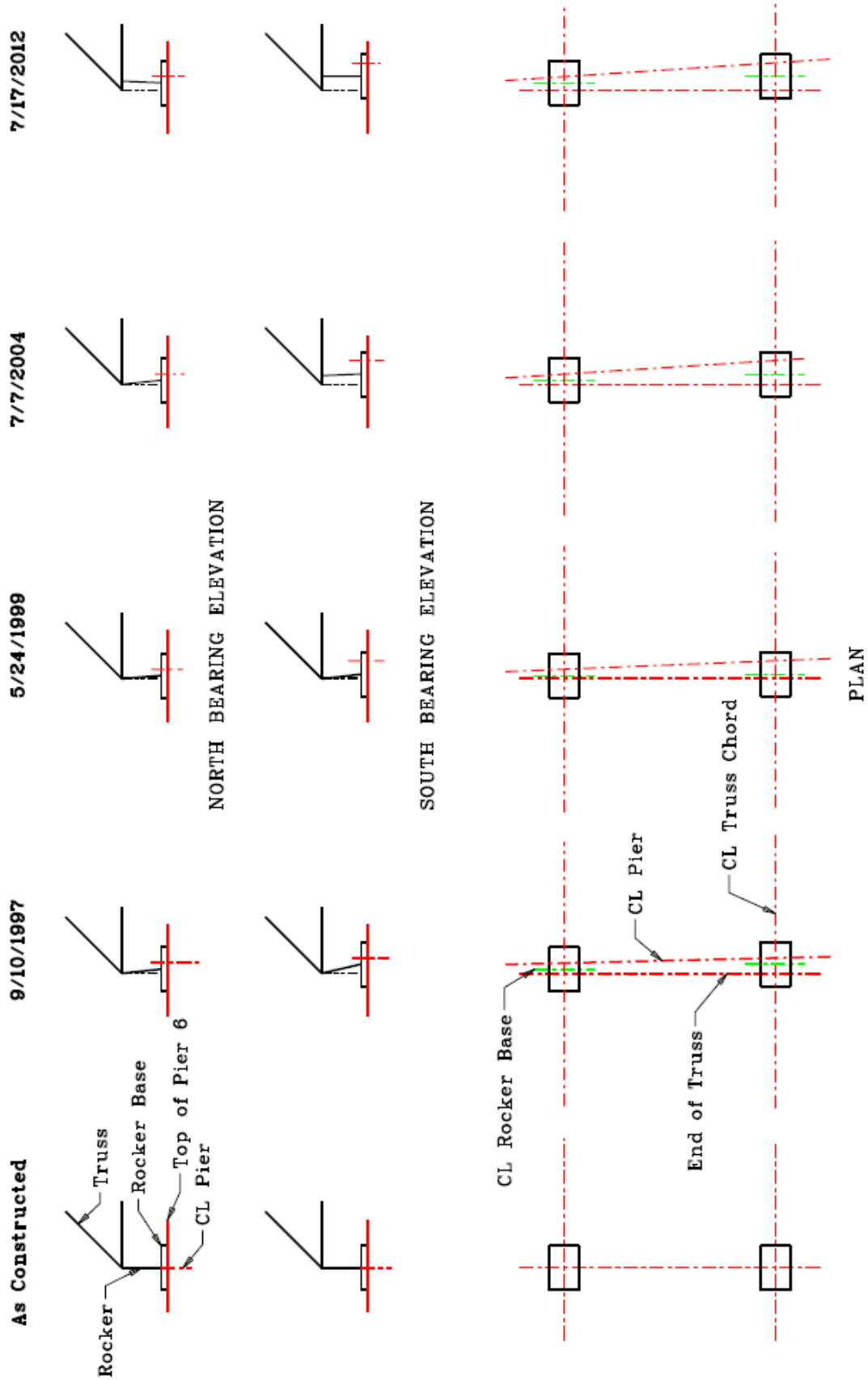


Figure 8 – Schematic of Bearing Movement over time

Pier Displacement Records

Bent 5 to Pier 6 Distance

Periodic measurements were made of the distance between Bent 5 and Pier 6, beginning in September of 2010 and extending to October 2012. These measurements are reproduced in Appendix B. Locations of measurements are not indicated in the movement tables.

The distance between Bent 5 and Pier 6 varies over time. The distance increases and decreases, as shown in the plot in Figure 9. This plot shows only the variation in distance, and is created by subtracting the initial value (the dimension on 9/23/2010) from the dimensions recorded by MnDOT. For the north measurements the value 72.35 is subtracted; 63.35 is subtracted from the center measurements, and 68.95 is subtracted from south measurements. This technique accentuates the differences between measurements.

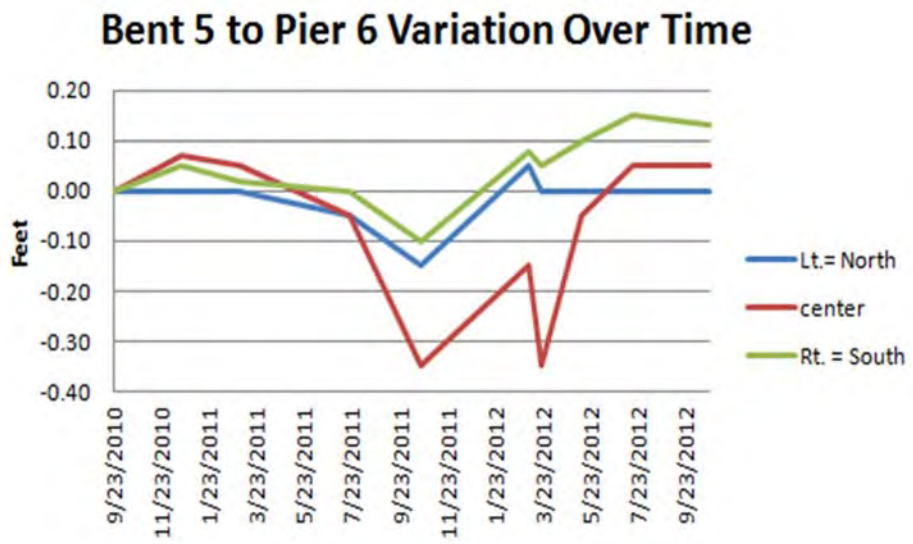


Figure 9 – Change in Distance between Bent 5 and Pier 6

A pertinent observation is that the distance between Bent 5 and Pier 6 both increases and decreases over time. There are some similarities in the movement curves; for example, the distance is a minimum for all three locations at the October 2011 measurement.

A second pertinent observation is that the distance between Bent 5 and Pier 6 at the north side of the bridge is more than 3 feet larger than the corresponding distance at the south side of the bridge. This difference is shown in the figure “Triangulation” in Appendix B as the difference between Series 2 and Series 8. This is the opposite of what is expected based on tilting of the rockers and tilting of the pier. This may be accounted for by differences in location of the measurements, but without specific information regarding the location of the measurements the significance of this difference cannot be estimated.

Bow in Pier 6 Wall

The distortion of the center wall between the north and south columns of Pier 6 was measured periodically, beginning in September of 2010 and extending to October 2012. Measurements are assumed to be in inches, measured from a horizontal stringline parallel to the concrete wall.

The distortion varies from 5.5 inches to 5.75 inches, and is observed to both increase and decrease. This variation in distortion of the center wall is not likely to be significant.

Pier Tilt

Detailed measurements of the tilt of Pier 6 were collected in July of 2012. Field notes for this measurement are contained in Appendix B.

The south column at Pier 6 is tilted at a slope of ½ inch horizontal per foot vertical. The top of the pier is further west (away from the Red River) than is the footing.

The north column at Pier 6 is tilted at a slope of $^{1}/_{8}$ inch horizontal per foot vertical. The top of the pier is further west (away from the Red River) than is the footing.

The pier height is 27 feet. Using the tilt as measured in July of 2012, the footing at the south of Pier 6 has moved approximately 13.5 inches relative to the top of the pier. The footing at the north of Pier 6 has moved approximately 3.4 inches relative to the top of the pier.

The measurements of rocker bearing tilt, which are described above, indicate that the top of Pier 6 is moving eastward relative to the steel truss. The pier tilt indicates that the pier footing is moving eastward relative to the top if the pier. This means that the movement at the rocker bearing and the footing displacement calculated based on the pier tilt are in the same direction.

Combining the rocker movement and the pier tilt, the footing under the north side of Pier 6 appears to have moved approximately 13.5 inches eastward relative to the steel truss above. The footing under the south side appears to have moved approximately 38.5 inches eastward relative to the steel truss above.

Figure 10 contains a summary of the movement of Pier 6 at the south side of the pier relative to the steel truss. This figure includes the effect of adjustment of the rocker base and rocker bearing.

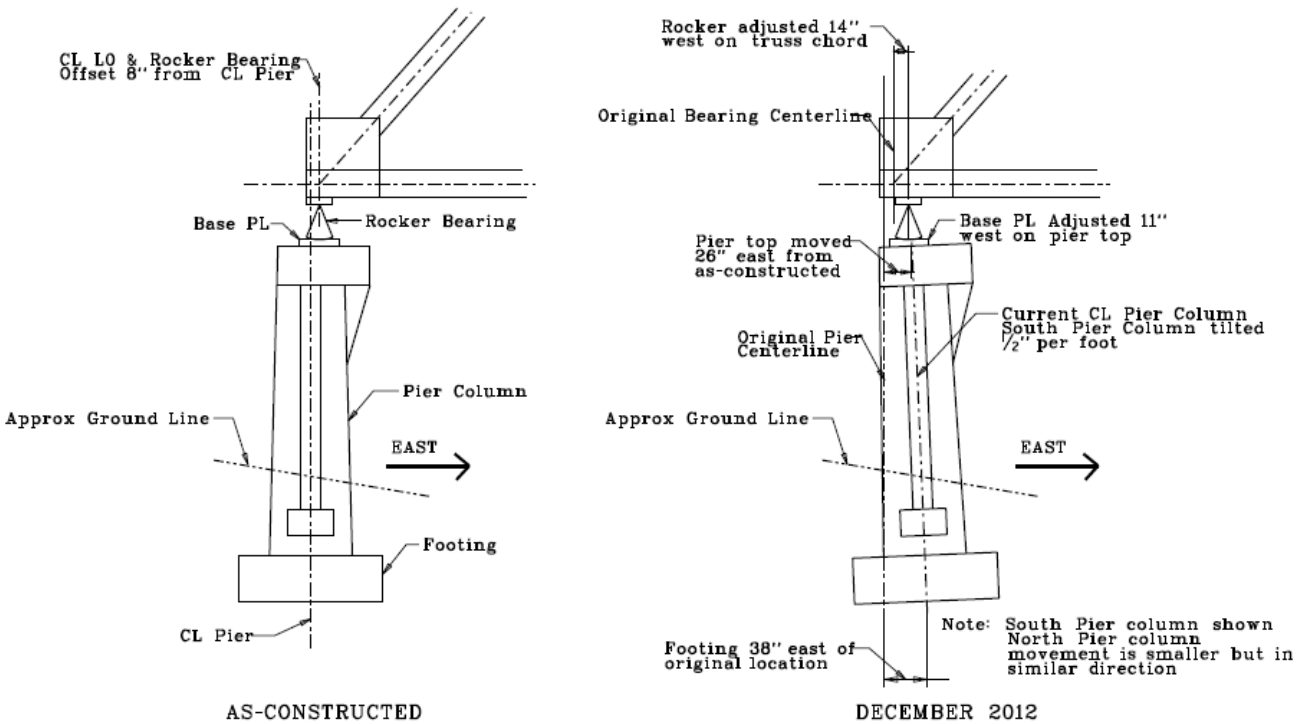


Figure 10 – Pier 6 Movement

Ground Movement Records

Three slope inclinometers were installed south of Pier 6. Records of the slope inclinometer results are shown in Appendix C. Slope inclinometer No. 1 was functional from June of 2004 through November of 2009. Slope inclinometer No. 3, installed adjacent to inclinometer No. 1, was functional from July 2010 and was still operational at the most recent reading in May of 2012.

Slope inclinometer No. 2 was distressed by buoyant forces. No valid readings were obtained from this device.

Inclinometers No. 1 and No. 3 provide readings in two orthogonal axes. The North Dakota Department of Transportation reports that the A-Axis is oriented approximately 131 degrees from true north, which is approximately 45 degrees from the longitudinal axis of the bridge. Positive movements are toward the Red River.

Both functioning inclinometers indicate a slip plane at a depth approximately 6 feet below ground surface and another slip plane approximately 27 feet below ground surface. The upper slip plane is above the pier footing, and the lower slip plane is approximately 17 feet below the pier footing. Relative displacement of the soil between the lower slip plane and the bottom of the inclinometers appears to be negligible.

Displacements in the A-Axis and the B-Axis are very similar in magnitude. The vector sum of movements in the A-Axis and the B-Axis is generally normal to the river and parallel to the longitudinal axis of the bridge.

Slope inclinometer data suggest that the ground and the bottom of Pier 6 sat the south side of the bridge have moved toward the river approximately 22 inches over the course of eight years. The movement has been fairly steady, at a little under 3 inches per year. Soil movement at a depth of 10 feet is significantly less, with a total movement of just under nine inches over the eight-year period of the movement records.

Evaluation of the slope inclinometer data presumes that the primary axes of inclinometers 1 and 3 are aligned. If this is the case, the direction of movement of the upper layer of soil (surface to a depth of approximately 6 feet) may be different than the direction of movement of the soil between depths of 6 feet and 27 feet. The difference in direction is approximately 20 degrees. If the A-axis and B-axis of inclinometers 1 and 3 are not aligned, the relative direction of movement of the two layers is inconclusive.

Appendix A – Rocker Bearing and Pier Movement

Appendix A contains summaries of rocker bearing movement, Pier 6 movement relative to Pier 5, and measurements of the bow in the web wall at Pier 6. Data was collected by MnDOT.

Page A-1 contains the rocker bearing movement measurements

Page A-2 contains distances between Piers 5 and 6, and measurements of the bow in the Pier 6 web wall.

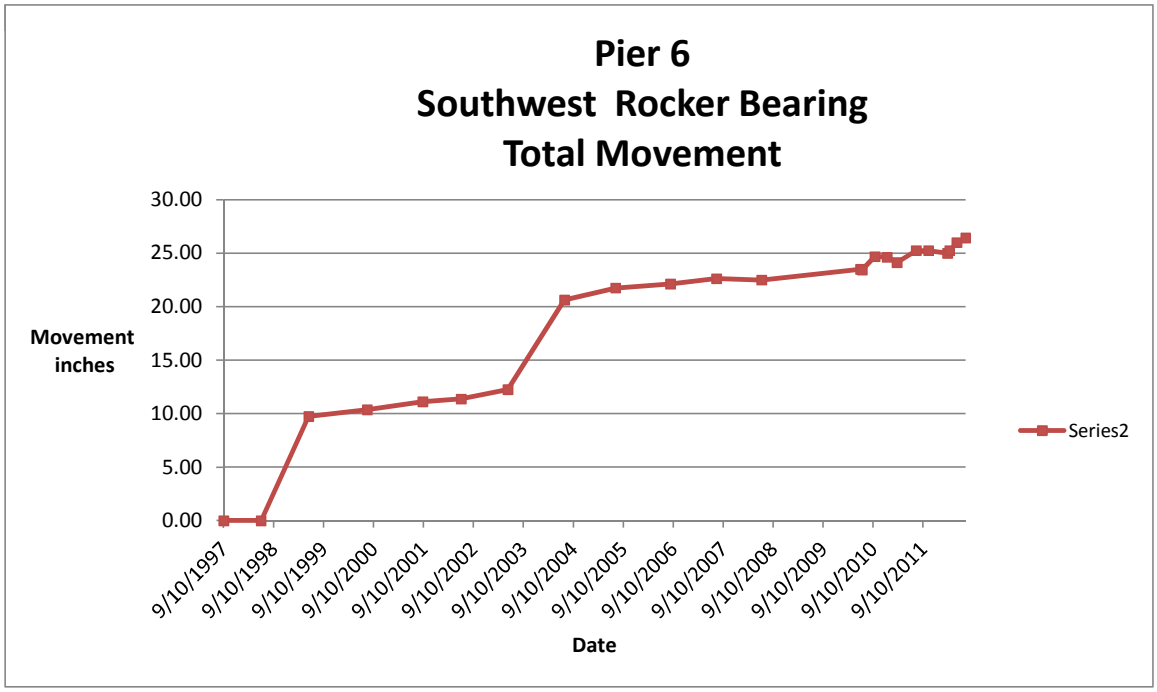
Page A-3 contains measurements of the rocker heel above the rocker base, for the northwest rocker, as taken on three dates in 2012.

The data is shown as provided by MnDOT. Page numbers were added by CH2M HILL.

Bridge # 9090 Rocker Measurements
Tilt of Rocker from Plumb

Date	Temp, ^{0}F	Location NE	Dir	SE	Dir	NW	Dir	SW	Dir	Total Movement	Increment
											Prior move approx 5 inches.
9/10/1997	80	3.13	E	2.13	E	3.63	W	7.50	W	0.00	
6/10/1998	61	2.25	E	1.94	E	2.81	W	7.50	W	0.00	
5/24/1999	55	2.25	E	1.63	E	2.75	W	3.75	W	9.75	moved baseplate 6 inches
7/25/2000	80	2.75	E	1.88	E	3.38	W	4.38	W	10.38	
9/4/2001	82	2.50	E	2.00	E	2.75	W	5.13	W	11.13	
6/12/2002	65	2.00	E	1.75	E	3.13	W	5.38	W	11.38	
5/20/2003	62	2.38	E	1.75	E	2.88	W	6.25	W	12.25	
7/7/2004	62	2.25	E	2.63	E	3.25w	W	1.38	W	20.63	Moved top of S rocker 7 inches
7/15/2005	74	2.25	E	2.00	E	3.38	W	2.50	W	21.75	
8/20/2006	80	2.50	E	1.88	E	3.25	W	2.88	W	22.13	
7/23/2007	88	2.13	E	1.75	E	1.63	W	3.38	W	22.63	
6/17/2008	75	2.13	E	1.63	E	3.375W		3.25	W	22.50	
6/22/2010	80	2.19	E	2.74	E	4.56	W	4.19	W	23.44	
6/9/2010	70					3.88	W	4.25	W	23.50	
9/23/2010	55						W	5.44	W	24.69	
12/20/2010	15						W	5.38	W	24.63	
3/3/2011	20						W	4.88	W	24.13	
7/22/2011	75					4.38	W	6.00	W	25.25	
10/20/2011	62					4.00	W	6.00	W	25.25	
3/7/2012	20					3.50	W	5.75	W	25.00	
3/22/2012	55					3.00	W	6.00	W	25.25	
5/14/2012	75						W	6.75	W	26.00	heel of rocker = 1/2 in. west / 4 7/8 in east
7/17/2012	80					1.38	E	0.18	W	26.43	Moved top of N & S rockers 7" W
10/24/2012	40					2.10	E	0.00		26.61	Oct 2012 SW rocker is plumb, but peir cap / base plate are at approx. 3.8 % grade causeing heel of rocker meas. to differ.

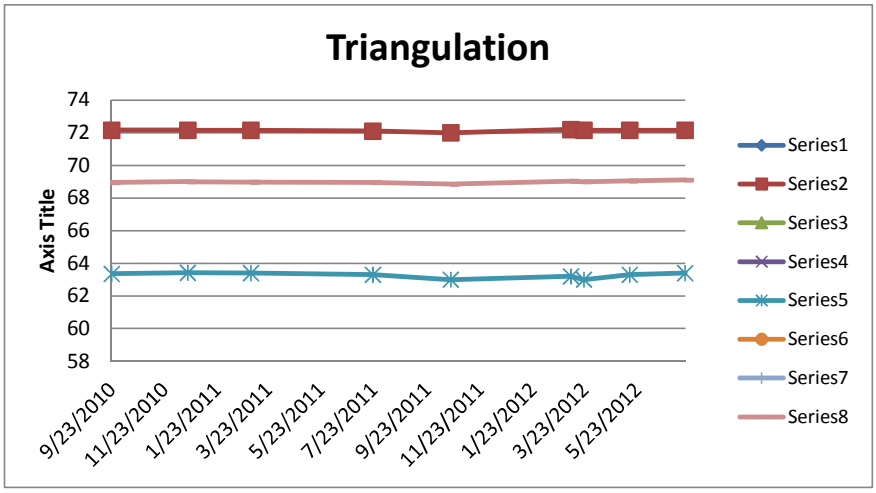
Note: As of 10-15-2010, the base plate had been moved approximately 11 inches, and the top of rocker approx 7 inches.



Stringline measurements from Bent 5 to Pier 6

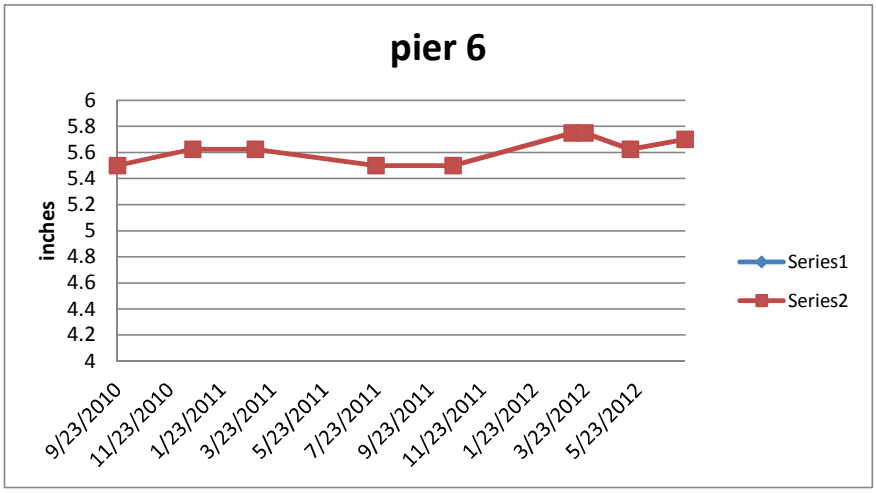
3 measurements from Bent 5 centerwall to bottom of Pie Pier 6 3 locations Rt. Center & Lt.

	Lt.= North	center	Rt. = South
9/23/2010	72.15	63.35	68.95
12/20/2010	72.15	63.42	69.00
3/3/2011	72.15	63.4	68.97
7/22/2011	72.10	63.3	68.95
10/20/2011	72.00	63	68.85
3/7/2012	72.20	63.20	69.03
3/22/2012	72.15	63	69.00
5/14/2012	72.15	63.3	69.05
7/17/2012	72.15	63.4	69.10
10/24/2012	72.15	63.4	69.08



pier 6 meas. Bow in wall

9/23/2010	5.500
12/20/2010	5.625
3/3/2011	5.625
7/22/2011	5.500
10/20/2011	5.500
3/7/2012	5.750
3/22/2012	5.750
5/14/2012	5.625
7/17/2012	5.700
10/24/2012	5.500



N.W. rocker heel measurement in inches

	east	west
march 22 nd 2012	4.6	0.8
may 14 2012	4.25	1
Oct 24 2012	1.88	2.2

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Appendix B – Pier Movement

Appendix B contains field sketches of the tilt of Pier 6, and sketches of the measurements of the tilt of the rocker bearings, and a graphic of the location of the rocker bearings on the top of Pier 6. Three photographs of the bearing are included. Data and photos were collected by MnDOT.

Page B-1 through B-2 illustrate the tilt of Pier 6.

Page B-3 contains overall dimensions of Pier 6 as measured by MnDOT

Page B-4 illustrates the location of the rocker bases on the top of Pier 6.

Pages B-5 and B-6 contain photographs of the south end of Pier 6.

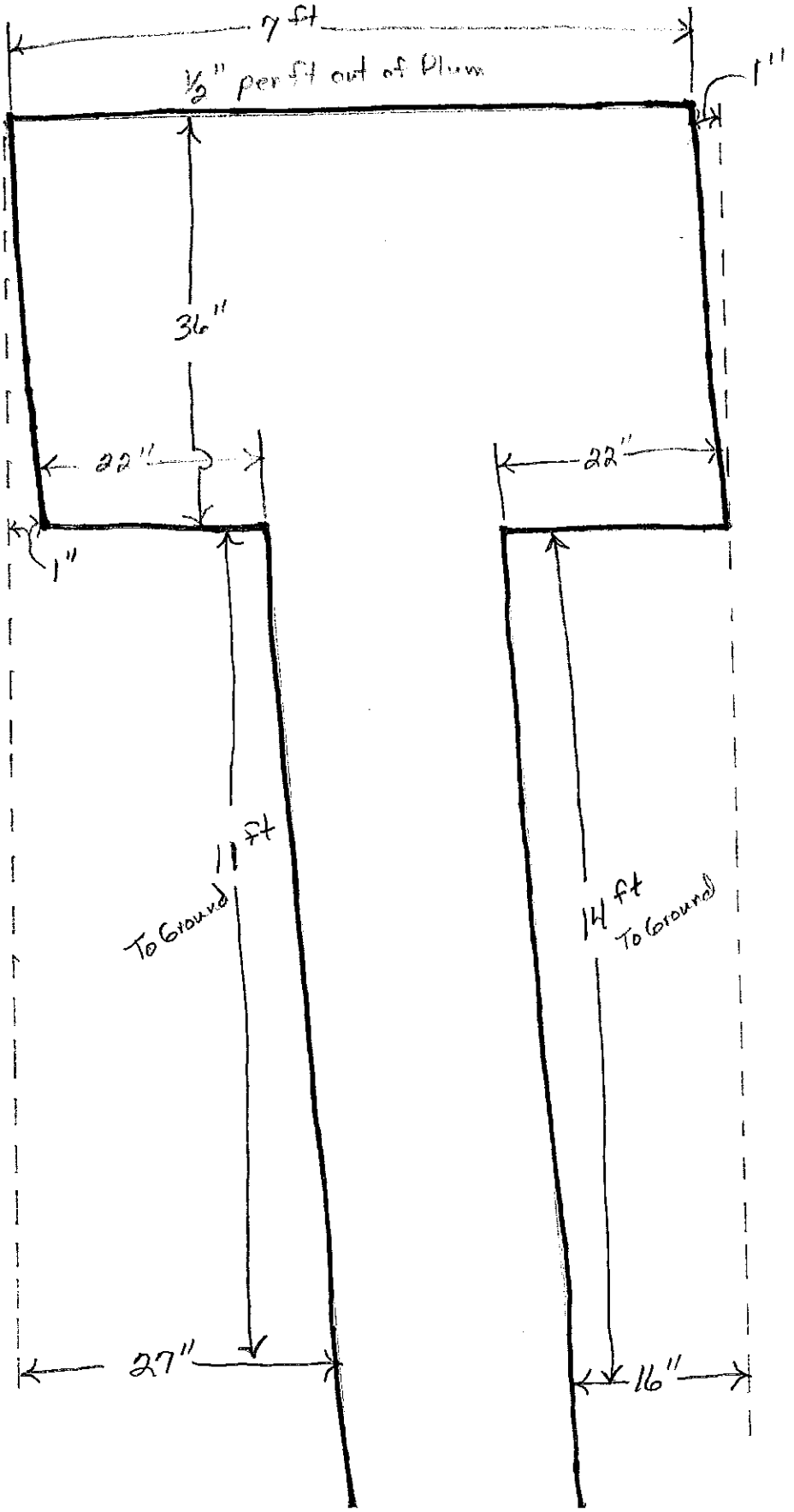
The data is shown as provided by MnDOT. Page numbers were added by CH2M HILL.

PIER 6 BR 9090
7/16/2012

FIELD MEASUREMENTS
TOM TUPPE

T2009090

South Side of Peir



MD side

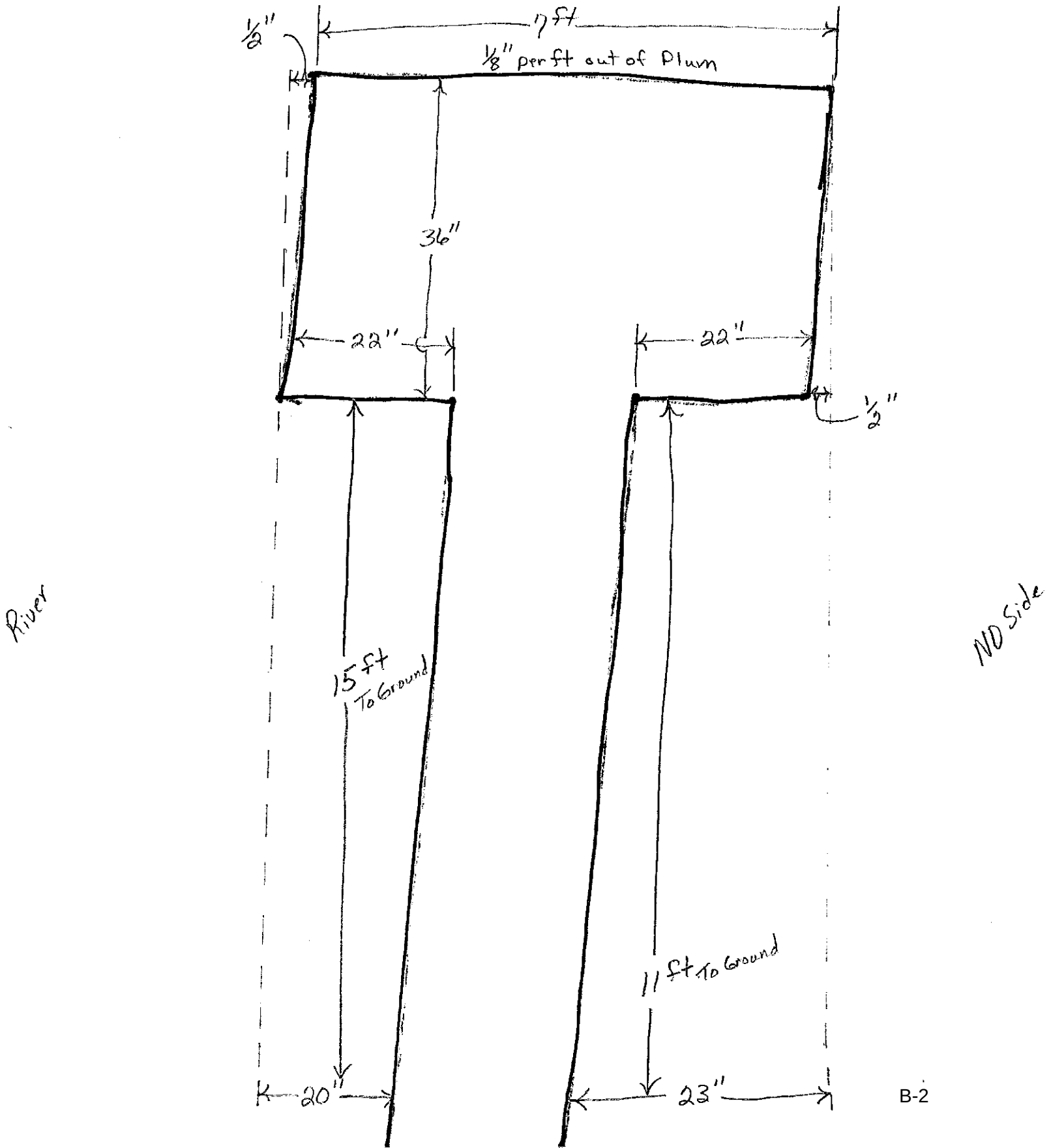
River

Pier 6
Bridge 9090
7/17/2012

FIELD MEASUREMENTS
TOM E. TUPPE

T5009090

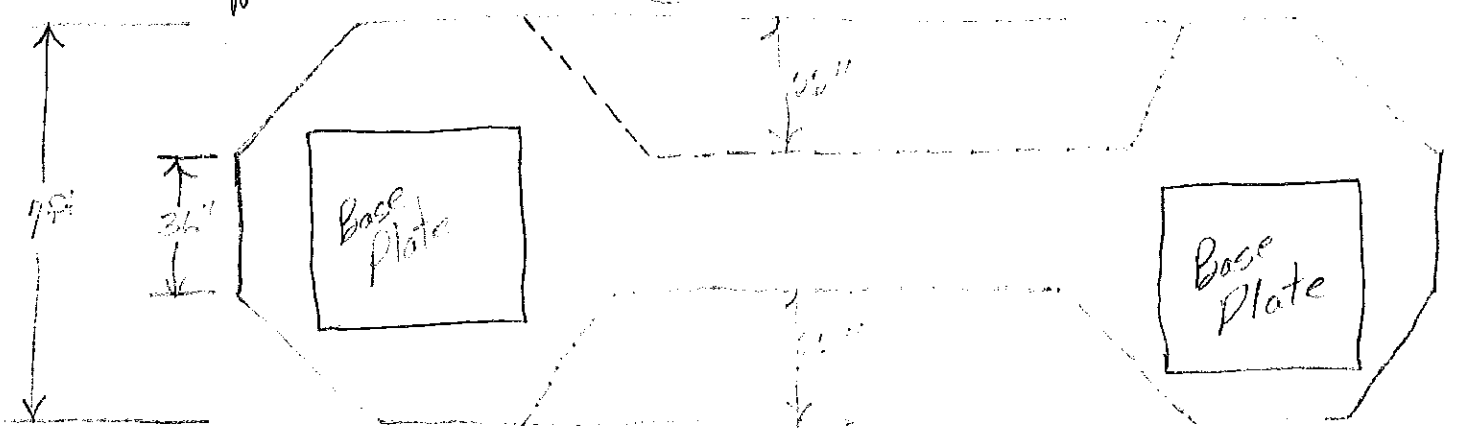
North Side of Pier

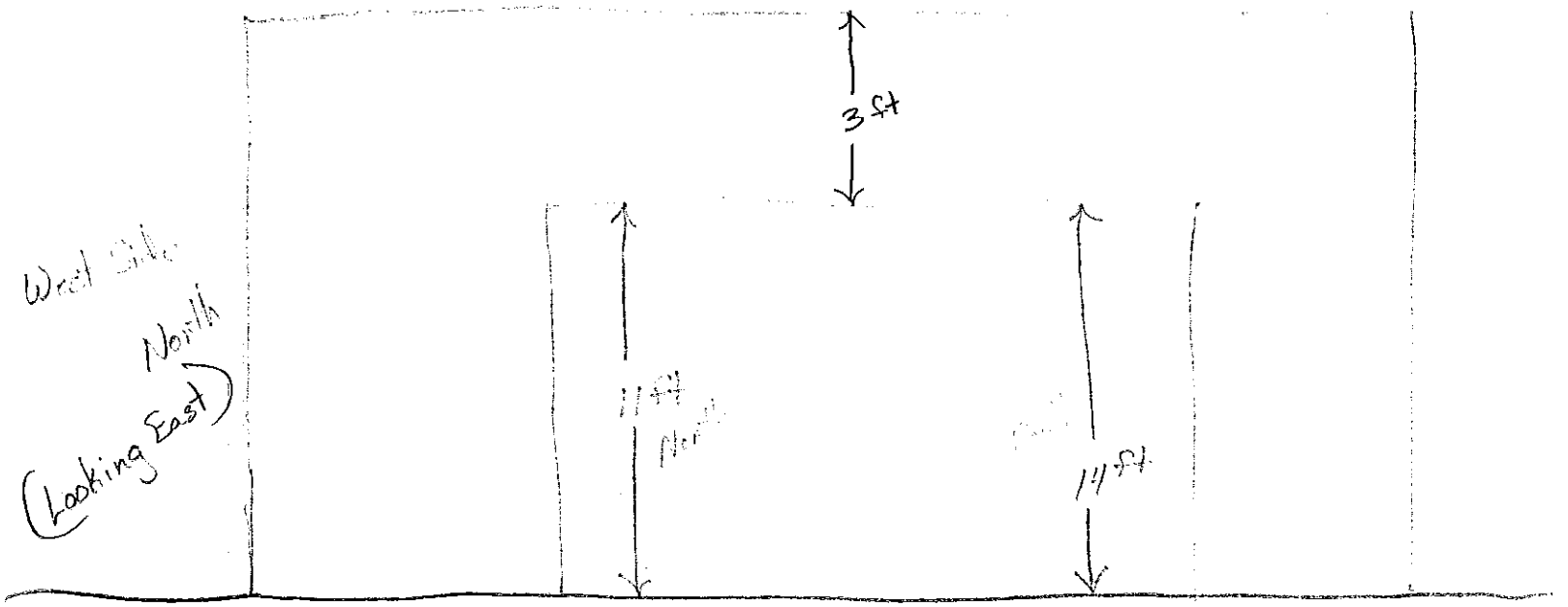


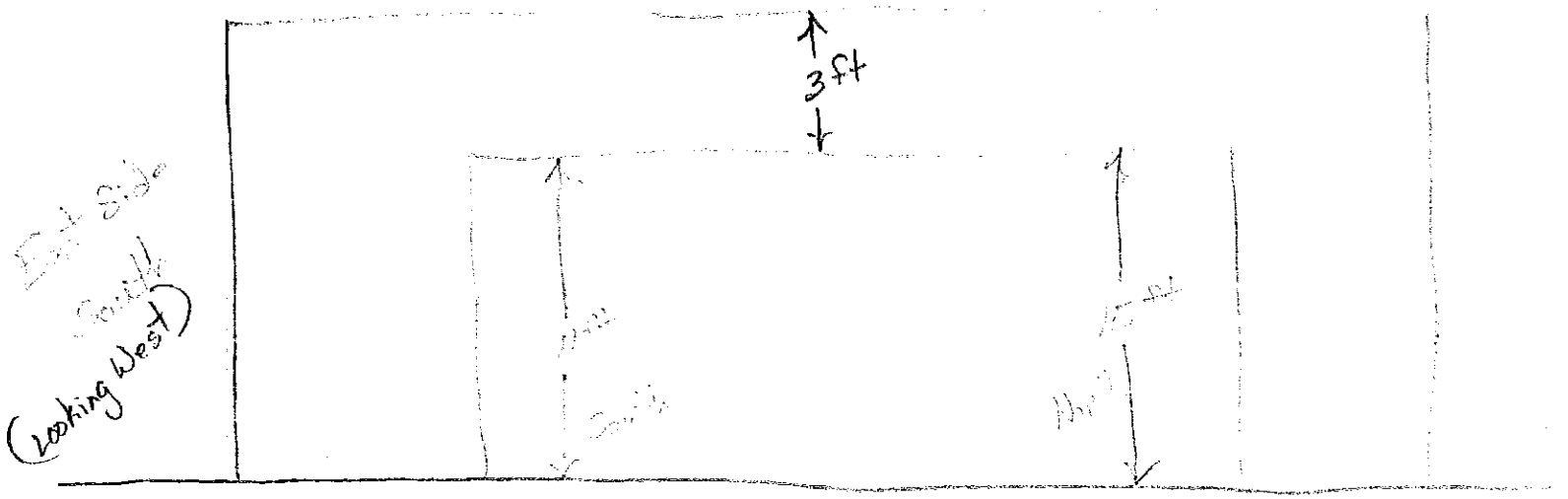
T5009090

PIER 6,
BR 9090
7-18-2012
(Top Looking Down)

FIELD MEASUREMENTS
TOM E. PUPPE







7/17/2012 AFTER RESET of BRG

PIER 6, BR 9090

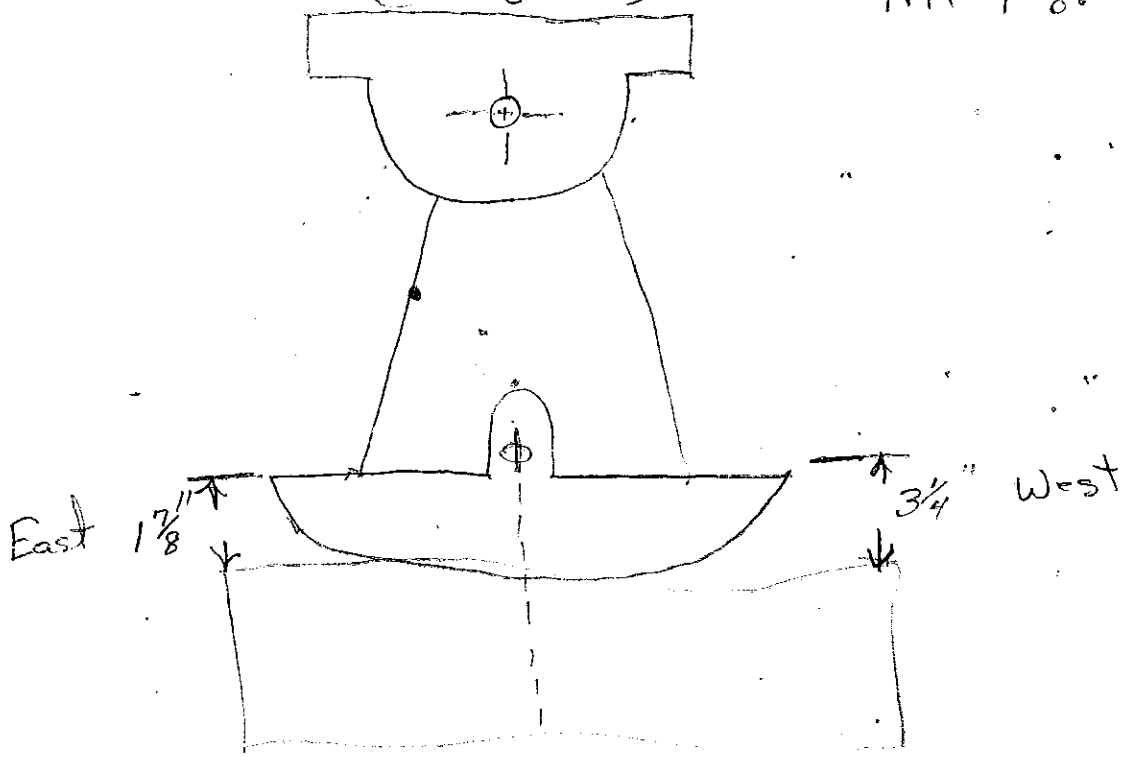
FIELD MEASUREMENTS
TOM PUPPE

as of 7-17-2012 TSP

TS009090

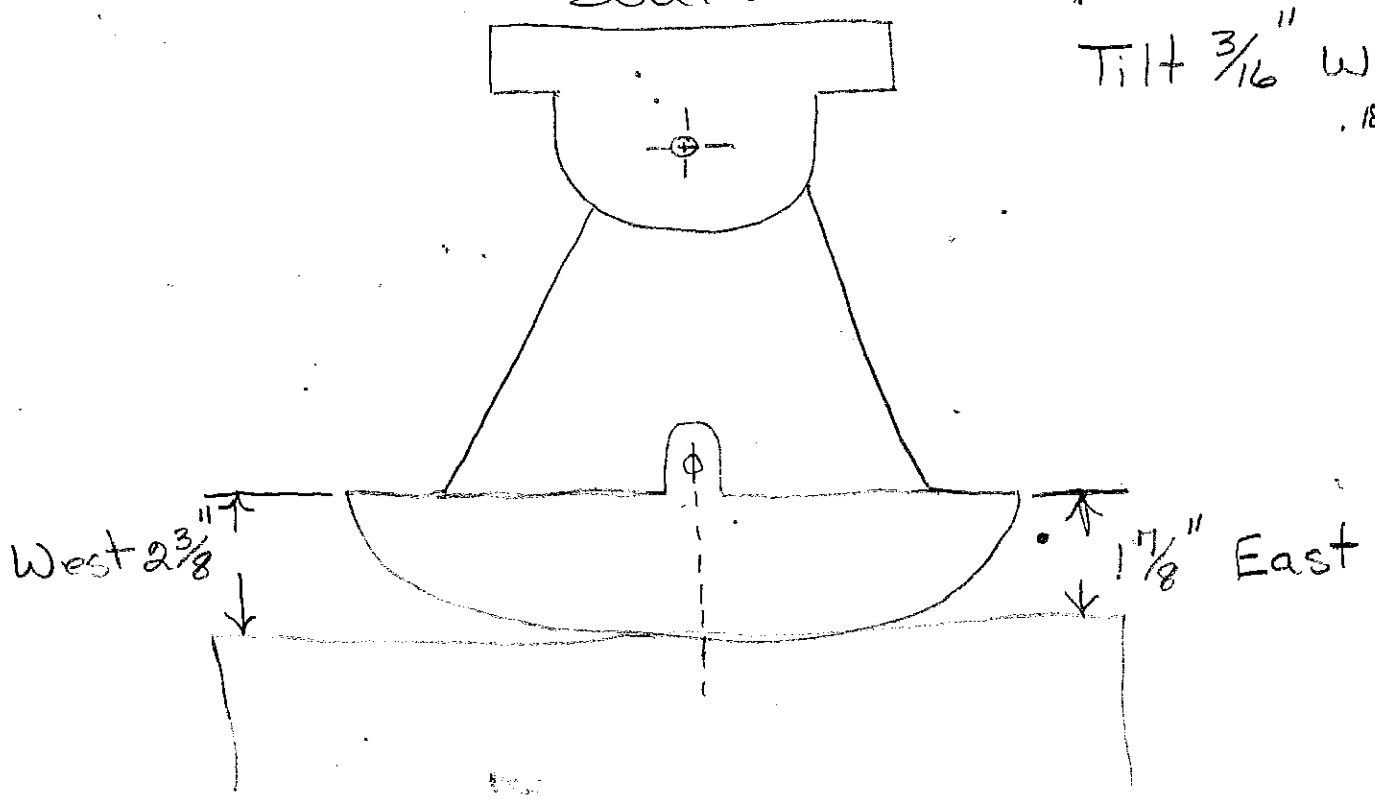
North Rocker
(Looking South)

Tilt 1 3/8" East

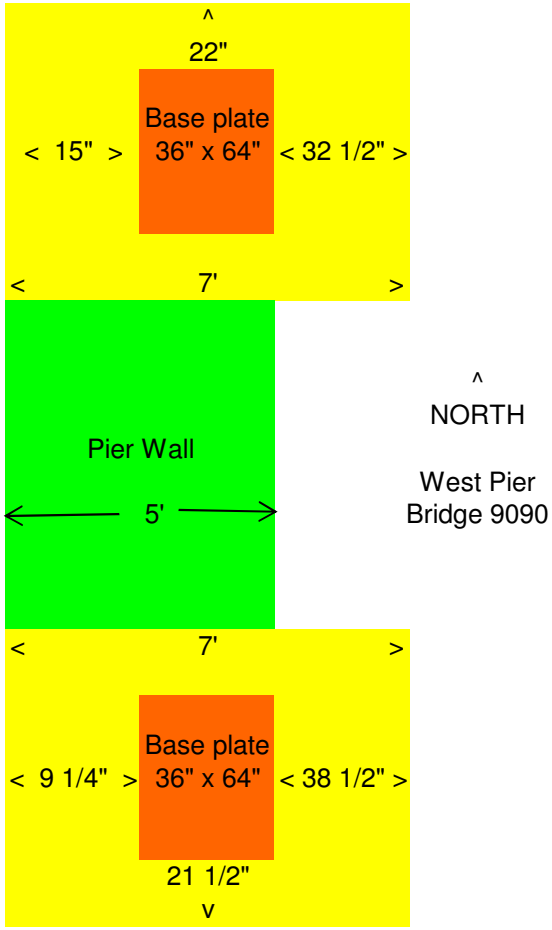


South Rocker (Looking North)

Tilt 3/16" West

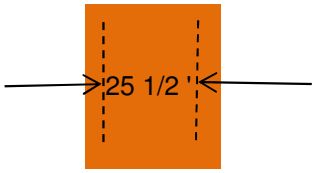


Pier cap measurements7/15/2012



83

Rocker bearing area is 25 1/2" wede



Rocker Bearing, South End of Pier 6

South End of Pier 6

L0 Connection at South End of Pier 6

Appendix C – Ground Movement

Appendix C contains a report prepared by the North Dakota Department of Transportation. This 8-page report describes installation and monitoring of slope inclinometers near Pier 6. The report was provided by MnDOT.

Kennedy Bridge

NDDOT Bridge No. 0002-358.090

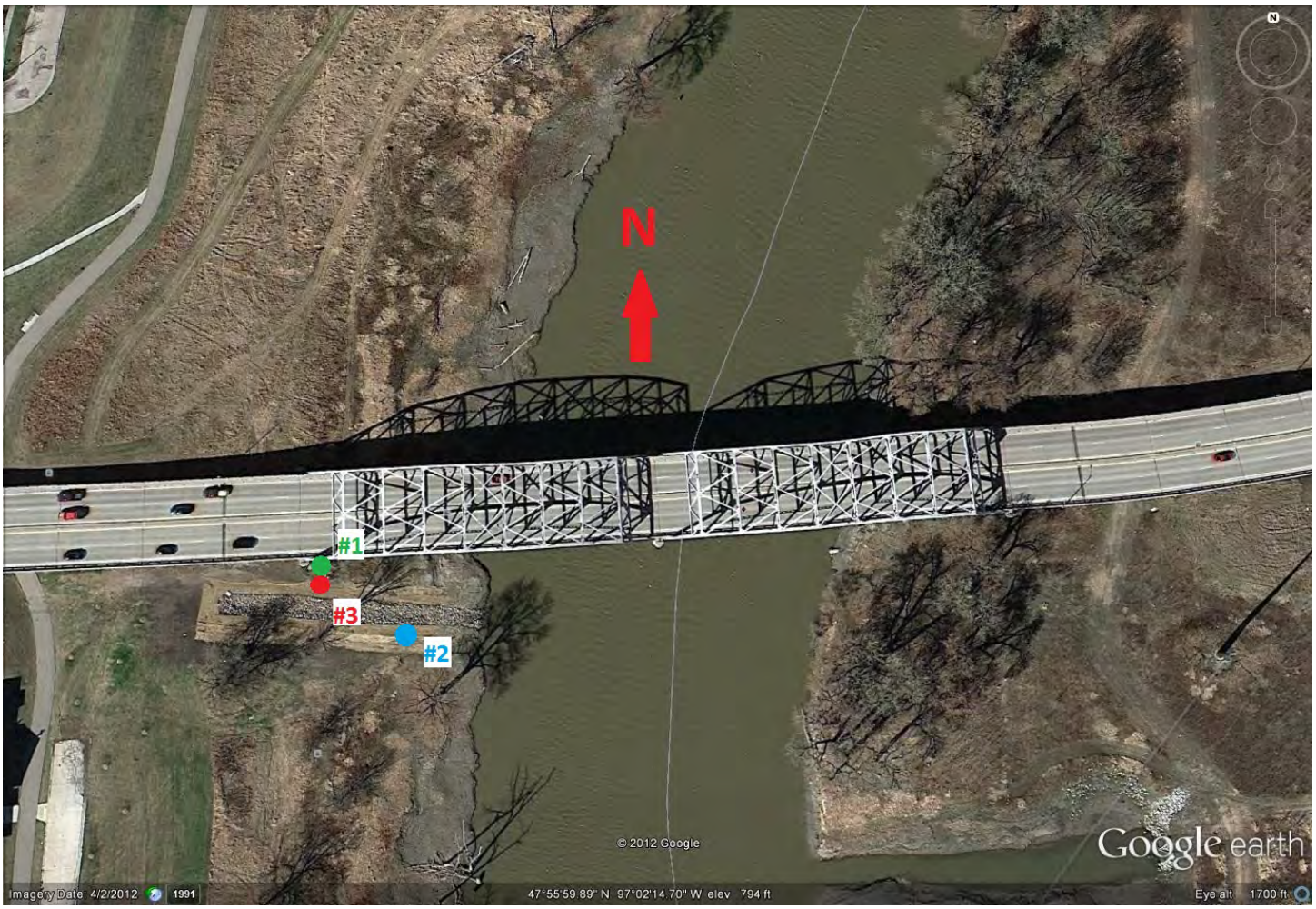
Hwy 2 RP 358.09

There is movement at the Kennedy Bridge West end-slope, and it is causing bent 5 and pier 6 to tilt.

Photo of Bent 5

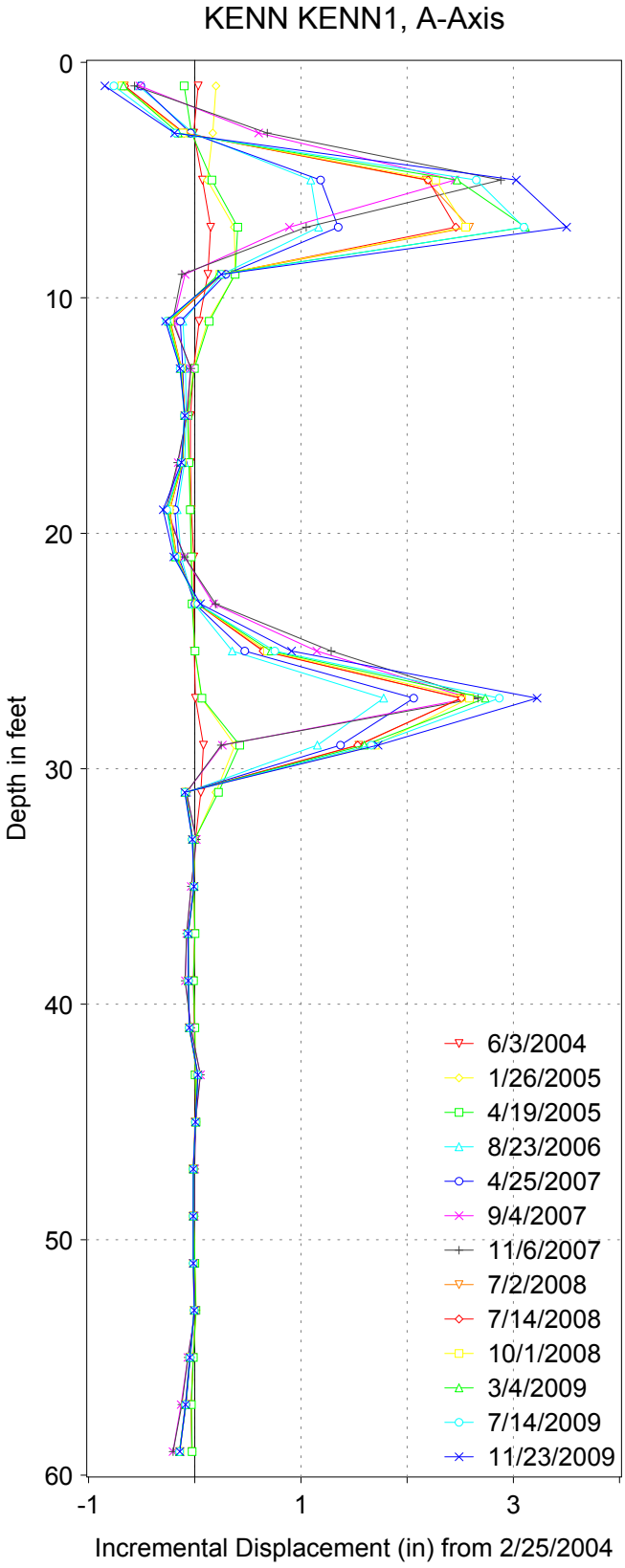
Investigation: The Corps installed three inclinometers in the Riverside Drive neighborhood which is just north of the Kennedy Bridge on the west bank. In the Corps investigation they discovered that, "this area has experienced foundation movements as evidenced by the existing failure scarp, pre-existing slickensides discovered during drilling, and the out of sequence geologic stratigraphy. Then larger noncircular failure surfaces that extended back to the secondary riverbank were analyzed."

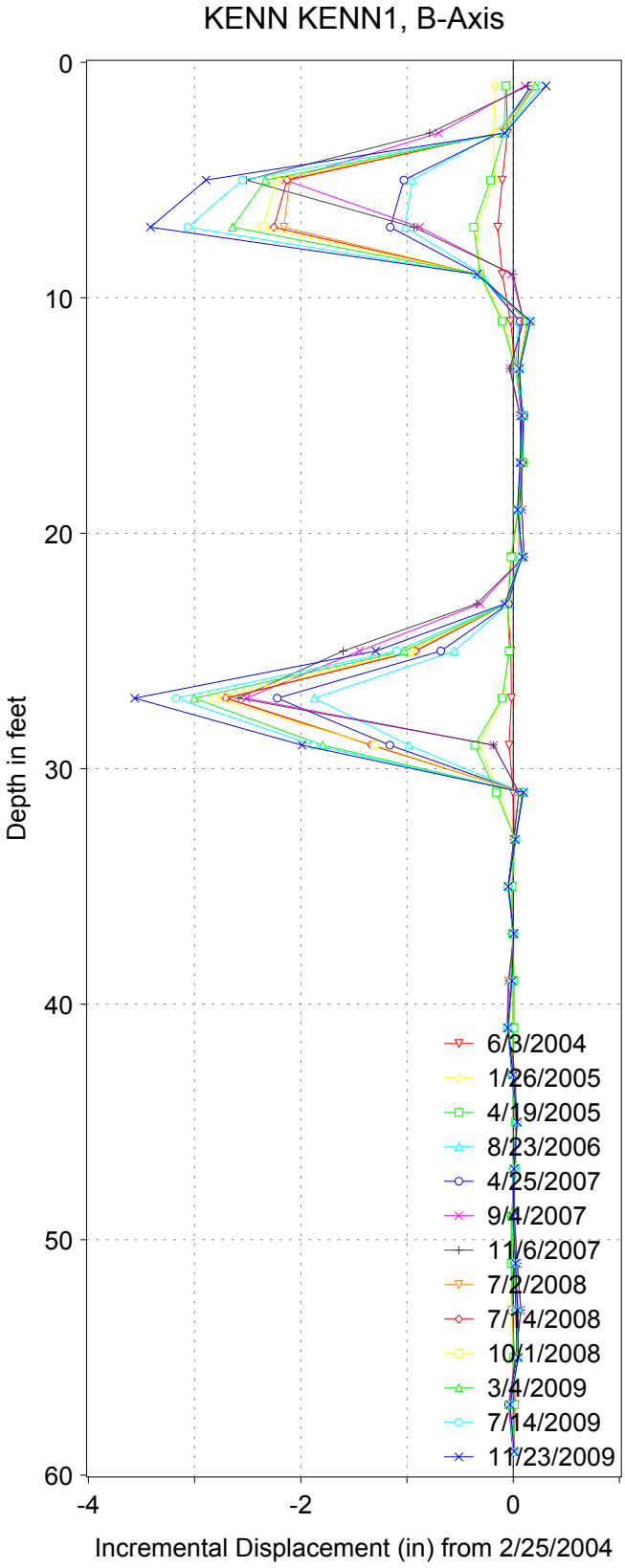
The NDDOT Geotechnical Section has monitored the west end-slope with inclinometers since 6/3/2004. We have installed three inclinometer tubes at this site. The tubes were placed on the south west side of the structure. The locations are shown in the following aerial image.



Aerial Image of Kennedy Bridge Showing the Inclinometer Locations

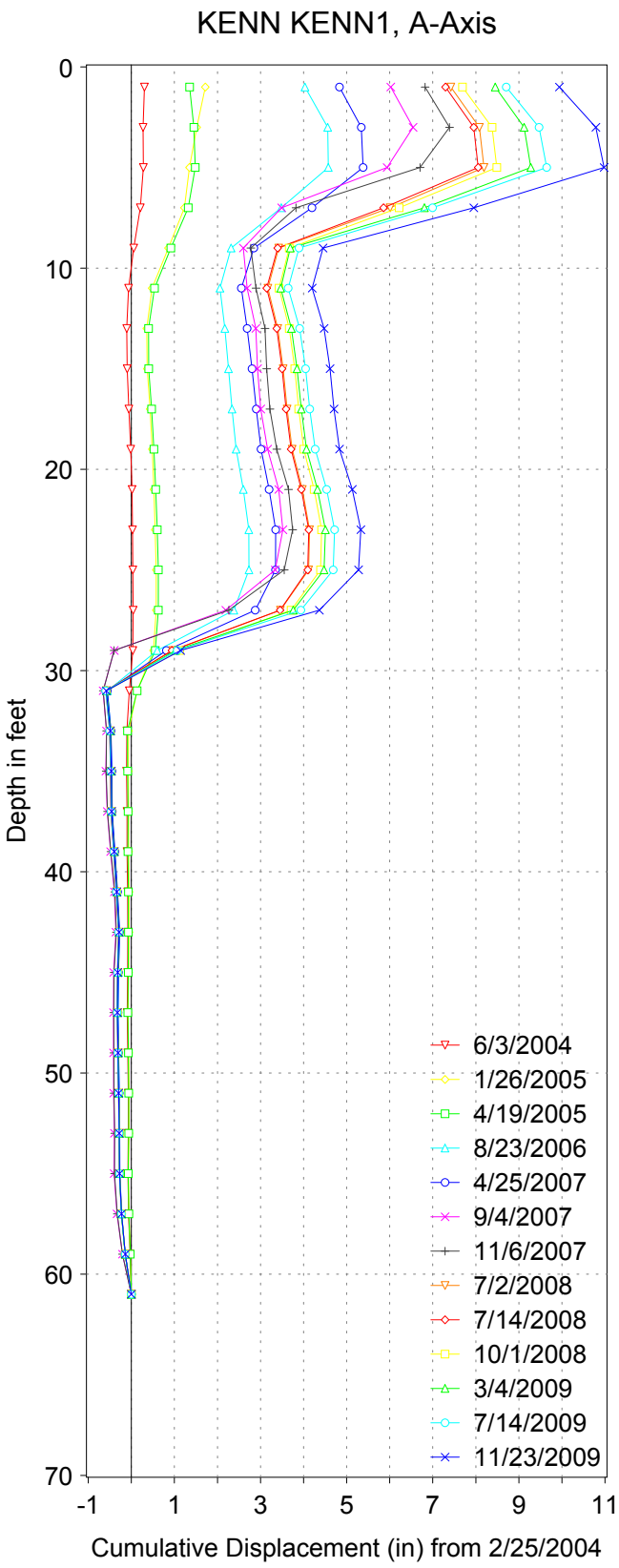
Tube #1 was sheared of in 2010 and tube #3 was installed to replace it. Tube #2 was installed, but it did not generate any valid readings as it was distressed by buoyant forces. The data from tubes #1 and tube #3 are on the following pages.

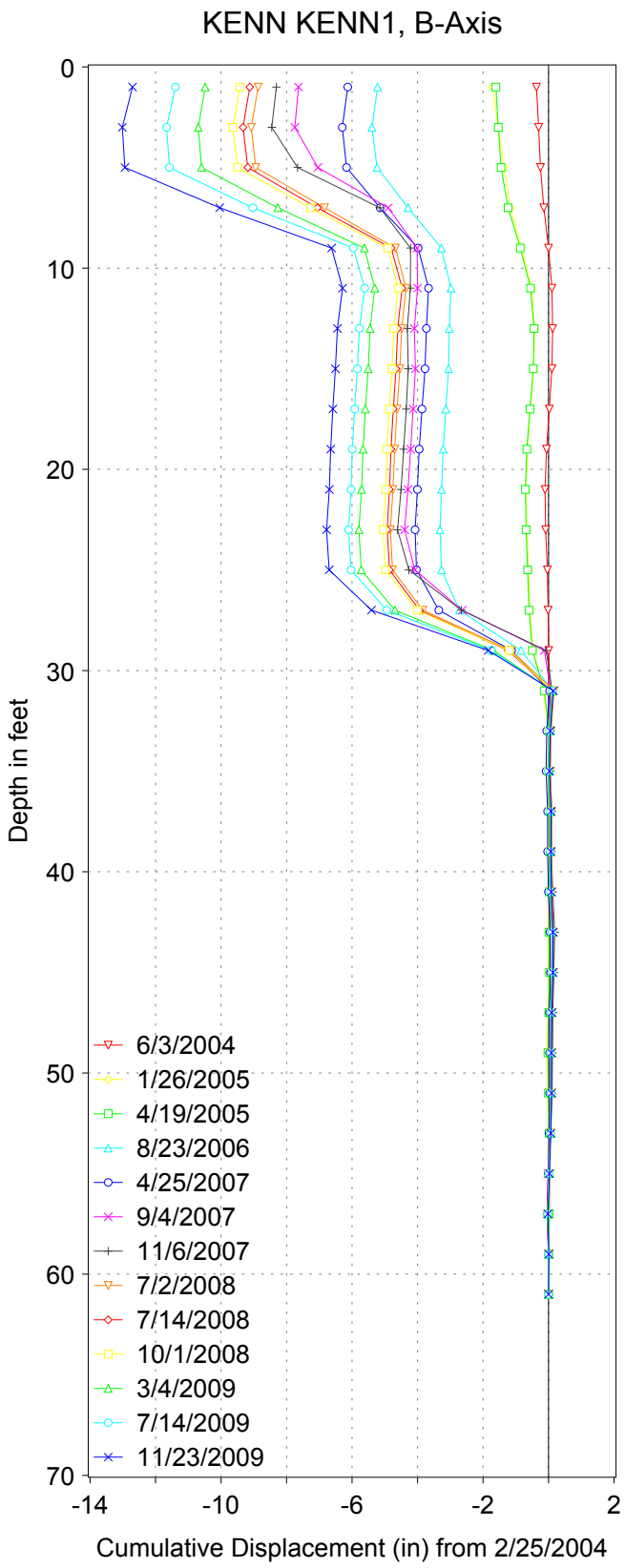




Incremental Displacement

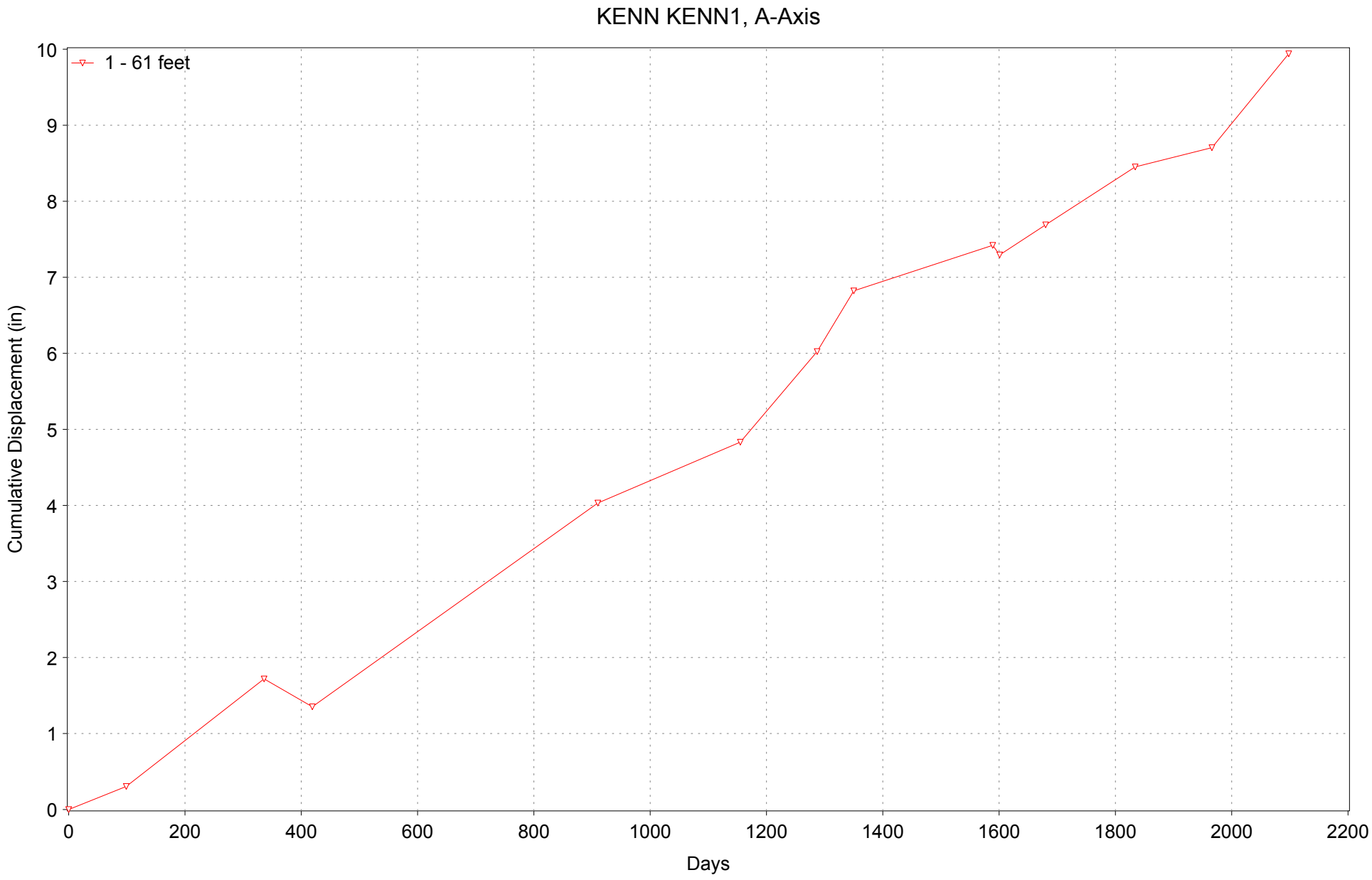
Tube #1





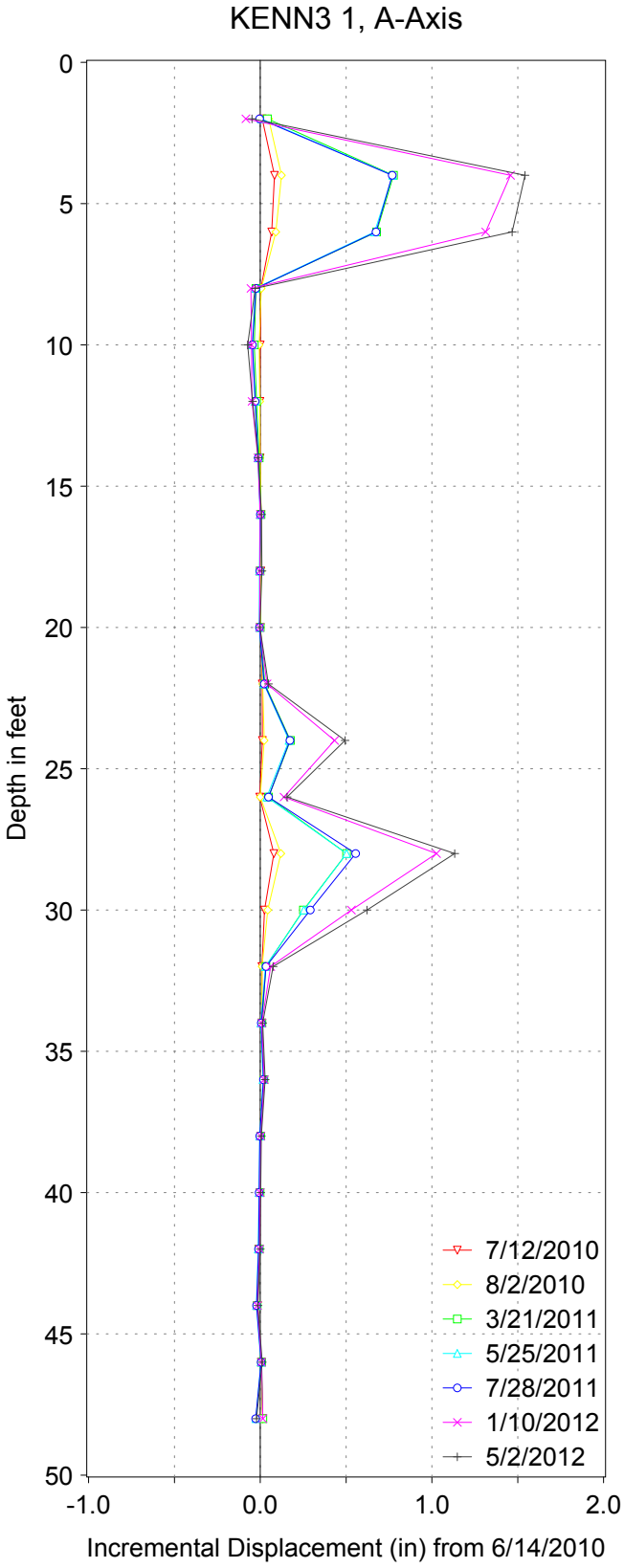
Cumulative Displacement

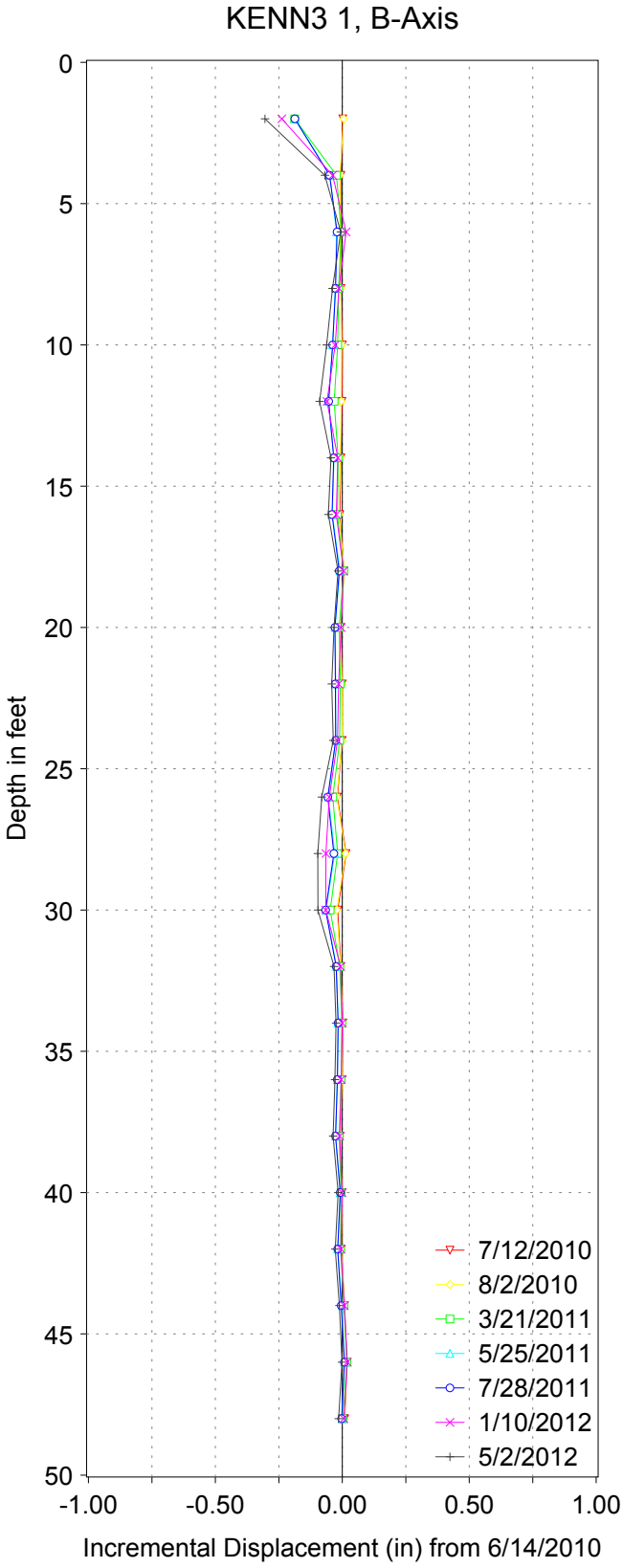
Tube #1



Time Displacement

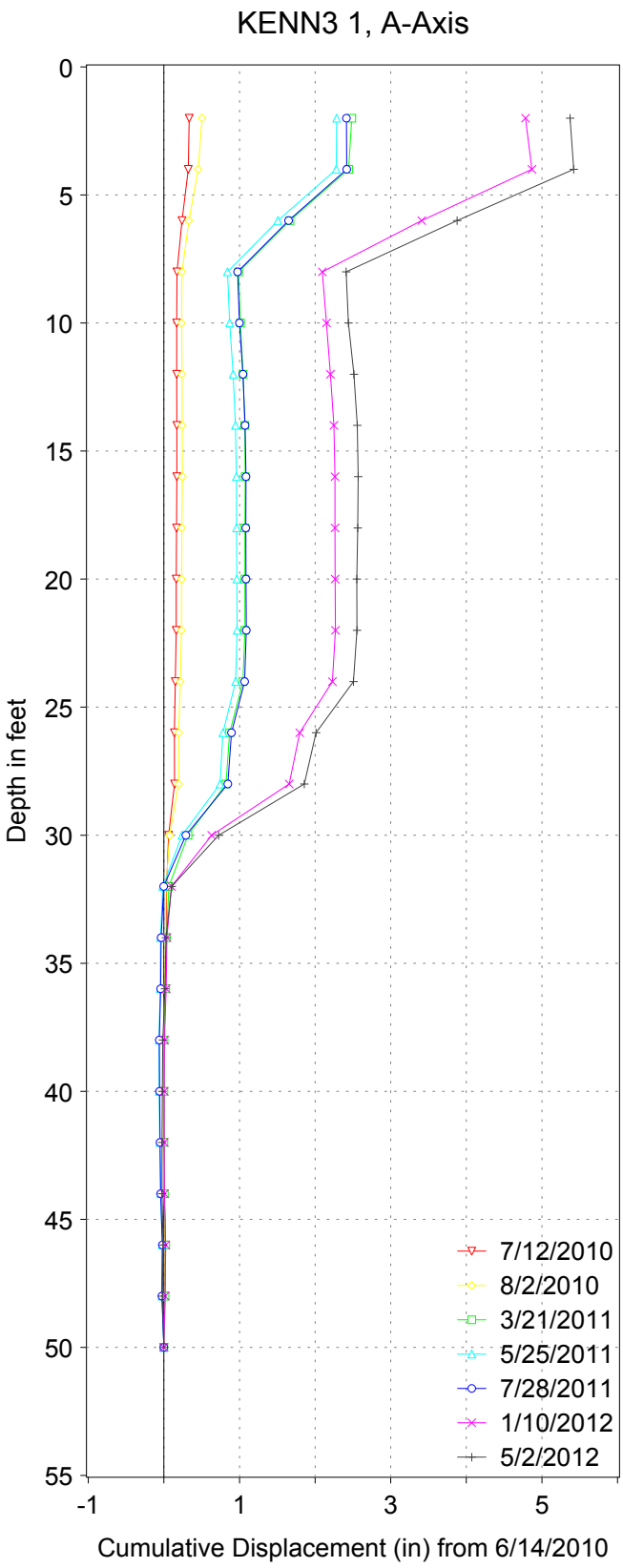
Tube #1

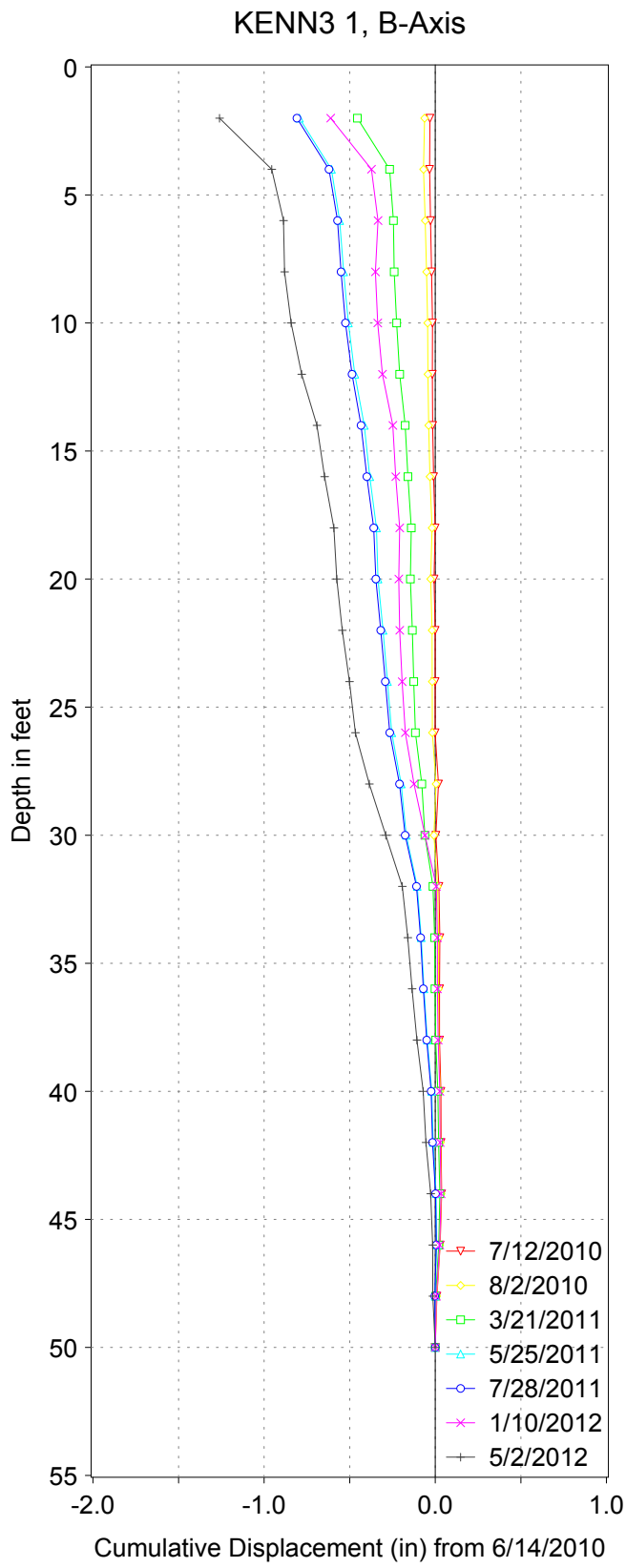




Incremental Displacement

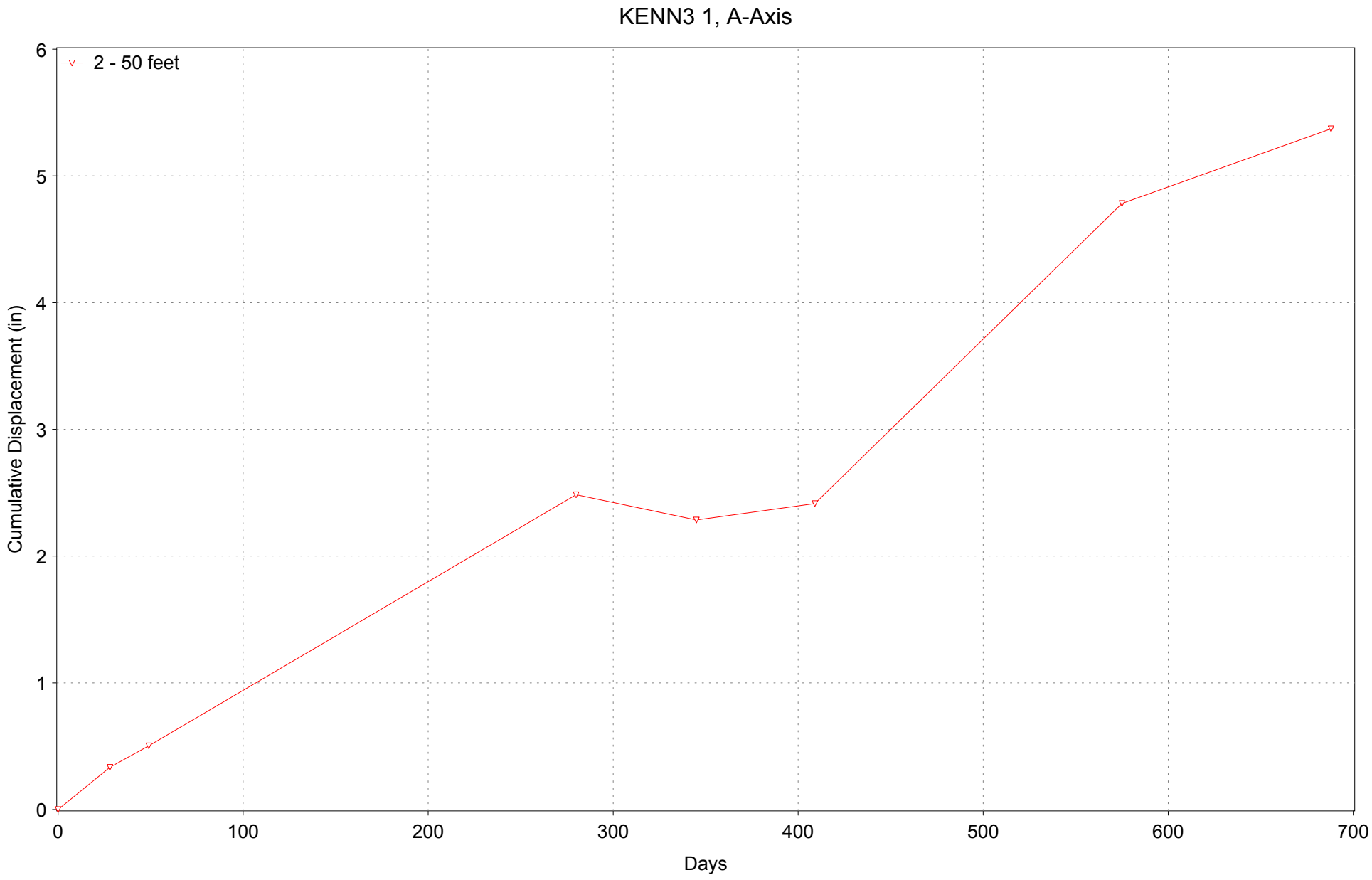
Tube #3





Cumulative Displacement

Tube #3



Time Displacement

Tube #3

Appendix G:
Memorandum:
Proposed Cross Section & Required Design Exceptions

SRF Consulting Group, Inc. | Engineers Planners Designers

Memorandum

SRF No. 8727

To: Angel Staples, P.E.
Minnesota Department of Transportation

From: Jamison Beisswenger, P.E. – SRF Consulting Group Inc.

Date: June 1, 2015 (Revised June 25, 2015)

Subject: Proposed Cross Section & Required Design Exceptions
Bridge No. 9090; US2 over the Red River (John F. Kennedy Memorial Bridge)

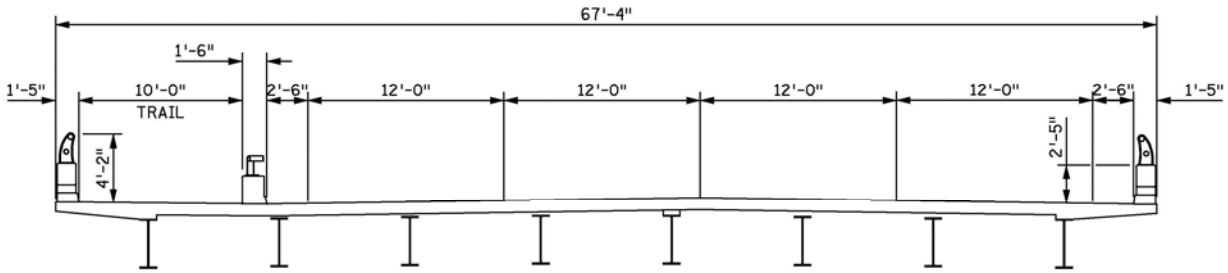
Introduction

Bridge No. 9090, U.S. TH2 over the Red River, also known as the John F. Kennedy Memorial Bridge, was built in 1963. The Bridge is eligible for the National Register of Historic Places under Criterion C (design and construction) in the area of Engineering, and under Criterion A (broad patterns of history) in the area of Transportation.

The primary purpose of this project is to provide a structurally sound crossing of U.S. TH2 over the Red River. The primary needs include rehabilitation of deteriorated or structurally inadequate structural elements. Secondary needs include providing a reliable crossing for current and future traffic demands and to Improve bicycle/pedestrian access and connectivity. In order to achieve both primary and secondary needs, the existing bridge deck has been identified for replacement.

The purpose of this memorandum is to identify the desired geometric elements of the proposed deck replacement. The overall width of the proposed deck replacement is constrained by the existing truss members and is limited to 67 foot, 4 inches. This memorandum compares the lane, shoulder, trail and barrier dimensions proposed by MnDOT District 2, the City of Grand Forks and MnDOT's Cultural Resource Unit (CRU) with current design requirements or guidelines. A design exception may be required for elements which do not meet design requirements or guidelines. The proposed cross section for Bridge No. 9090 is shown in Figure 1.

Figure 1. Bridge 9090 Proposed Cross Section



Trail Width

Proposed Trail Width

Table 1 below indicates the trail width requirements in accordance with MnDOT's *LRFD Bridge Design Manual*. In order to meet the desires of the community, a barrier separated trail with a width of 10 feet has been proposed.

Table 1. Trail Width Requirements when Bicycle Traffic is Expected (MnDOT LRFD Bridge Design Manual – Section 2.1.2)

Minimum	Desirable	Maximum
8'-0"	10'-0"	12'-0"

Design Issue

In order to accommodate the trail within the confines of the existing truss, a roadway width of approximately 53 feet is proposed (Figure 1). However, MnDOT's Fiscal Year 2016-2020 *Bridge Preservation, Improvement and Replacement Guidelines* requires that the roadway width meet the requirements of FHWA's *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* with a minimum deck geometry rating of 5.

Table 2. Deck Geometry Rating (FHWA's Recording and Coding Guide – Table 2C)

Deck Geometry Rating	Roadway Width (Curb-to-Curb)
6	60.7'
5	54.5'
4	50.6'
3	49.6'

Lane Width

Proposed Lane Width

Both the City of Grand Forks and MnDOT's District 2 have expressed the desire for 12 foot lanes to match the lane width of the approach roadways. This exceeds the minimum value of 11'-0" as given in Table 1 of MnDOT's Fiscal Year 2016-2020 *Bridge Preservation, Improvement and Replacement Guidelines*.

Design Issue

In order to accommodate four, 12 foot lanes and 10 foot trail, shoulder widths of approximately 2 foot, 6 inches have been proposed. However, Table 9-2.03A of MnDOT's *Road Design Manual* requires minimum bridge shoulder widths of 4 feet.

Traffic Barrier Height

Proposed Barrier Height

In order to provide a railing that meets the TL-3 crash testing requirement of bridges on the National Highway System and closely replicates the aesthetics of the existing railing, a 29 inch tall, vertical faced Nebraska railing has been proposed.

Design Issue

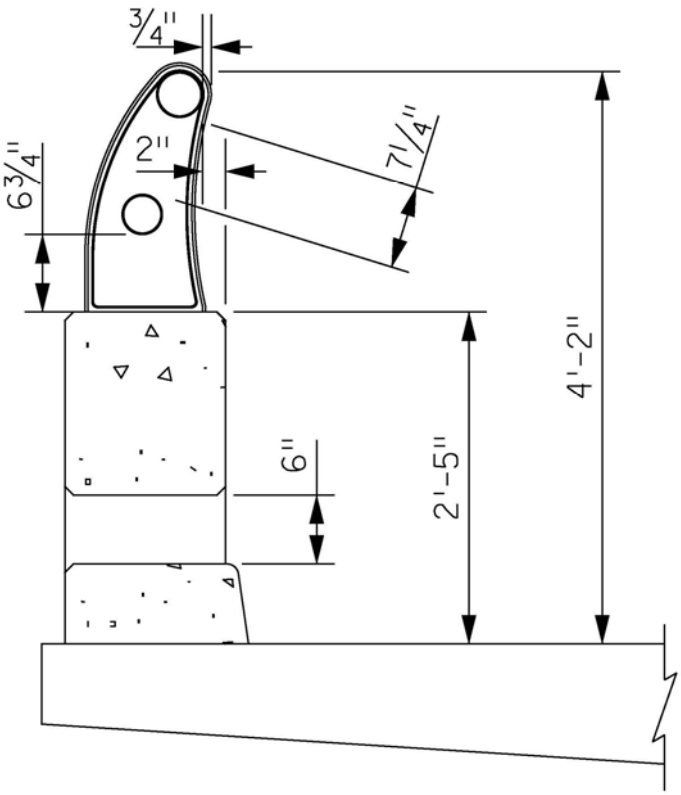
Reuse of the existing aluminum railing is desired atop the traffic barrier. Section 13.2.1(2) of MnDOT's LRFD Bridge Design Manual requires the use of a 32 inch high vertical faced concrete barrier to reduce the tendency for the vehicle to climb or roll over the barrier face and keep the vehicle back from the metal rail.

Pedestrian/Bicycle Barrier Dimensions

Proposed Barrier Dimensions

The proposed pedestrian barrier on the north side of the truss consists of a 2 foot, 5 inch vertical faced barrier and the salvaged, existing aluminum railing. The overall height of the railing is 50 inches. A 6 inch opening in the concrete portion of the railing has been proposed to match the opening in the existing concrete railing. The maximum opening between the aluminum rail elements is 7.25 inches.

Figure 2. Bridge 9090 Proposed Pedestrian Railing Dimensions



Design Issue

Required pedestrian/bicycle railing dimensions vary between State specifications (MnDOT) and Federal specifications (AASHTO's LRFD Bridge Design Specifications). Table 3 lists the requirements of both the State and Federal specifications.

Table 3. Required Pedestrian/Bicycle Railing Requirements

Element	AASHTO LRFD Requirement	MnDOT Requirement	Proposed
Railing Height	42"	54"	50"
Openings (Lower 27")	6"	4"	6"
Openings (Above 27")	8"	6"	7.25"
Rail Projection	Rails should project beyond the face of posts	None	Rails 3/4" back of posts

Summary

Table 4 summarizes the proposed cross section element dimensions which affect vehicular traffic and compares them to current design requirements. The table indicates if a design exception is required and how much each dimension deviates from current design requirements or guidelines.

Table 4. Vehicular Cross Section Design Elements

Design Element	Proposed Dimension	Required Dimension	Exception Required?	Notes
Trail Width	10'-0"	10'-0" (desired) 8'-0" (min.)	No	Meets desired dimension. Exceeds minimum by 2'-0".
Lane Width	12'-0"	11'-0"	No	Exceeds minimum by 1'-0".
Shoulder Width	2'-6"	4'-0"	Yes	1'-6" less than minimum required
Roadway Width	53'-0"	54'-6"	Yes	1'-6" less than required for NBI Deck Geometry Rating of 5
Traffic Barrier Height	29"	32"	Yes	3" less than minimum required.

Table 5 summarizes the proposed pedestrian rail design elements which will require a design exception. Design exceptions for the proposed rail height and openings anticipated to be routine as noted in MnDOT's memorandum entitled *Design Exceptions and Variances on Historic Bridges: Effective Application and Utilization Guidelines* since the requirements of AASHTO are met. The requirement that the rails project beyond the face of the posts could be mitigated by the fact that the posts are set back from the face of the concrete barrier.

Table 5. Pedestrian Rail Design Elements

Design Element	Proposed	Required	Notes
Ped/Bike Rail Height	50"	42" (AASHTO) 54" (MnDOT)	Meets Federal minimum. 4" less than state minimum
Ped/Bike Rail Openings (Lower 27")	6"	4" (MnDOT) 6" (AASHTO)	Meets Federal maximum. 2" greater than State maximum
Ped/Bike Rail Openings (Above 27")	7.25"	6" (MnDOT) 8" (AASHTO)	Meets Federal maximum 1.25" greater than State maximum
Rail Projection	Rails set back ¾"	Rails project past the posts	May be mitigated by setting posts back 1¼" from face of concrete

Appendix H:
Memorandum:
Proposed Kennedy Bridge Railings & Cross Section Considerations

Minnesota Department of Transportation

Office of Environmental Stewardship
Mail Stop 620
395 John Ireland Boulevard
St. Paul, MN 55155-1899

Office Tel: (651) 366-3604
Fax: (651) 366-3603

MEMO

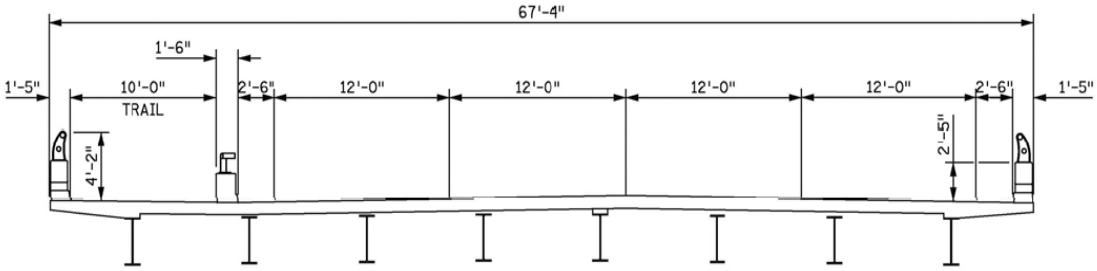
June 1, 2015

To: Joe McKinnon, MnDOT District 2
Angel Staples, MnDOT Bridge Office
Matt Cramer, SRF

From: Liz Abel, MnDOT CRU

Re: Proposed Kennedy Bridge Railings and Cross Section Considerations

During the past few weeks we have been reviewing potential configurations for the Kennedy Bridge cross section. As discussed at our May 21 weekly conference call, the option currently being discussed (see drawing below) would likely require several design exceptions including for roadway width, shoulder width, and traffic barrier height.



The design exception related to traffic barrier height arises from the proposed use of the Nebraska Open barrier as part of the assembly replacing the historic railing, which would need to be replaced along with the deck. This memo has two purposes: to explain CRU's preference for the Nebraska Open railing option, and to suggest an adjustment to the proposed cross section that might reduce the number of design exceptions, thereby making it more likely that the traffic barrier height exception would be deemed acceptable.

Existing Bridge Railing and Replacement Options

The existing railing is a significant component of the bridge's historic fabric and contributes to its historic integrity and National Register eligibility. It is composed of an aluminum post and tube system bolted to the top of a concrete parapet. Its design is not unique to the Kennedy Bridge, but was used on bridges of similar age in Minnesota and elsewhere. The railing is a late example of Streamline Moderne design. Its shapes, materials, and surface treatment contribute to the bridge's Modernist aesthetic. From many locations it is one of the most visually dominant design elements.

The railing is one of the elements that would be most changed by the proposed rehabilitation, and it is important to find a replacement rail that diminishes the integrity of the bridge design as little as possible.

The existing railing is 39 inches high. The aluminum portion is 21 inches tall and composed of curved posts and a pair of extruded aluminum tubes. The concrete portion is 18 inches tall and composed of a 12-by-12 inch beam supported by posts that are 2 feet long. The posts are spaced about 8 feet apart, creating a visually bold repeating pattern of approximately 6 foot openings alternating with 2-foot posts.

The current rail does not meet modern standards. MnDOT standards require a TL-3 rail in this situation (35 mph speed limit, on the National Highway System). It is proposed that the aluminum portion of the historic rail be salvaged (with some damaged parts replaced in-kind) and placed on a new crash-tested concrete barrier.

A search for possible TL-3 and TL-4 barriers identified the Nebraska Open TL-4 barrier as being the best option for providing crashworthiness in a way that is least disruptive to the bridge's historic integrity.

While similar to the existing parapet, the Nebraska Open would be taller and bulkier. The new concrete barrier would be 60% higher than the existing concrete parapet. The new beam would be 16 inches tall, rather than the existing 12 inches. Among the characteristics this would affect is the visual relationship between the size of the beam and the edge of the deck when the bridge is viewed from the outside.

The Nebraska Open barrier has an advantage relative to other barriers that would help mitigate the changes: it is the only barrier identified that has longitudinal post-to-opening spacing very similar to the existing. This would preserve an important aspect of the design.

The Kansas Corral TL-4 barrier, another option considered, would push the rail design even farther from existing. If the Kansas Corral were used, the concrete portion of the rail would be 77% higher than existing. The new beam would be considerably taller: 19 inches compared to the existing 12. In the Kansas Corral the posts would be longer (3 feet) and the openings shorter (5 feet), letting less light pass through the rail and changing the key visual pattern of posts to openings.

Possible Design Exceptions

Use of the Nebraska Open barrier would provide a railing stronger than required: TL-4 rather than TL-3.

Use of the barrier and the salvaged aluminum rail would provide a pedestrian/bike rail height of 50 inches. This would meet the AASHTO requirement (42 inches) but not the MnDOT

requirement (54 inches). Use of the salvaged aluminum rail would result in a rail opening size of 7.25 inches at locations above 27 inches. This would meet the AASHTO requirement (8-inch opening) but not the MnDOT requirement (6-inch opening). As per the 2008 Programmatic Agreement on historic bridges, these types of design exceptions have been approved in the past where there are differing standards, no extenuating circumstances, and use of the lesser standard would help preserve the character of the historic bridge.

Use of the Nebraska Open barrier would require a design exception for traffic barrier height. According to MnDOT standards, if a concrete barrier has an ornamental railing on top, the concrete barrier should be 32 inches high. The concrete barrier in the above scenario would be 29 inches high.

The likelihood of this design exception being approved is not yet known because of the need to consider all possible exceptions cumulatively. To reduce the number of design exceptions needed, thereby making it more likely that the traffic barrier height exception would be approved, I am asking that the following adjustment to the proposed cross section be considered:

Create more roadway width by reducing the width of the trail from 10 feet to 8 feet 6 inches. The trail would still meet the 8-foot minimum required by the Bridge Design Manual. While 10 feet is desirable per the Manual, the level of expected pedestrian and bike traffic may make this an acceptable option. MnDOT's 2007 Bikeway Facility Design Manual suggests signage might be useful in some situations to mitigate trail design constraints. For example, a sign could remind users to be watchful of other traffic:

The reduction in trail width would provide 18 inches to distribute in the roadway. Roadway width would increase from 53 feet to 54 feet 6 inches, which would meet the MnDOT standards for width. Shoulder width has been a consideration in the barrier height discussion. If lane width could be reduced from 12 feet to 11 feet 6 inches (the MnDOT standard requires 11 feet), shoulder width could be widened to 4 feet, which would also meet the standards.

cc: MnDOT Project File
Sue Granger, Gemini Research

Appendix I:
Memorandum:
Longitudinal Joints & Swivels

SRF Consulting Group, Inc. | Engineers Planners Designers

Memorandum

SRF No. 8727.00020.011

To: Angel Staples, P.E.
Minnesota Department of Transportation

From: Jamison Beisswenger, P.E.

Date: June 5, 2015

Subject: Longitudinal Joints & Swivels – Bridge No. 9090; US2 over the Red River

Introduction

Bridge No. 9090, also known as the Kennedy Bridge, carries U.S. Highway 2 over the Red River between East Grand Forks, Minnesota and Grand Forks, North Dakota. In order to accommodate foundation movement of the approach span pier bents, the bridge was built with a longitudinal joint in the median (Figure 1) and horizontal swivels (Figure 2) at each pier bent. This purpose of this memorandum is to evaluate the effects of removing the longitudinal deck joint in the median between the eastbound and westbound lanes.

Figure 1. Longitudinal Deck Joint

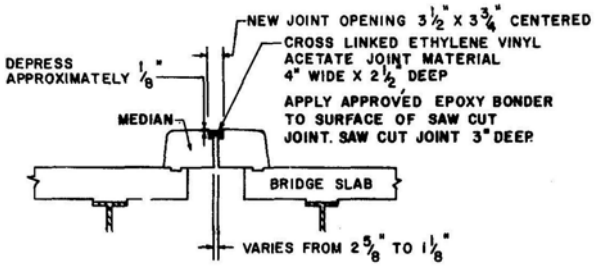
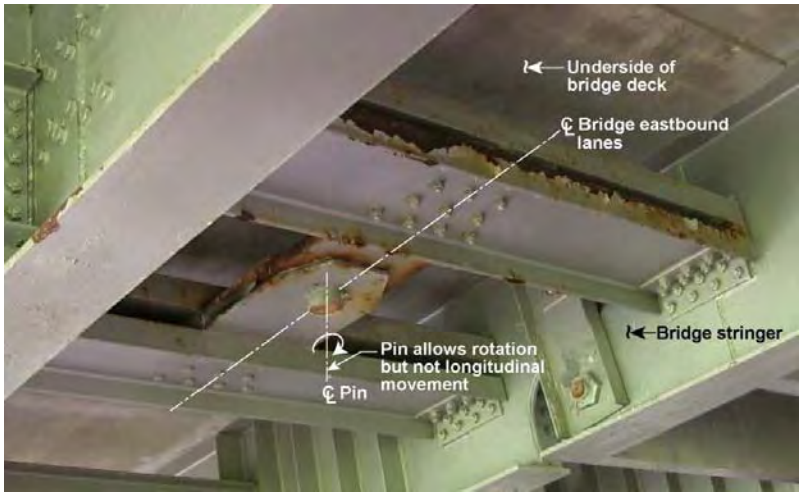


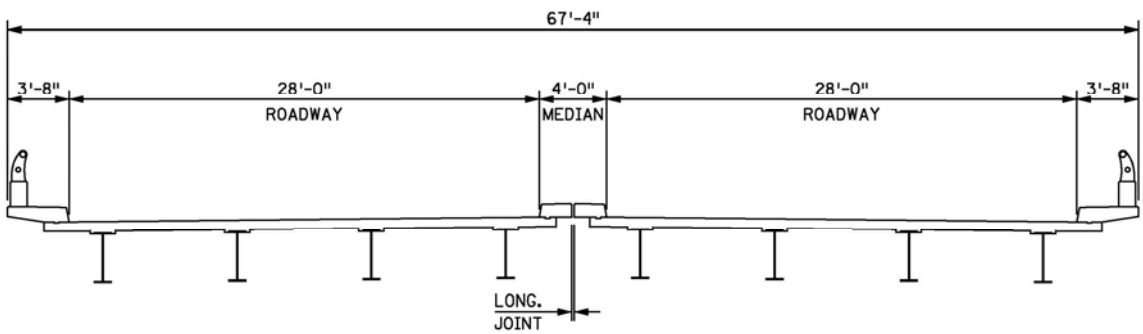
Figure 2. Typical Approach Span Swivel
(Source: Kennedy Bridge Planning Study Technical Memorandum dated 4/28/2014 prepared by CH2MHill)



Description of Problem

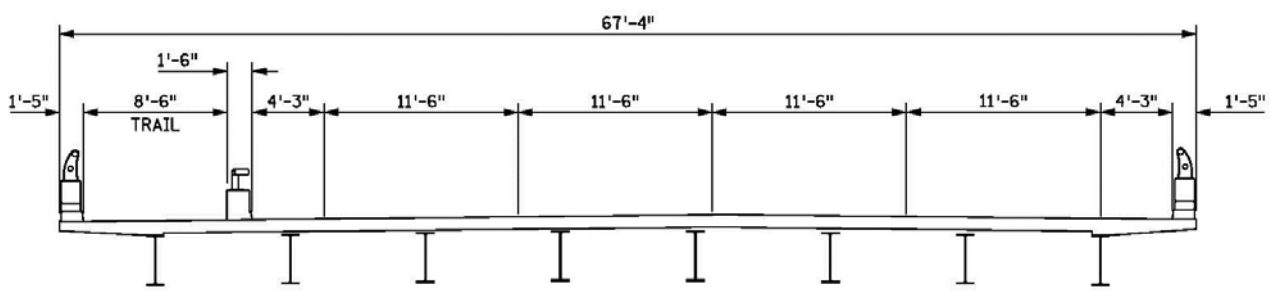
The approach spans were constructed by dividing the superstructure into two segments. Each segment consists of a 28 foot roadway, a 3 foot-8 inch curb/barrier and a 2 foot median supported by 4 steel beams (Figure 3). A longitudinal joint in the median allows for independent movement of each segment.

Figure 3. Existing Typical Section



In order to accommodate a new trail, four traffic lanes, and two shoulders, the configuration of a replacement bridge deck would require elimination of the median and longitudinal deck joint (Figure 4). A longitudinal deck joint would be easily damaged and difficult to maintain if placed within the roadway surface. Additionally, a joint placed within the roadway surface could prove to be a safety hazard by catching or snagging vehicle wheels.

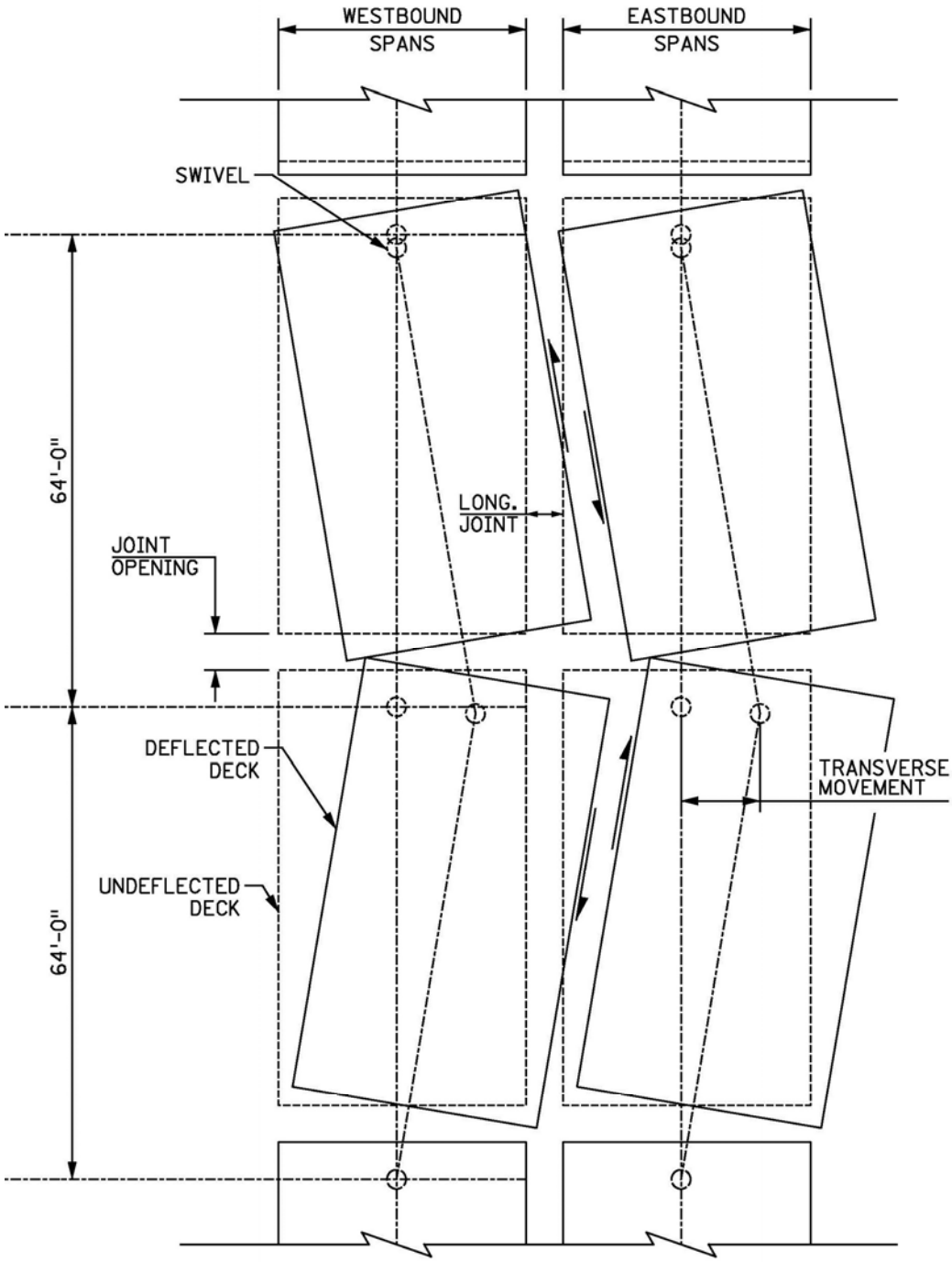
Figure 4. Proposed Typical Section



Purpose of Joint and Swivels

The primary purpose of the longitudinal joint and swivels is to accommodate differential transverse movements of the pier bents as soil moves perpendicular to the bridge centerline. Figure 5 shows that, as one pier moves relative to the adjacent piers, the swivels allow each deck segment to rotate horizontally. To accommodate the swivel rotation in the deck, the longitudinal joint allows each half of the bridge to translate longitudinally to one another.

Figure 5. Transverse Bridge Movement



Removing the longitudinal joint would prevent the independent movement of each bridge segment and would not allow the swivels to rotate. The swivels assemblies would be placed in tension and compression as one pier bent moved transversely relative to the adjacent pier bents.

Observed Pier Movements

Except at Pier 6, no records regarding pier lateral movements exist. However, measurements of the existing transverse joint openings and measurements of existing pin and hanger/hinge openings can give an indication if excessive movements have occurred in the past. Large, relative differences in opening measurements at the outer and inner gutter lines or at the outer and inner beams would give an indication of the magnitude of transverse pier movement.

Joint Opening Measurements

Opening measurements of the existing joints were taken during SRF's field investigation on May 19th and 20th, 2015. In 1984, all joints (excluding joints at two hinge locations) were replaced with strip seal style expansion joints. Plans indicate that the joint openings were to be set at 2 inches at all temperatures. Plans indicate that the existing finger joints at hinge locations were reinstalled with an opening of 10.5 inches at 60 degrees Fahrenheit. Measurements of the current joint openings are shown in Table 1. While some differences in joint openings would be expected due to construction tolerances, excessive joint opening differences (greater than 0.375 inches) were highlighted below.

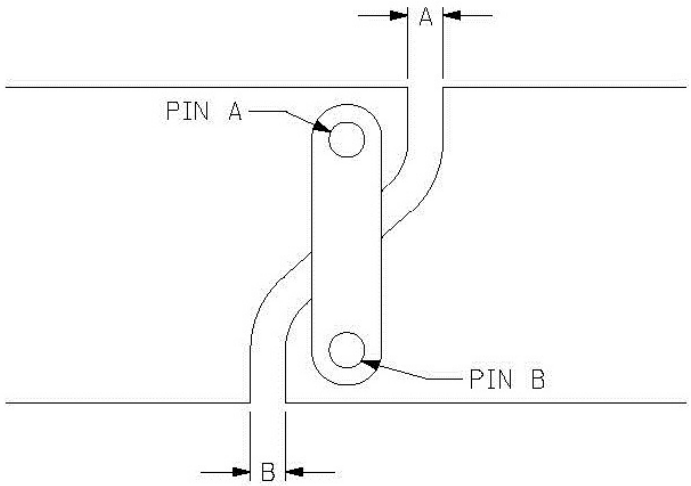
Table 1. Strip Seal and Finger Joint Openings

	Westbound Bridge Joint Openings (in.)			Eastbound Bridge Joint Openings (in.)		
Joint Adjacent to Bent No.	@ Median (in.)	@ N. Gutter (in.)	Difference (in.)	@ S. Gutter (in.)	@ Median (in.)	Difference (in.)
1 (Abutment)	NA	NA	NA	NA	NA	NA
2	2	1.875	0.125	2	2	0
3	2	2	0	2.25	2	0.25
4	2	1.75	0.25	1.75	2	-0.25
5 (Finger Jnt.)	7	8.375	-1.375	8	8.75	-0.75
6	1.25	2.375	-1.125	2.125	0.875	1.25
7	1.75	1.625	0.125	1.25	1.625	-0.375
8	2	1.625	0.375	1.625	1.75	-0.125
9 (Finger Jnt.)	7.75	10	-2.25	8	8.125	-0.125
10	2	2	0	1.625	1.75	-0.125
11	2	1.75	0.25	1.875	1.75	0.125
12	2	2	0	1.625	1.75	-0.125
13	2	1.875	0.125	1.75	2	-0.25
14 (Abutment)	NA	NA	NA	NA	NA	NA

Pin and Hanger/Hinge Joint Opening Measurements

Measurements of the existing pin and hanger joint openings were taken during MnDOT's 2013 Fracture Critical Bridge Inspection. A measurement was taken at the top opening ('A') and the bottom opening ('B') at all pin and hanger locations (Figure 6.)

Figure 6. Pin and Hanger Openings



Similar measurements were taken at hinge locations during SRF's field investigation on May 19th and 20th, 2015. Measurements values for the pin and hanger openings can be found in MnDOT's 2013 Fracture Critical Inspection Report; measurement values for the hinge joints are available from SRF upon request.

Evaluation of Joint Opening Measurements

Theoretical transverse pier movements can be calculated by comparing measurements of the joint, pin and hanger, and hinge openings at the extreme sides of each bridge segment. This evaluation is not intended to be an accurate representation of past substructure movements as construction/measurement tolerances could be responsible for differences. This exercise is only intended to give a relative indication of any excessive movements that may have occurred. Calculated relative pier movements (movements of one pier relative to the adjacent pier) based on the different joint opening measurements for the westbound and eastbound spans are shown in Table 2.

Table 2. Theoretical Relative Transverse Movement

	Calculated Westbound Pier Relative Movements (in.)			Calculated Eastbound Pier Relative Movements (in.)		
Bent No.	Based On Joint Measurement	Based On Hinge Measurement 'A'	Based on Hinge Measurement 'B'	Based On Joint Measurement	Based On Hinge Measurement 'A'	Based on Hinge Measurement 'B'
1 (Abutment)	0.0	0.0	0.0	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0	0.0	0.0
3	0.3	0.0	0.7	0.0	0.7	0.0
4	0.3	-0.7	-0.3	0.6	2.3	1.0
5	0.9	-0.3	0.3	0.0	2.3	1.0
6	-2.3	-6.3	-5.7	-1.7	1.3	0.0
8	-4.3	-6.3	-4.0	-1.1	-6.0	-3.3
9	0.9	-0.3	2.0	-0.9	-5.0	-2.3
10	0.9	0.3	2.7	-0.6	-2.3	-5.0
11	0.3	0.3	0.0	-0.9	-1.0	-2.0
12	0.3	0.0	0.3	-0.6	-1.3	-2.3
13	0.0	0.0	0.0	0.0	0.0	0.0
14 (Abutment)	0.0	0.0	0.0	0.0	0.0	0.0

(+ indicates movement north; - Indicates movement south)

Calculated transverse movements of the typical pier bents that are greater than 1 inch are highlighted in green in Table 2. While movements greater than 1 inch are noted at several locations, these are based on hinge opening measurements. In every case, the movements based on hinge opening measurements are not substantiated by the measurements taken at the strip seal joints. This could indicate that transverse movements occurred prior to the 1984 joint repairs and have slowed since, or that the bridge was originally constructed with these hinge openings.

Calculated transverse movements of the cast-in-place concrete piers (piers 6 and 8) that are greater than 1 inch are highlighted in red in Table 2. Measurements of both the deck joint openings and the hinge openings indicate that excessive transverse movements may have occurred at these piers since the bridge's construction in 1963.

Existing Movement Capacity with Longitudinal Joint

In order to evaluate the effects of removing the longitudinal joint, the tolerable differential movement of the existing pier bents was determined. The two mechanisms identified as governing transverse movements were the closing of the transverse joints and the flexural capacity of the piling

Joint Closing

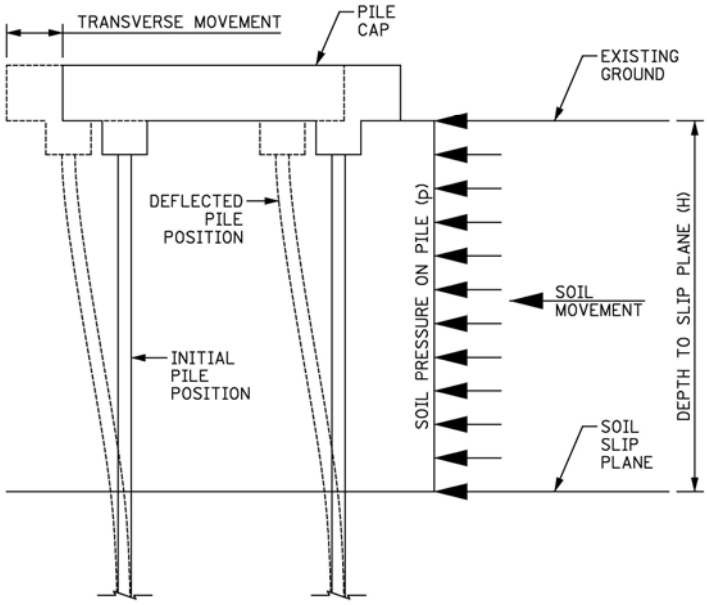
As the piers move relative to each other, the transverse joints open on one side of the bridge and close on the other. Differential transverse movement is limited by joints closing as shown in Figure 5.

The original transverse deck joints were finger-style joints which had an opening of 3½ inches. In 1984 most of the transverse deck joints were replaced with strip seal joints with an opening of 2 inches. Assuming a transverse joint opening of 2 inches, the differential pier movement that could occur before a joint would close was calculated as 4 inches.

Pile Flexural Capacity

In addition to closure of the transverse joints, the tolerable movement of the pier bents is limited by the combined axial compression and flexure in the pile. Figure 7 shows how the piles supporting the pier bents deflect under soil movement. The pressure on the piling due to soil movements is assumed to be constant for the cohesive clay soils.

Figure 7. Transverse Soil Movement – Pier Bent Allowed to Translate



An exercise was conducted to determine the magnitude of soil pressure that would reduce the structural capacity of the pile. The maximum factored pile bearing (ϕR_n) resistance was assumed to be 190 Tons for an HP14x73 pile per MnDOT practice. The yield stress of the pile was assumed to be 33 ksi per Table 6A.6.2.1-1 of AASHTO's *Manual for Bridge Evaluation*. Using these assumptions, the maximum factored moment that could be applied to the pile without reducing the axial capacity was determined to be 73 ft-kips.

The commercial program AllPile7 was used to calculate the soil pressure that would cause a 73 ft-kip moment in the piling. The soil parameters used in AllPile7 for this evaluation are given in Table 1. Table 2 gives the maximum tolerable soil pressures for several assumed slip plane depths that were calculated for this exercise.

Table 3. Soil Parameters for AllPile7 Analysis

Soil Parameter	Value
Soil Type	Soft Clay
Dry Unit Weight	120 lbs./ft^3
Buoyant Unit Weight	57.6 lbs./ft^3
Friction Angle	0°
Cohesion	300 lbs./ft^2
Horizontal Modulus of Subgrade Reaction	30 lbs./in^3
Strain @ 50% Deflection (e50)	2%
N_{spt} (blows per ft. for a 140 lb. hammer)	5

Table 4. Maximum Tolerable Soil Pressure on Pile – 73 ft-kip Max. Pile Moment

Depth to Slip Plane (ft)	Soil Pressure on Pile (lbs/ft)	Transverse Movement (in)
10	2050	1.6
20	1810	2.6
30	1920	3.6
40	2080	4.5*
50	2230	5.2*

*Transverse movement exceeds tolerable movement due to joint closure

Movement Resistance without Longitudinal Joint

Removal of the longitudinal deck joint effectively prevents any relative transverse movements of the pier bents. To determine the bridge's resistance to future soil movements without a longitudinal joint, the structural capacities of the piling and the superstructure were evaluated assuming the pile cap was prevented from moving transversely.

Pile Structural Resistance

The piles were evaluated to determine the moments that would result from preventing translation of the pier bents using the tolerable soil pressures determined in Table 4. The reaction at the top of the pile cap caused by the superstructure preventing pier bent movement was modeled in AllPile7 by applying a point load at the top of the pile that would cause no tip deflection as illustrated in Figure 8. Resulting pile moments and axial capacities are given in Table 5.

Figure 8. Transverse Soil Movement – Pier Bent Translation Prevented

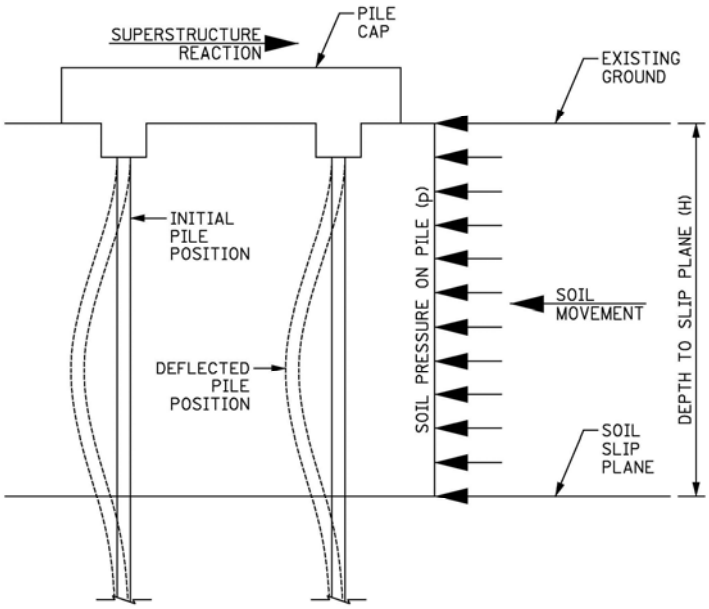


Table 5. Pile Moments and Superstructure Reactions – Pier Bent Translation Prevented

Depth to Slip Plane (ft)	Soil Pressure on Pile (lbs/ft)	Max. Pile Moment (kip-ft)	Superstructure Reaction (kip/pile)	Revised Pile Axial Capacity (kip/pile)
10	2050	58.6	13.0	380*
20	1810	57.3	14.0	380*
30	1920	79.2	15.0	355
40	2080	95.8	16.0	285
50	2230	109.2	17.0	240

*Based on MnDOT practice for geotechnical capacity

For assumed slip planes at 10 and 20 feet, the axial pile capacity is not reduced when pier bent movements are restrained. For assumed slip planes at 30 and 40 feet, the pile structural capacity is less than the assumed geotechnical capacity. However, these reduced capacities are still greater than the actual factored axial pile load (estimated to be less than 250 kips). For the assumed slip plane at 50 feet, the pile capacity is reduced slightly below the estimated factored axial pile load. However, the pressure used in this calculation results in a pile deflection in the existing condition greater than what has been observed at the typical pier bents.

Superstructure/Pier Bent Structural Resistance

As shown in Figure 8, preventing pier bent movement will cause a corresponding reaction at the base of the pier bent. In order to evaluate the effects of this force, a finite element model of the west approach spans was created in Staad.Pro (Figures 9 and 10). The west approach spans were modeled as it was determined that any force effects due to pier bent movements would be greater at the west approach due to the shorter and stiffer piers compared to the east approach.

Figure 9. Staad.Pro Finite Element Model – West Approach Spans
(Deck not shown for clarity)

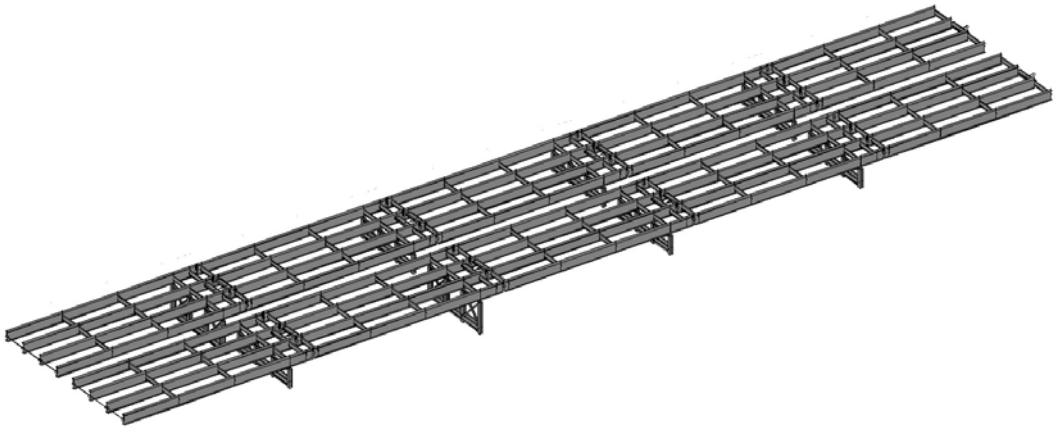


Figure 10. Staad.Pro Finite Element Model – Pier Bent & Swivel
(Deck not shown for clarity)

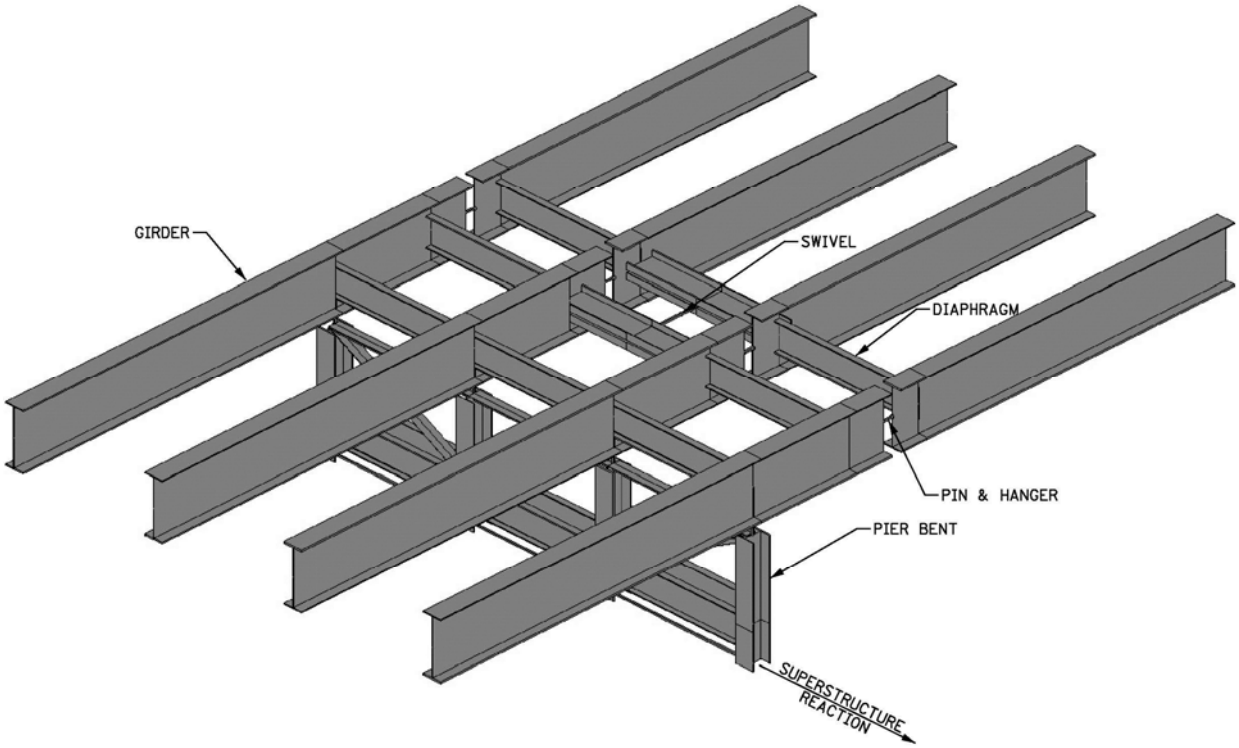


Table 6 indicates the members for which pier movements could impact structural capacity. The capacity of each member was calculated and compared to the results of the finite element model to determine the maximum tolerable magnitude of superstructure reaction. The tolerable superstructure reactions were found to be greater than those that would occur due to the pressures shown in Tables 4 and 5.

Table 6. Tolerable Superstructure Reactions

Element	Capacity 'C' (kip)	Member Force Due to Unit Pile Load 'P' (kip)	Tolerable Superstructure Reaction 'C'/'P' (kip/pile)
Swivel (Channel Flexure)	240	4.42	54.2
Pier Bent - Diagonal Compression	144	2.33	61.8
Pier Bent - Column Uplift*	73	1.67	43.7

*Based on 0.9 x DL Compression in Column

Recommendations

Swivels at Pier Bents

Evaluation of the existing expansion joint measurements (Table 1) and the calculated relative pier movements (Table 2) provides evidence that the typical pier bents (bents 2-5 and bents 9-13) have undergone little, if any, relative transverse movements. Additionally, analysis indicates that the structural effects of any future transverse soil movements resulting from removal of the existing longitudinal joint are not significant. Therefore, it is recommended that no modifications to the existing swivels at the pier bents is required due to removal of the longitudinal deck joint.

Swivels at Truss Floor Beams

Evaluation of the existing expansion joint measurements (Table 1) and the calculated relative pier movements (Table 2) provides evidence that the cast-in-place concrete piers (piers 6 and 8) may have undergone significant transverse movements in the past. If future movements occur at these piers and are not accommodated, significant stresses could be induced in the pier bents adjacent to the hinges/finger joints (bents 5 and 9). Evaluation of the finite element model shown in Figures 9 and 10 indicates that a transverse movement at pier 6 of less than 2" would be enough to cause uplift of the outer bearing at pier bent 5. Movements of even smaller magnitudes could impact the function of the finger joints or lock up hinge expansion bearings. Therefore, it is recommended that the swivel connection at the truss floor beams be modified to accommodate future movements.

Figure 11. Swivel at Truss Floor Beam

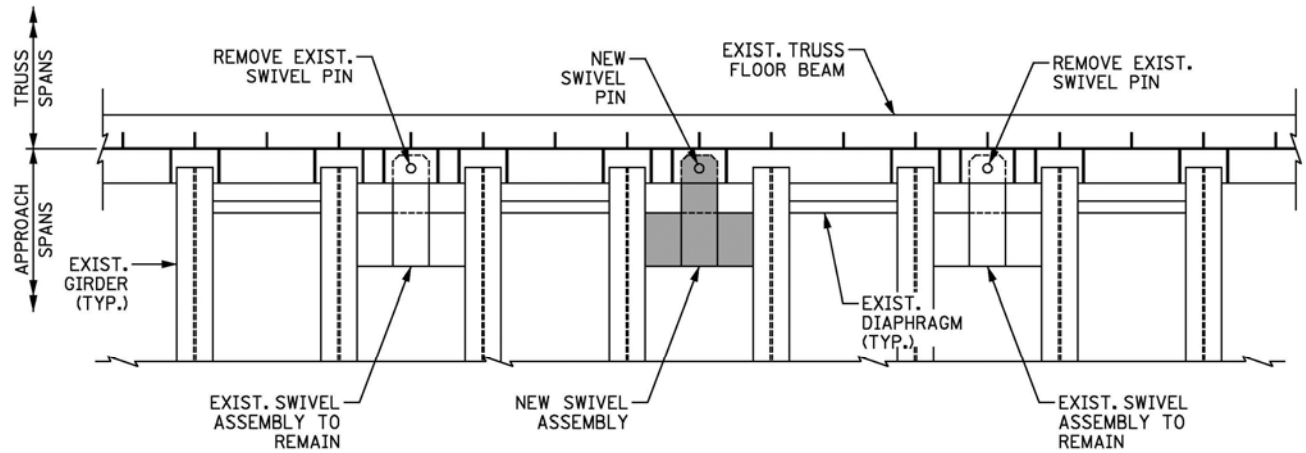
Swivel Modifications at Truss Floor Beams

Figure 12 shows the proposed modification to the swivel connections that would accommodate transverse movements of piers 6 and 8. A new swivel would be added at the center of the bridge while the pins at the existing swivels would be removed. This would allow rotation about the center of the bridge.

With the 2 inch strip seal expansion joint and the longitudinal deck joint, a transverse movement of approximately 4 inches could occur before the joint would close at one end. With the removal of the longitudinal deck a transverse movement of approximately 1 inch per inch of expansion joint width could occur before the joint would close at one end (i.e. a 2 inch joint would allow 2 inches of pier movement).

Section 14.2.2 of MnDOT's LRFD Bridge Design Manual allows for joint openings up to 4 inches for non-skewed bridges. Therefore, in order to maintain a movement capacity close to current conditions, a 3 inch strip seal joint with a movement capacity of 6 inches is recommended at the floor beams.

Figure 12. Proposed Swivel Modification
(Not to Scale)



Conclusion

If the modifications to the swivels at the truss floor beams shown in Figure 12 are implemented, removal of the longitudinal deck joint is feasible. No modifications would be required at the swivels adjacent to pier bents 2-5 and 9-13.

Appendix J:
Memorandum:
Modified Nebraska Open Concrete Bridge Rail

SRF Consulting Group, Inc. | Engineers Planners Designers

Memorandum

SRF No. 8727

To: Angel Staples, P.E.
Minnesota Department of Transportation

From: Jamison Beisswenger, P.E – SRF Consulting Group, Inc.

Date: August 13, 2015

Subject: Modified Nebraska Open Concrete Bridge Rail on MnDOT Bridge No. 9090; US2 over the Red River (John F. Kennedy Memorial Bridge)

Introduction

MnDOT Bridge No. 9090, U.S. Trunk Highway 2 over the Red River, also known as the John F. Kennedy Memorial Bridge (Kennedy Bridge), was built in 1963. The Bridge is eligible for the National Register of Historic Places in the area of Engineering and in the area of Transportation. MnDOT is undertaking a rehabilitation of the bridge in order to provide a structurally sound crossing of the Red River in Grand Forks, ND and East Grand Forks, MN while avoiding an adverse effect under Section 106 of the National Historic Preservation Act.

The existing bridge rail has been determined to be a character-defining feature of the Kennedy Bridge that contributes significantly to its physical character, historic integrity, and significance. The *Secretary of the Interior's Standards for Rehabilitation* states that "new features will match the old in design, color, texture, and, where possible, materials" and "The historic character of a property will be retained and preserved. The removal of distinctive materials or alteration of features, spaces, and spatial relationships that characterize a property will be avoided."

U.S. Trunk Highway 2 is part of the National Highway System (NHS) and has a design speed of 35 mph. This design speed warrants the use of a TL-2 barrier. However, on May 30, 1997, the Federal Highway Administration (FHWA) published an action letter regarding the crash testing of Bridge Railings. Attached to this letter was a document entitled *Bridge Railing Design and Testing*. This document states FHWA's position that, on all NHS projects, "The minimum acceptable bridge railing will be a TL-3 (MSL-2 until August 1998) unless supported by a rational selection procedure."

Throughout the development of this project, MnDOT has worked closely with a bridge historian and MnDOT's Cultural Resources Unit (CRU) to develop a railing that meets the functional requirements for bridges on the NHS and the *Secretary of the Interior's Standards for Rehabilitation*.

The purpose of this memorandum is to provide a comparison of the geometry, strength and potential crash worthiness of the crash tested TL-4 Nebraska Open Concrete Bridgerail to the bridge rail proposed on this rehabilitation project.

Existing Bridge Railing

The existing bridge railing consists of an aluminum post-and-beam railing mounted to an open, vertically faced concrete barrier. The concrete barrier is mounted on top of an 8 inch tall, 44 inch wide, raised concrete curb. The existing bridge cross section is shown in Figure 1 and the existing bridge railing is shown in Figure 2.

Figure 1. Existing Bridge 9090 Cross Section

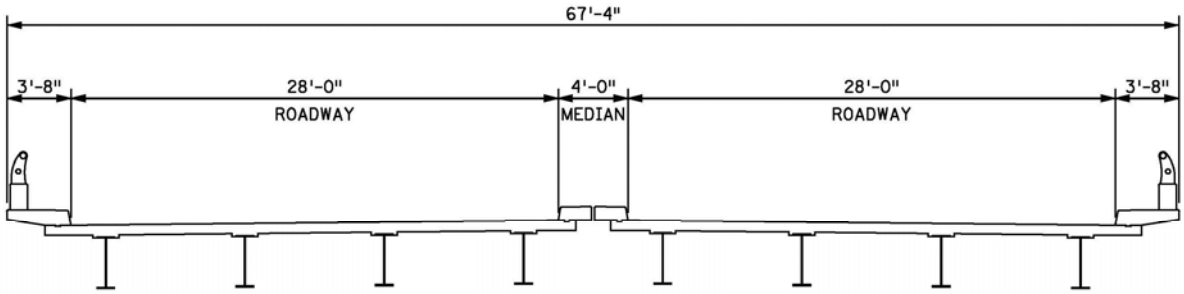
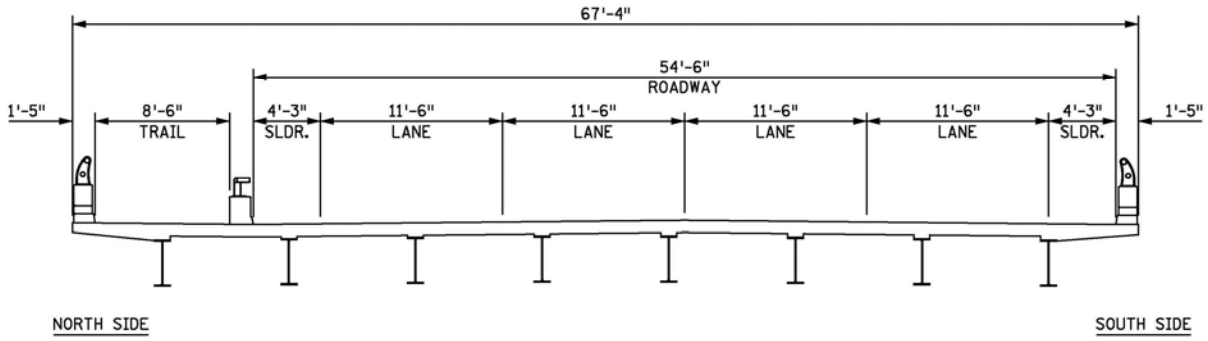


Figure 2. Existing Bridge 9090 Railing

Proposed Bridge Railing

The MnDOT Bridge office has worked closely with stakeholders to develop a new bridge cross section that includes both a four-lane roadway and a shared use trail as shown in Figure 3. The barrier separating the roadway is a Minnesota Combination Bridge Rail, Design #3 and has a crash test level of TL-4. No modifications are proposed for this rail. The barrier on the north side of the bridge is a combination pedestrian/bicycle railing that will have dimensions and a structural capacity that meets section 13 of the *AASHTO LRFD Bridge Design Specifications*. Crash testing requirements are not applicable to this railing.

Figure 3. Existing Bridge 9090 Railing

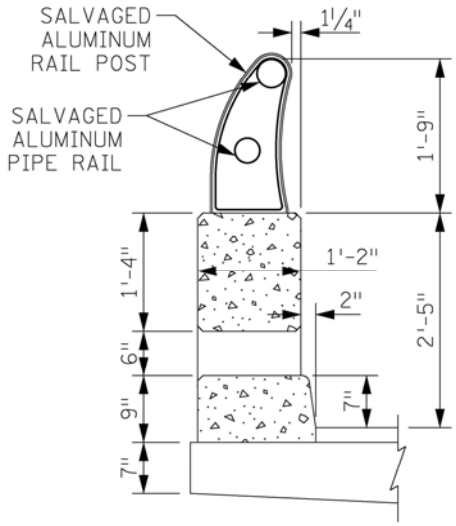


The south barrier is the subject of this memorandum as the geometry is proposed to be altered from a crash tested railing. MnDOT's CRU, in collaboration with a bridge historian, have determined that the risk of an adverse effect under Section 106 of the National Historic Preservation Act may be mitigated if:

- The existing aluminum post-and-beam railing is salvaged and reinstalled.
- The concrete portion of the railing is an open style type with curb, post, and opening dimensions that closely match those of the existing railing when viewed in elevation.
- The concrete post elements are flush with the faces of the beam element.
- The width of the concrete portion is minimized.
- The overall height of the new railing closely matches the height of the existing curb and railing when viewed from the bridge exterior.

The concrete barrier that has been crash tested and most closely met the requirements listed above was determined to be the TL-4 version Nebraska Open Concrete Bridgerail (modified from earlier TL-2 design) as listed in Attachment 4 of FHWA's Action Letter. The drawing for this railing (as contained within Appendix B7 of the Action Letter) is shown in Attachment 1.

Figure 4. Proposed Railing Cross Section



An elevation of the proposed barrier is shown in Attachment 2 and a cross section is shown in Figure 4. The crash tested TL-4 Nebraska Open Concrete Bridgerail is proposed to be modified as follows:

- Installing the salvaged aluminum posts and pipe railing on top of the concrete bridge rail.
- Increasing the barrier height by 2 inches to accommodate a 2 inch, low slump, concrete overlay.
- Increasing the maximum post spacing from 8 feet to 9 feet to accommodate the geometry of the existing truss and aluminum railing.
- Adding a curb to reduce the vertical opening from 13 inches to 6 inches and to protect the rail from plow damage.
- Thickening the post elements to be flush with the faces of the beam element.
- Reducing the width and spacing of post elements on the final 64 inch long segment of rail at the end of the bridge (see elevation in Attachment 2).
- Reducing the gap at strip seal expansion joints from 4½ inches to 2 inches.
- Increasing the gap at two, finger style expansion joints from 4½ inches to 12¾ inches. A plate will be installed to cover the widened opening (see elevation in Attachment 2).
- Replacing the guardrail end connection with an ornamental end on top of the existing wing wall (see elevation in Attachment 2).

Evaluation of Proposed Bridge Railing

GEOMETRY

Historic Aluminum Railing

FHWA, the State Historic Preservation Office (SHPO), the Advisory Council on Historic Preservation (ACHP), the U.S. Army Corps of Engineers (USACE), and MnDOT signed a Programmatic Agreement (PA) in 2008 to streamline the review process for historic bridges. Stipulation 4 of this agreement requires the use of Context Sensitive Solutions (CSS) for FHWA and MnDOT projects. The PA defines CSS as "a collaborative, interdisciplinary approach that involves all stakeholders to develop a transportation facility that fits its physical setting and preserves scenic, aesthetic, historic and environmental resources, while maintaining safety and mobility."

The aluminum railing has been an identified as a character-defining feature of the Kennedy Bridge and elimination of this railing may constitute an adverse effect under Section 106 of the National Historic Preservation Act. To meet the goals of the project and the PA, it is proposed that the existing aluminum railing be included on top of the crash tested Nebraska Bridge Rail.

The primary concern for this rail is the snagging potential of a truck hood. To mitigate this potential, the post element of the aluminum railing would be set back 1¼ inches from the front face of the concrete beam element. The *MnDOT LRFD Bridge Design Manual* states that, for roadways with a design speed less than 40 mph, using a vertical faced concrete barrier "will cause more damage to a vehicle for minor hits but reduces the tendency for the vehicle to climb the face or roll over and will keep the vehicle back from the metal rail."

A diagram of the historic aluminum rail post is shown in Attachment 3. The Nebraska Bridge Rail has been crash tested without this metal railing attached to the top of the rail. Attachment of the aluminum railing could be considered as a design exception for the proposed, modified bridge rail.

Post Setback

The setback of the proposed concrete railing post element is reduced from 2 inches to 0 inches when compared with the crash tested Nebraska Bridge Rail. However, the addition of the curb element reduces the opening height.

Figure 5, from the *AASHTO LRFD Bridge Design Specifications*, shows the potential for a wheel to impact the post element for the crash tested and proposed rail configurations based on post setback and vertical clear opening. As shown in the figure, the Nebraska bridge rail meets NCHRP requirement for post setback vs. clear opening, but is not in the preferred zone. The proposed, modified barrier is in the preferred zone with a low potential for impact.

Figure 5. Post Impact Potential

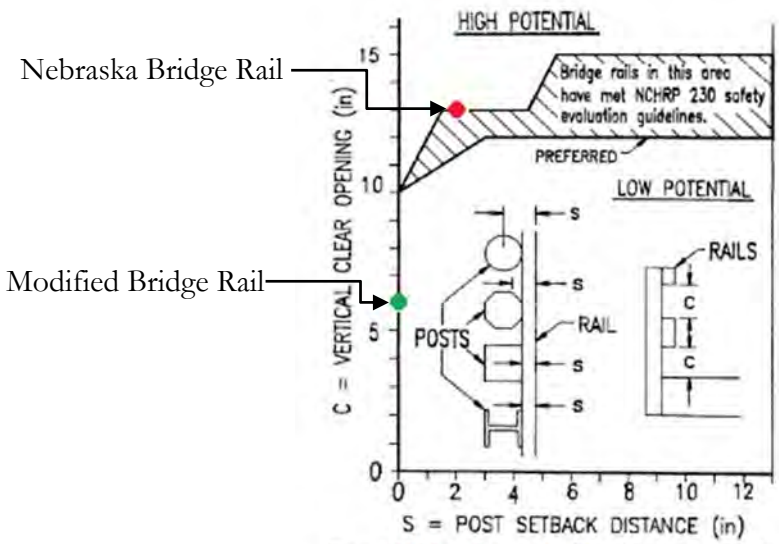
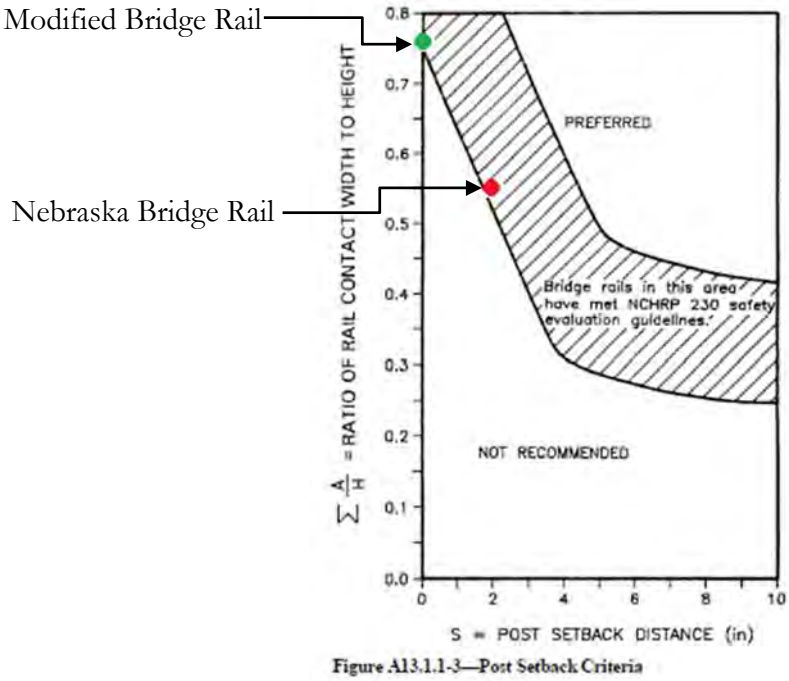


Figure 6, from the *AASHTO LRFD Bridge Design Specifications*, shows the wheel snagging potential based on the post setback versus the ratio of contact height to rail height. As shown in the figure, both the Nebraska Bridge Rail and the modified bridge rail meet NCHRP 230 safety evaluation guidelines.

Figure 6. Snagging Potential



Expansion Joint Gap

As shown in Attachment 1, the gap in the railing at expansion joints for the Nebraska Bridge Rail is 4½ inches. This gap in the proposed, modified barrier is reduced to 2 inches (see Attachment 2). The reduced gap will decrease the snagging potential of the bridge rail.

At two locations on the bridge, a 12¾ inch wide opening for a finger style expansion joint is required to accommodate thermal expansion/contraction. To reduce the potential for snagging, a cover plate (as shown in Attachment 2) would be installed over this opening to create a continuous surface for the beam and curb element.

STRUCTURAL CAPACITY

Modified Bridge Rail

The primary concern for the structural capacity of the proposed, modified bridge rail is the increased, maximum post spacing of 9 feet. Attachment 4 contains calculations for the bridge rail with the increased post spacing. Capacity calculations are based on Appendix A13 of the *AASHTO LRFD Bridge Design Specifications*.

The governing capacity of the bridge rail was found to be 64.2 kips, based on the two-span failure mechanism. This capacity exceeds the 54 kip capacity required for a TL-3 bridge rail.

Modified Bridge Rail End Segment

As shown in Attachment 2, the post geometry of the bridge rail is modified for the 64 inch long segment at the end of the bridge. Calculations for the capacity of this segment are shown in Appendix 5. The calculated capacity of 72.3 kips exceeds the 54 kip capacity required for a TL-3 bridge rail.

Bridge Rail End Segment on the Abutment Wing Wall

The segment of bridge rail at the bridge approaches would be constructed on top of the existing abutment wing wall. Achieving the 54 kip required capacity at this location would require reconstruction of the existing wing wall and a portion of the existing abutment; and would not be cost-effective for this small portion of bridge rail.

Federal-Aid Policy Guide Non-Regulatory Supplement NS CFR 23 625 (Transmittal 33) indicates that exceptions to upgrading railing may be considered when cost-effective considerations prevent full upgrades.

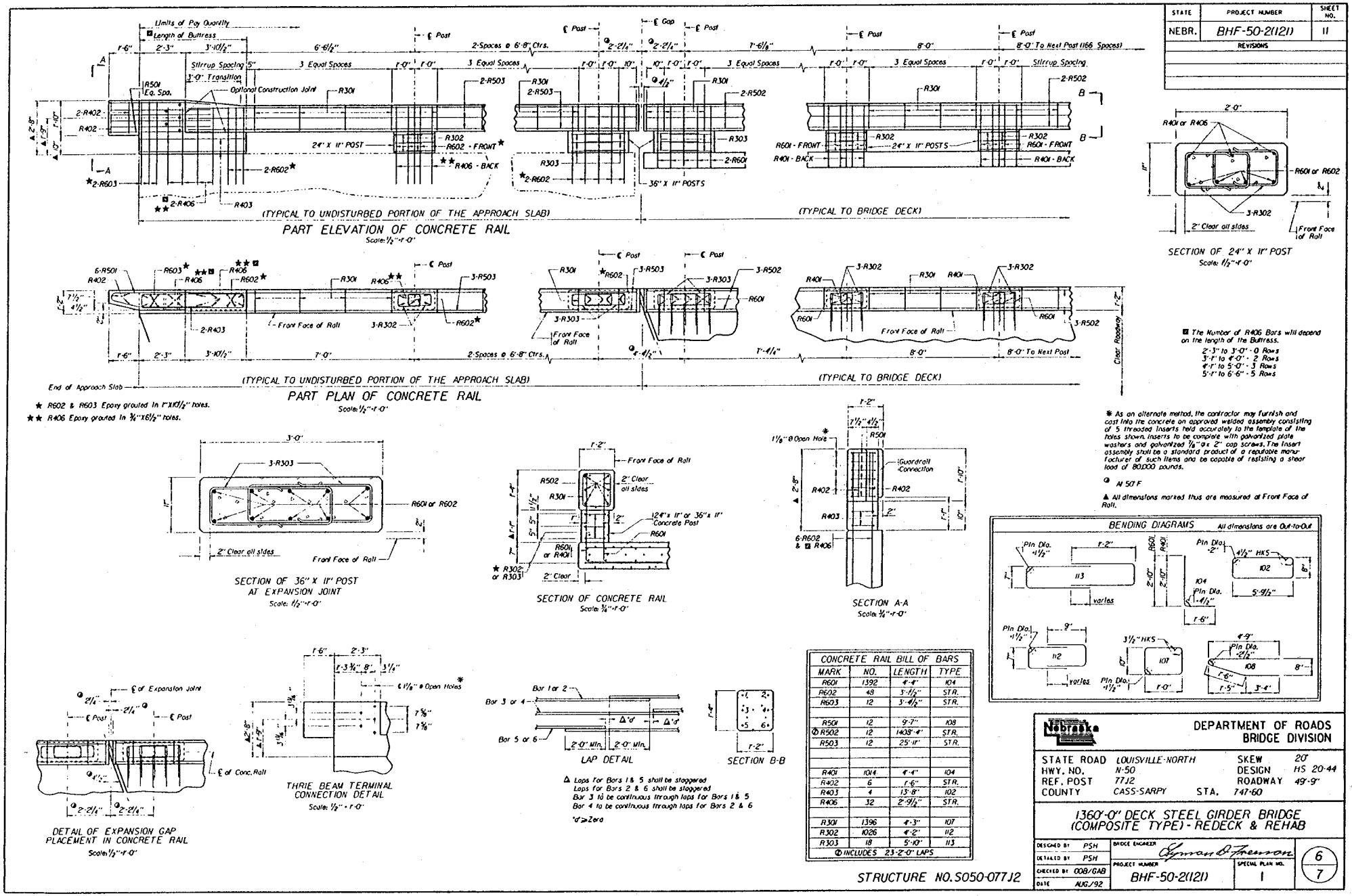
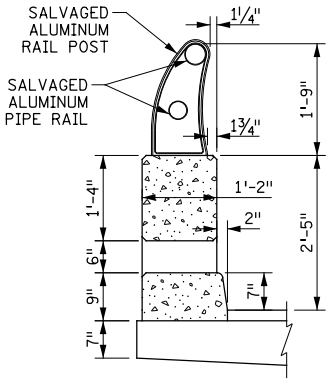
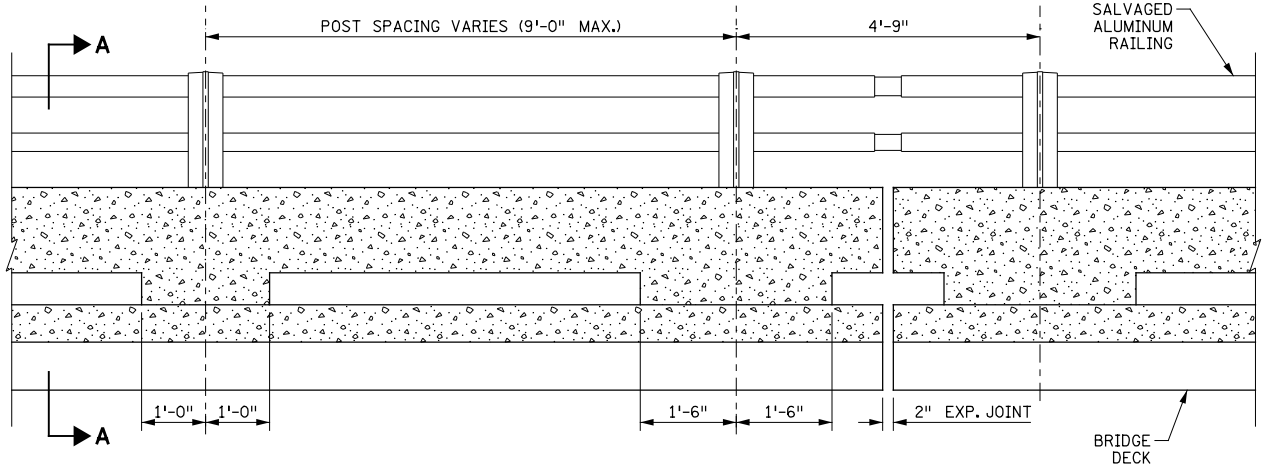


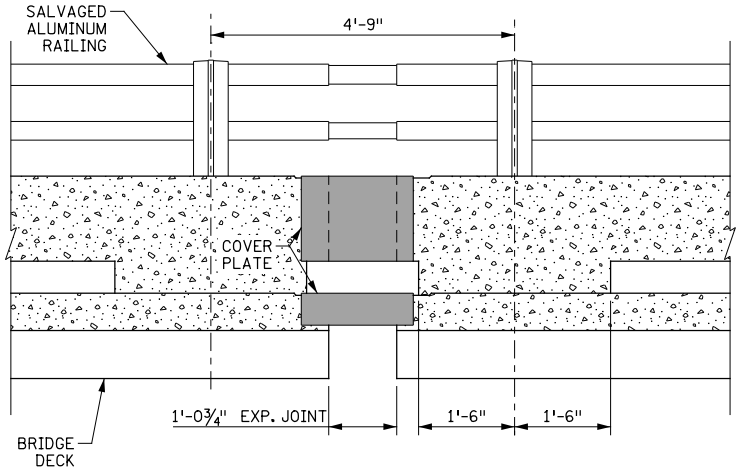
Figure B7.50. Nebraska Open Concrete Bridgerail (55,56).



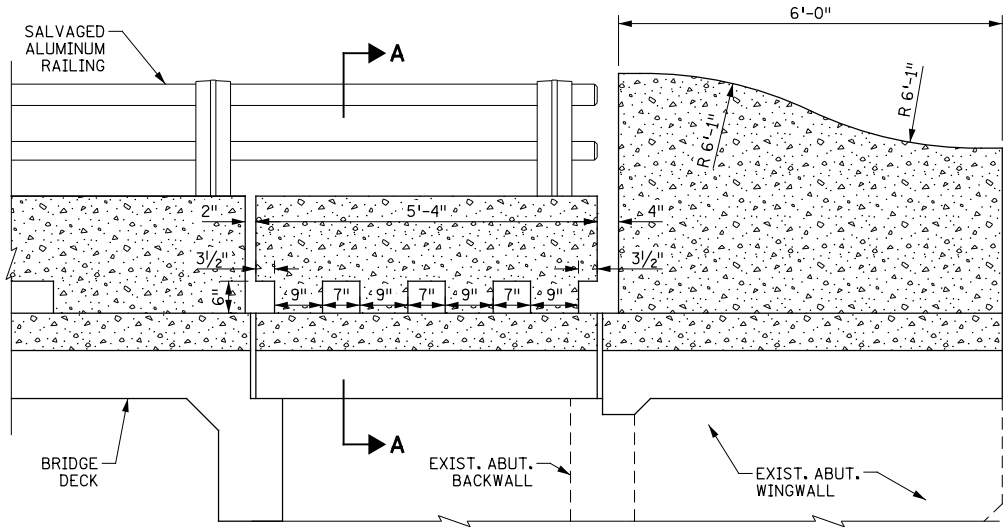
SECTION A-A



TYPICAL RAILING ELEVATION



RAILING ELEVATION
@ FINGER EXPANSION JOINT



RAILING ELEVATION
@ ABUTMENT

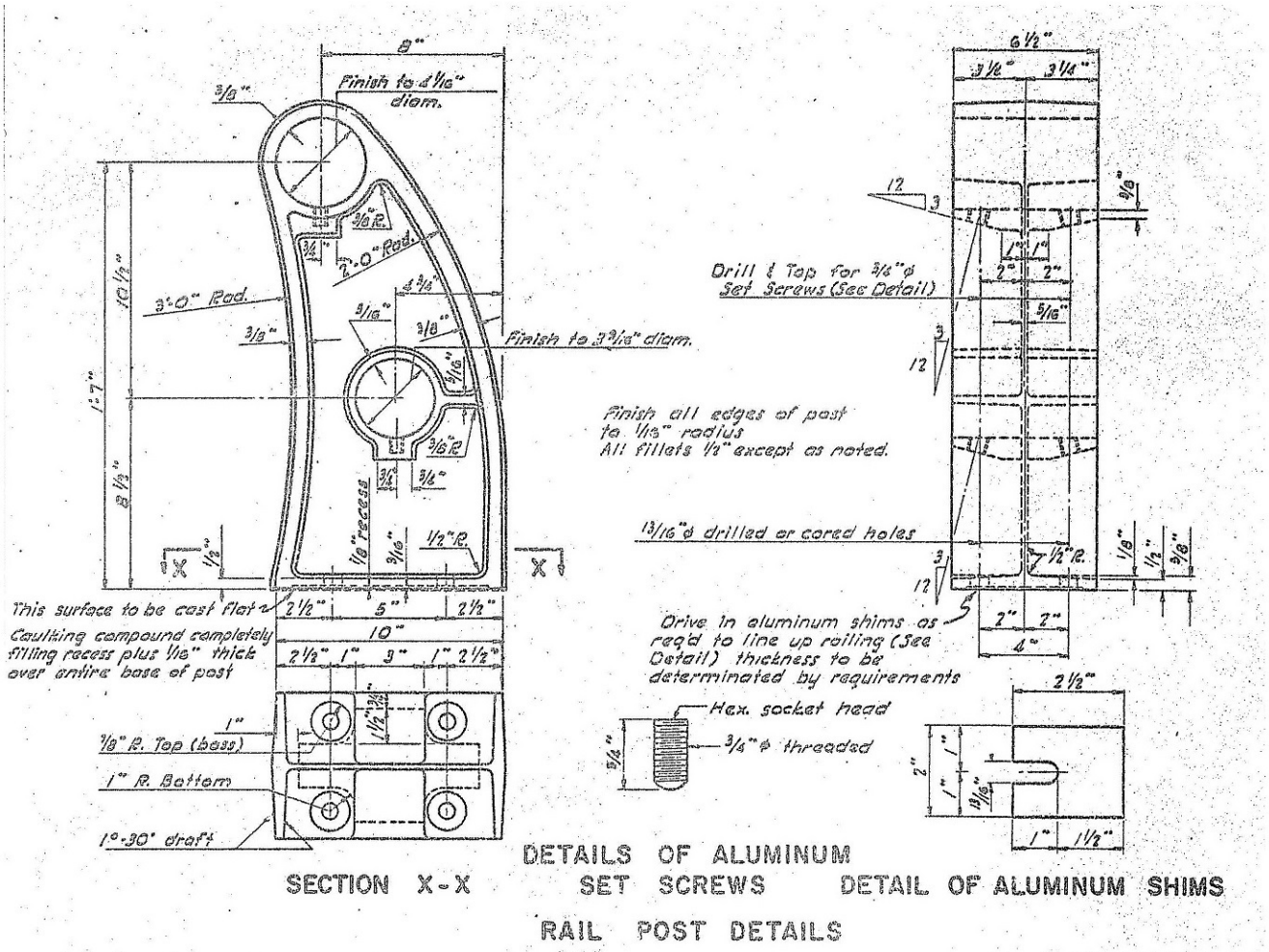
SRF Consulting Group, Inc.

Modified Nebraska TL-4 Bridge Rail

Bridge No. 9090 (Kennedy Bridge)

U.S. Trunk Highway 2 over the Red River of the North

Attachment 2


Finish to 4 1/16" diam.
Finish to 3 3/16" diam.
3'-0" Rad.
7'-0" Rad.
This surface to be cast flat
Caulking compound completely filling recess plus 1/16" thick over entire base of post
1/8" R. Top (base)
1" R. Bottom
1°-30' draft
Drill & Tap for 3/4" ϕ Set Screws (See Detail)
Finish all edges of post to 1/16" radius
All fillets 1/2" except as noted.
13/16" ϕ drilled or cored holes
Drive in aluminum shims as req'd to line up railing (See Detail) thickness to be determined by requirements
Hex. socket head
3/4" ϕ threaded
SECTION X-X
DETAILS OF ALUMINUM SET SCREWS
DETAIL OF ALUMINUM SHIMS
RAIL POST DETAILS

Structrual Capacity of Modified TL-4 Nebraska Open Bridge Rail

Designer: JMB

Checker: RMS

Bridge No. 9090 **Comm. No. 8727**

Design Criteria:

2014 AASHTO LRFD Bridge Design Specifications

I. Input Parameters:

$E_s := 29000 \cdot ksi$ Reinforcement modulus of elasticity

$f_y := 60 \cdot ksi$ Reinforcement Steel yield strength

$f_c := 4ksi$ Concrete Compressive Strength

$L_p := 9ft$ Post Spacing

$h_r := 29in$ Height of Rail

$h_b := 1ft + 4in$ Height of Beam Element

$t_b := 1ft + 2in$ Thickness of Beam Element

$w_p := 2ft$ Width of Interior Post Element

$w_{pe} := 3ft$ Width of Exterior Post Element

$t_p := 1ft + 2in$ Thickness of Post Element

$H_e := 24in$ Effective Height of TL-3 Rollover Force (AASHTO Table A13.2-1)

$t_{wc} := 2in$ Thickness of Wearing Course

$L_t := 4ft$ Length of TL-3 Horizontal Vehicle Impact Force (AASHTO Table A13.2-1)

$F_t := 54kip$ TL-3 Horizontal Vehicle Impact Force (AASHTO Table A13.2-1)

$clr := 2in$ Cover to Transverse Reinforcement

$\beta := 2.0$ $\theta := 45deg$ Diagonal Concrete Crack Factor/Angle (2.0 for members <16") per AASHTO 5.8.3.4.1

$\phi_f := 0.90$ Flexure Resistance Factor (AASHTO 5.5.4.2.1)

$\phi_v := 0.90$ Shear Resistance Factor (AASHTO 5.5.4.2.1)

$$\beta_1 := \max\left[0.85 - 0.05 \cdot \frac{\left(f_c - 4000 \cdot psi\right)}{1000 \cdot psi}, 0.65\right] = 0.85$$

Stress Block Factor (AASHTO 5.7.2.2)

II. Find Capacity of Interior Post:

$A_{sp} := 4 \cdot 0.44in^2 = 1.76 \cdot in^2$ — Area of Flexural Steel

$d_{bp} := 0.75in$ — Diameter of Flexural Steel

$A_{vp} := 4 \cdot 0.11in^2 = 0.44 \cdot in^2$ — Area of Shear Stirrup Steel

$d_v := 0.375in$ — Diameter of Shear Stirrup Steel

$s_{vp} := 5in$ — Spacing of Shear Stirrup Steel

$d_{sp} := t_p - clr - d_v - 0.5 \cdot d_{bp} - 0.625in = 10.6 \cdot in$ — Depth to Flexural Steel

$a_p := \frac{A_{sp} \cdot f_y}{0.85 \cdot f_c \cdot w_p} = 1.29 \cdot in$ — Depth of Compression Block

$d_{vp} := \max\left(0.72 \cdot t_p, 0.9 \cdot d_{sp}, d_{sp} - \frac{a_p}{2}\right) = 10.1 \cdot in$ — Effective Shear Depth (AASHTO 5.8.2.9)

$M_{pp} := \phi_f \cdot A_{sp} \cdot f_y \cdot \left(d_{sp} - \frac{a_p}{2}\right) = 79 \cdot ft \cdot kip$ — Yield Resistance Moment

$R_{fp} := \frac{M_{pp}}{H_e + t_{wc}} = 36.5 \cdot kip$ — Total Ultimate Flexural Resistance

$V_{cp} := 0.0316 \cdot \beta \cdot \sqrt{f_c \cdot ksi} \cdot w_p \cdot d_{vp} = 30.6 \cdot kip$ — Concrete Shear Resistance

$V_{sp} := \frac{A_{vp} \cdot f_y \cdot d_{vp} \cdot \cot(\theta)}{s_{vp}} = 53.2 \cdot kip$ — Steel Shear Resistance

$V_{np} := \phi_v \cdot \min\left(0.25 \cdot f_c \cdot w_p \cdot d_{vp}, V_{cp} + V_{sp}\right) = 75.4 \cdot kip$ — Total Ultimate Shear Resistance

$P_p := \min\left(R_{fp}, V_{np}\right) = 36.5 \cdot kip$ — Total Ultimate Post Resistance

II. Find Capacity of End Post (Adjacent to Expansion Gap):

$A_{spe} := 5 \cdot 0.44in^2 = 2.2 \cdot in^2$ — Area of Flexural Steel

$d_{bpe} := 0.75in$ — Diameter of Flexural Steel

$A_{vpe} := 4 \cdot 0.11in^2 = 0.44 \cdot in^2$ — Area of Shear Stirrup Steel

$d_{spe} := t_p - clr - d_v - 0.5 \cdot d_{bp} - 0.625in = 10.6 \cdot in$ — Depth to Flexural Steel

$a_{pe} := \frac{A_{spe} \cdot f_y}{0.85 \cdot f_c \cdot w_{pe}} = 1.08 \cdot in$ — Depth of Compression Block

$d_{vpe} := \max\left(0.72 \cdot t_p, 0.9 \cdot d_{spe}, d_{spe} - \frac{a_{pe}}{2}\right) = 10.1 \cdot in$ — Effective Shear Depth (AASHTO 5.8.2.9)

$M_{ppe} := \phi_f \cdot A_{spe} \cdot f_y \cdot \left(d_{spe} - \frac{a_{pe}}{2}\right) = 99.8 \cdot ft \cdot kip$ — Yield Resistance Moment

$R_{fpe} := \frac{M_{ppe}}{H_e + t_{wc}} = 46.1 \cdot kip$ — Total Ultimate Flexural Resistance

$V_{cpe} := 0.0316 \cdot \beta \cdot \sqrt{f_c \cdot ksi} \cdot w_{pe} \cdot d_{vpe} = 45.9 \cdot kip$ — Concrete Shear Resistance

$V_{spe} := \frac{A_{vpe} \cdot f_y \cdot d_{vpe} \cdot \cot(\theta)}{s_{vp}} = 53.3 \cdot kip$ — Steel Shear Resistance

$V_{npe} := \phi_v \min\left(0.25 \cdot f_c \cdot w_{pe} \cdot d_{vpe}, V_{cpe} + V_{spe}\right) = 89.2 \cdot kip$ — Total Ultimate Shear Resistance

$P_{pe} := \min\left(R_{fpe}, V_{npe}\right) = 46.1 \cdot kip$ — Total Ultimate Post Resistance

II. Find Capacity of Rail for Transverse Impact:

$A_{st} := 3 \cdot 0.31 in^2 = 0.93 \cdot in^2$ — Area of Flexural Steel

$d_{bt} := 0.625 in$ — Diameter of Flexural Steel

$L_b := L_p - w_p = 7\ ft$ — Length of Beam Element Between Posts (Modified as Mp takes place at edge of post)

$d_{st} := t_b - clr - d_v - 0.5 d_{bt} = 11.3 \cdot in$ — Depth to Flexural Steel

$a_t := \frac{A_{st} \cdot f_y}{0.85 \cdot f_c \cdot h_b} = 1.03 \cdot in$ — Depth of Compression Block

$d_{vt} := \max\left(0.72 \cdot t_b, 0.9 \cdot d_{st}, d_{st} - \frac{a_t}{2}\right) = 10.8 \cdot in$ — Effective Shear Depth (AASHTO 5.8.2.9)

$M_{pt} := \phi_f \cdot A_{st} \cdot f_y \cdot \left(d_{st} - \frac{a_t}{2}\right) = 45.2 \cdot ft \cdot kip$ — Yield Resistance Moment

$N := 1$ — Number of railing spans

$R_{fti} := \frac{16 \cdot M_{pt} + (N-1) \cdot (N+1) \cdot P_p \cdot L_b}{2 \cdot N \cdot L_b - L_t} = 72.3 \cdot kip$ — Total Ultimate Resistance of Interior Segment (AASHTO 13.3.2.1 for Single Span Failure)

$R_{fte} := \frac{2 \cdot M_{pt} + 2.P_{pe} \cdot L_b \cdot \sum_{i=1}^{N} i}{2 \cdot L_b - L_t} = 73.6 \cdot kip$ — Total Ultimate Resistance of End Segment (AASHTO 13.3.2.1 for Single Span Failure)

$R_1 := \min(R_{fti}, R_{fte}) = 72.3 \cdot kip$ — Total Ultimate Resistance of Railing (One Span Failure)

$N := 2$ — Number of railing spans

$R_{fti} := \frac{16 \cdot M_{pt} + N^2 \cdot P_p \cdot L_p}{2 \cdot N \cdot L_p - L_t} = 63.6 \cdot kip$ — Total Ultimate Resistance of Interior Segment (AASHTO 13.3.2.1 for Two Span Failure)

$R_{fte} := \frac{2 \cdot M_{pt} + 2.P_{pe} \cdot L_p \cdot \sum_{i=1}^{N} i}{2 \cdot N \cdot L_p - L_t} = 80.6 \cdot kip$ — Total Ultimate Resistance of End Segment (AASHTO 13.3.2.1 for Two Span Failure)

$R_2 := \min(R_{fti}, R_{fte}) = 63.6 \cdot kip$ — Total Ultimate Resistance of Railing (Two Span Failure)

$N := 3$ Number of railing spans

$$R_{fti} := \frac{16 \cdot M_{pt} + (N-1) \cdot (N+1) \cdot P_p \cdot L_p}{2 \cdot N \cdot L_p - L_t} = 67 \cdot kip$$

Total Ultimate Resistance of Interior Segment
(AASHTO 13.3.2.1 for Three Span Failure)

$$R_{fte} := \frac{2 \cdot M_{pt} + 2 \cdot P_{pe} \cdot L_p \cdot \sum_{i=1}^{N} i}{2 \cdot N \cdot L_p - L_t} = 101.3 \cdot kip$$

Total Ultimate Resistance of End Segment
(AASHTO 13.3.2.1 for Three Span Failure)

$$R_3 := \min(R_{fti}, R_{fte}) = 67 \cdot kip$$

Total Ultimate Resistance of Railing
(Three Span Failure)

$N := 4$ Number of railing spans

$$R_{fti} := \frac{16 \cdot M_{pt} + N^2 \cdot P_p \cdot L_p}{2 \cdot N \cdot L_p - L_t} = 87.9 \cdot kip$$

Total Ultimate Resistance of Interior Segment
(AASHTO 13.3.2.1 for Four Span Failure)

$$R_{fte} := \frac{2 \cdot M_{pt} + 2 \cdot P_{pe} \cdot L_p \cdot \sum_{i=1}^{N} i}{2 \cdot N \cdot L_p - L_t} = 123.3 \cdot kip$$

Total Ultimate Resistance of End Segment
(AASHTO 13.3.2.1 for Four Span Failure)

$$R_4 := \min(R_{fti}, R_{fte}) = 87.9 \cdot kip$$

Total Ultimate Resistance of Railing
(Four Span Failure)

$N := 5$ Number of railing spans

$$R_{fti} := \frac{16 \cdot M_{pt} + (N-1) \cdot (N+1) \cdot P_p \cdot L_p}{2 \cdot N \cdot L_p - L_t} = 100 \cdot kip$$

Total Ultimate Resistance of Interior Segment
(AASHTO 13.3.2.1 for Five Span Failure)

$$R_{fte} := \frac{2 \cdot M_{pt} + 2 \cdot P_{pe} \cdot L_p \cdot \sum_{i=1}^{N} i}{2 \cdot N \cdot L_p - L_t} = 145.7 \cdot kip$$

Total Ultimate Resistance of End Segment
(AASHTO 13.3.2.1 for Five Span Failure)

$$R_5 := \min(R_{fti}, R_{fte}) = 100 \cdot kip$$

Total Ultimate Resistance of Railing
(Five Span Failure)

Total Ultimate Resistance of Railing $\boxed{R := \min(R_1, R_2, R_3, R_4, R_5) = 63.6 \cdot kip}$

Adequacy of Railing for TL-3 Loading $\boxed{Check := \text{if}(F_t < R, \text{"OK for TL-3"}, \text{"NO GOOD"}) = \text{"OK for TL-3"}}$

SRF Consulting Group, Inc.

8/6/2015

Structrual Capacity of Ornamental End Section on Bridge

Designer: JMB

Checker: RMS

Bridge No. 9090 **Comm. No. 8727** **Design Criteria:**

2014 AASHTO LRFD Bridge Design Specifications

I. Input Parameters:

$E_s := 29000 \cdot ksi$ Reinforcement modulus of elasticity

$f_y := 60 \cdot ksi$ Reinforcement Steel yield strength

$f_c := 4ksi$ Concrete Compressive Strength

$h_r := 29in$ Height of Rail

$t_r := 1ft + 2in$ Thickness of Rail

$w_p := 9in$ Width of Post Element

$N_p := 4$ Number of Post Elements Effective in Resisting Rail Impact Force

$H_e := 24in$ Effective Height of TL-3 Rollover Force (AASHTO Table A13.2-1)

$t_{wc} := 2in$ Thickness of Wearing Course

$L_t := 4ft$ Length of TL-3 Horizontal Vehicle Impact Force (AASHTO Table A13.2-1)

$F_t := 54kip$ TL-3 Horizontal Vehicle Impact Force (AASHTO Table A13.2-1)

$clr := 2in$ Cover to Transverse Reinforcement

$\beta := 2.0$ $\theta := 45deg$ Diagonal Concrete Crack Factor/Angle (2.0 for members <16") per AASHTO 5.8.3.4.1

$\phi_f := 0.90$ Flexure Resistance Factor (AASHTO 5.5.4.2.1)

$\phi_v := 0.90$ Shear Resistance Factor (AASHTO 5.5.4.2.1)

$$\beta_1 := \max\left[0.85 - 0.05 \cdot \frac{(f_c - 4000 \cdot psi)}{1000 \cdot psi}, 0.65\right] = 0.85$$

Stress Block Factor (AASHTO 5.7.2.2)

II. Find Capacity of Ornamental Railing End Section:

$A_s := 2 \cdot 0.44in^2$ <-------Use 2 #6 bars at the front face of each post — Area of Flexural Steel Per Post

$d_b := 0.75in$ — Diameter of Flexural Steel

$A_v := 2 \cdot 0.11in^2$ — Area of Shear Stirrup Steel Per Post

$d_v := 0.375in$ — Diameter of Shear Stirrup Steel

$s_v := 5in$ — Spacing of Shear Stirrup Steel

$d_s := t_r - clr - d_v - 0.5 \cdot d_b - 0.625in = 10.6 \cdot in$ — Depth to Flexural Steel

$a_p := \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot w_p} = 1.73 \cdot in$ — Depth of Compression Block

$d_v := \max\left(0.72 \cdot t_r, 0.9 \cdot d_s, d_s - \frac{a_p}{2}\right) = 10.1 \cdot in$ — Effective Shear Depth (AASHTO 5.8.2.9)

$\phi M_n := \phi_f \cdot A_s \cdot f_y \cdot \left(d_s - \frac{a_p}{2}\right) = 38.7 \cdot ft \cdot kip$ — Yield Resistance Moment of Single Post

$R_M := N_p \cdot \frac{\phi M_n}{H_e + t_{wc}} = 71.4 \cdot kip$ — Flexural Impact Resistance of End Segment

$V_c := 0.0316 \cdot \beta \cdot \sqrt{f_c \cdot ksi} \cdot w_p \cdot d_v = 11.5 \cdot kip$ — Concrete Shear Resistance of Single Post

$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{s_v} = 26.6 \cdot kip$ — Steel Shear Resistance of Single Post

$\phi V_n := \phi_v \cdot \min\left(0.25 \cdot f_c \cdot w_p \cdot d_v, V_c + V_s\right) = 34.3 \cdot kip$ — Total Ultimate Shear Resistance of Single Post

$R_V := N_p \cdot \phi V_n = 137.1 \cdot kip$ — Shear Impact Resistance of End Segment

$R := \min\left(R_M, R_V\right) = 71.4 \cdot kip$ — Total Impact Resistance of End Segment

$Check := \text{if}\left(F_t < R, \text{"OK for TL-3"}, \text{"NO GOOD"}\right) = \text{"OK for TL-3"}$ — Adequacy of Railing for TL-3 Loading

Appendix K:
Existing Bridge Plans

21-161 cont. 11240

A. E. LaBonte.

FED. ROAD DIV. NO.	STATE	FED. AID PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	F-708(1)		1	43

NORTH DAKOTA
STATE HIGHWAY DEPARTMENT

PLANS

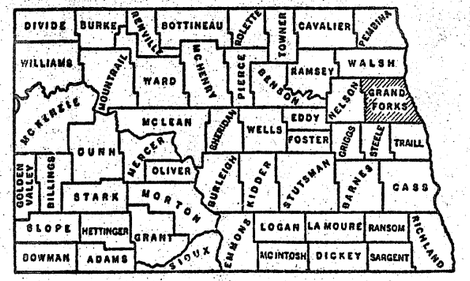
FOR THE PROPOSED IMPROVEMENT OF A

STATE HIGHWAY

IN GRAND FORKS CO. N.D. & POLK CO. MINN.

FEDERAL AID PROJECT NO. F-708(1)

RED RIVER BRIDGE



SKETCH-MAP OF NORTH DAKOTA SHOWING COUNTIES

SCALES
- LAYOUT SHEET: 1 IN. = 750'
- PLAN AND PROFILE DRAWINGS: HOR. 1 IN. = 100 FT. VERT. 1 IN. = 10 FT.
- STRUCTURAL DRAWINGS:- AS SHOWN
- CROSS SECTION SHEETS:- 1 IN. = 10 FT.

INDEX OF DRAWINGS

SHEET NO. 1 TITLE PAGE
SHEET NO. 2 TO 43 INCL. STRUCTURAL DRAWINGS
SHEET 1A,1B,1C PLAN & PROFILE SHEETS

SHEETS 1A & 1B FROM PROJECT F-364(10) (Sts.19 & 20).

LENGTH OF PROJECT		
PROJECT	MILES-GROSS	MILES-NET
F-708(1)	0.238	0.238
TOTALS	0.238	0.238

LIST OF STRUCTURAL DRAWINGS
708(1)-1 thru 708(1)-40
708(1)

LIST OF STANDARDS

Bridge Bench Marks	Std. 7G
Standard Signs-Type A.	Std. 14.1A-1 & 2
Constr. Identification Sign	Std. 14.25
Pile Splice Details	Std. H-0401
Screed chair	Std. H-0501

End F-708(1) Sta. 1161+37.5 =
A point 115.9' North and 2489.3' East of the NW Cor. Sec. 3, T-151 N., Rge. 50 W.

GOVERNING SPECIFICATIONS:

Standard Specifications adopted by the North Dakota State Highway department July, 1961 and approved as standard by the Bureau of Public Roads Dec. 29, 1961 Required Special Provisions dated May 7, 1962 and approved by the Bureau of Public Roads May 18, 1962 and others submitted herewith

KEY TO CONVENTIONAL SIGNS

STATE & NATIONAL LINES
COUNTY LINE
TOWNSHIP & RANGE LINES
GRADE LINE
CENTERLINE OF CONSTRUCTION
OLD RIGHT OF WAY LINE
NEW RIGHT OF WAY LINE
ABANDONED RIGHT OF WAY LINE
PROPERTY LINE
STONE WALL
OTHER FENCES
POLE LINES
POWER LINES
BRIDGE
GROUND GRADE ELEVATION
TRAVELED WAY
RAILROADS
HEDGES AND TREES
TRAILS
CITY OR VILLAGE CORPORATE LIMITS
SECTION CORNER
QUARTER SECTION CORNER
BUILDINGS
OLD CULVERTS
NEW CULVERTS
DRAINAGE
BENCH MARKS
WATERS EDGE
MARSH
WIRE ROPE GUARD RAIL
SNOW FENCE

DESIGN DATA

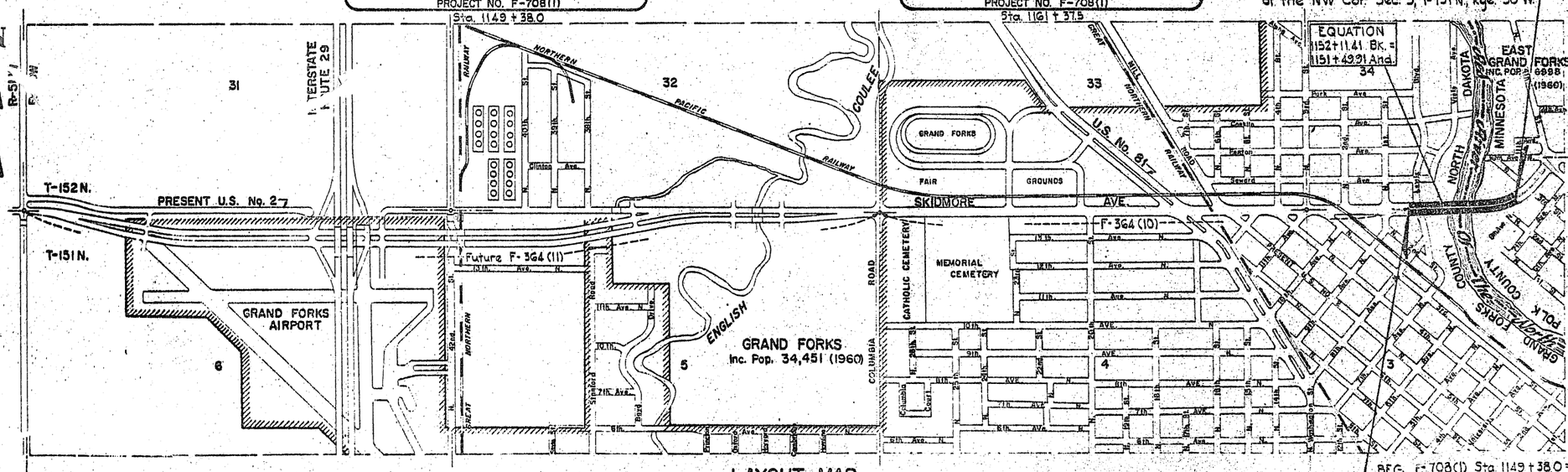
TRAFFIC		AVERAGE DAILY	EST. 30TH MAX. HR.
CURRENT TRAFFIC	(1962)	9,900 PASS. 1,100 TRUCKS 11,000 TOTAL	1500
TRAFFIC FORECAST	(1982)	19,500 PASS. 1,500 TRUCKS 15,000 TOTAL	1900

DESIGN SPEED 35 MPH
TRAFFIC CLASSIFICATION "M"
BRIDGE DESIGN LOADING - H20 S16 (1957) and alternate loading designated in P.P.M. 20-4, Section 4C.

STATE-FEDERAL HIGHWAY CONSTRUCTION
0.2 MILES COST $0,000,000
STATE FUNDS 50% FEDERAL-AID FUNDS 50%
NORTH DAKOTA HIGHWAY DEPARTMENT — U.S. DEPT. OF COMMERCE BUREAU OF PUBLIC ROADS
PROJECT NO. F-708(1)

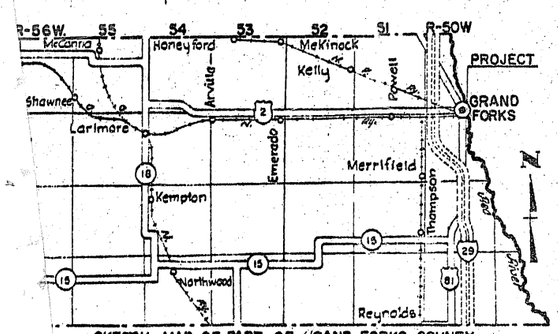
Construction Identification Sign No. PC1-1

STATE-FEDERAL HIGHWAY CONSTRUCTION
0.2 MILES COST $0,000,000
STATE FUNDS 50% FEDERAL-AID FUNDS 50%
MINNESOTA HIGHWAY DEPARTMENT — U.S. DEPT. OF COMMERCE BUREAU OF PUBLIC ROADS
PROJECT NO. F-708(1)



BEG. F-708(1) Sta. 1149+38.0 =
a. 1149+38.0 End F-364(10)
a point 1.4' North and 1235.2' East of the NW Cor. Sec. 3, T-151 N., Rge. 50 W.

LAYOUT MAP
750' 0 750'
SCALE IN FEET



SKETCH MAP OF PART OF GRAND FORKS COUNTY

NOTE:

This contract shall be let on a work day basis with the penalties imposed as outlined in the special provisions. Cooperation must be maintained between various contractors to enable each to complete his contract within the specified time limits.

Structural operations must be completed to such an extent that the project can be opened to traffic by November 1, 1963.

① SEE SPECIAL PROVISIONS

QUANTITIES

LOCATIONS	15B Class 3 Excavation	15C Class 3 Excavation	60A Class AE-2 Concrete	60B Class BE-2 Concrete	62A Reinforc'g Steel (Intermed. Grade)	65C Steel Piling (14BP73)	65K Steel Test Pile (14BP73) 135 ft. Long	Loading Test 300 Tons	63A Structural Steel	63B Structural Low-Alloy Steel	81 Ornamental Metal Railing	Structural Quenched & Tempered Steel	Bridge Bench Marks	Lighting System	Linseed Oil Treatment
	C.Y.	C.Y.	C.Y.	C.Y.	LB.	L.F.	No.	No.	LB.	LB.	L.F.	LB.	Sta.	Lump Sum	Gal.
1154 + 75.0 ℄	1200	965	4001.1	162.0	740,415	25,925	2	2	1485,262	2,327,680	2520.67	70,000	1	1	450
TALS	1200	965				25,925					2520.67	70,000	1		
	① P100 76A	① SP										①			

STATE OF MINNESOTA
DEPARTMENT OF HIGHWAYS
S.P. 6018-9090 (T.H. 307=307)
APPROVED: 12-7-61
A. E. LaBonte — BRIDGE ENGR.
DEPUTY CHIEF ENGR.

APPROVED DATE 7-26-62
CHIEF ENGINEER
NORTH DAKOTA STATE HIGHWAY DEPARTMENT

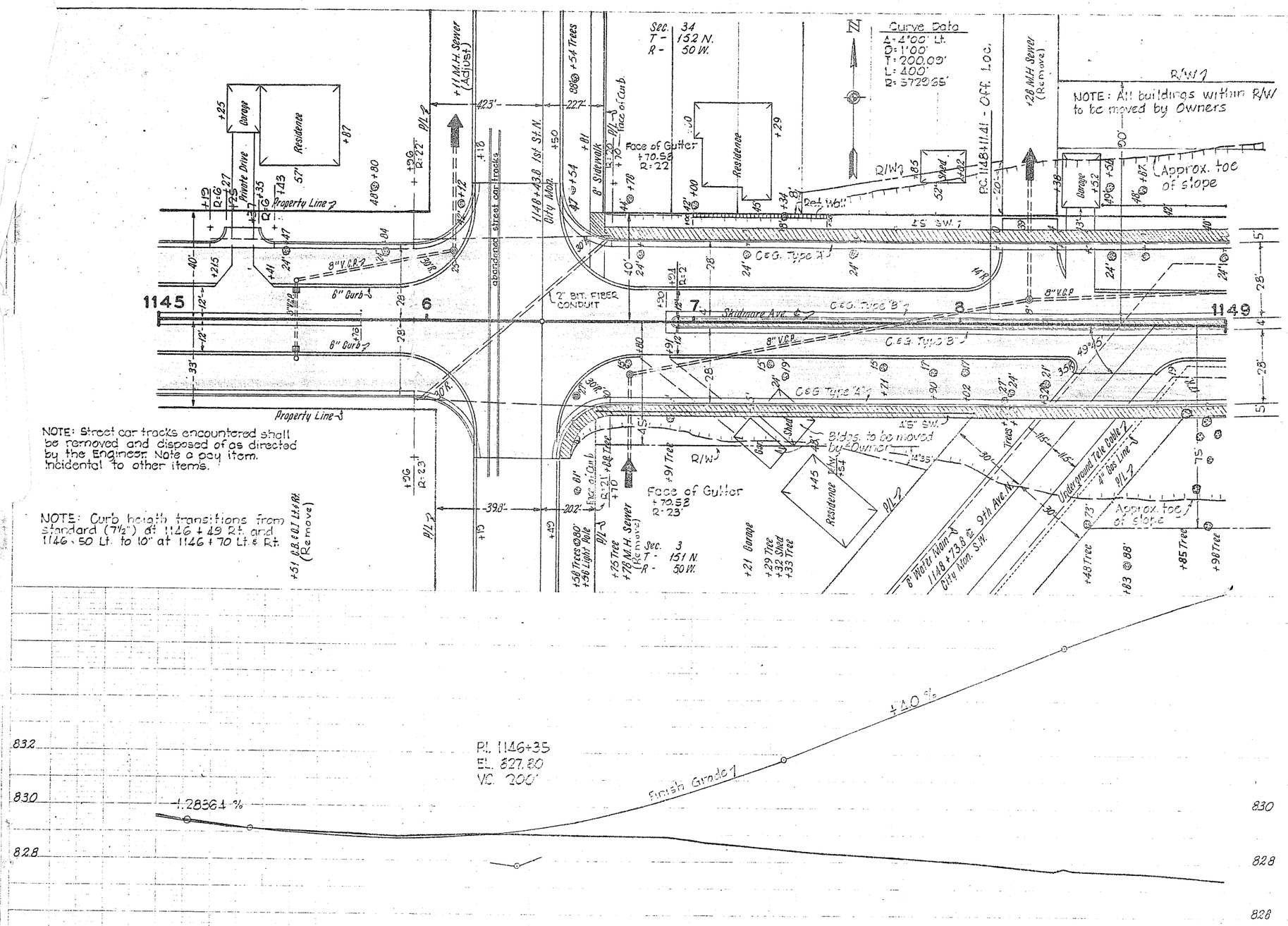
Br. 9090

DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS

APPROVED: 900

FED. AID DIV. NO.	STATE	FED. AID PROJECT NO.		SHEET NO.	TOTAL SHEETS
5	N.D.	F-384	(10)	19	10

F 708 (1) 1A 43



NOTE: Street car tracks encountered shall be removed and disposed of as directed by the Engineer. Note a pay item. Incidental to other items.

NOTE: Curb height transitions from standard (7½") at 1146+49 Rt. and 1146+50 Lt. to 10" at 1146+70 Lt. & Rt.

INSTALL 8" P.C.C. PAVEMENT

1145+00	to	1146+43.6	967.7 S.Y.
1146+43.6	to	1149+00	1455.2 S.Y.

INSTALL CONC. DRIVEWAYS

1145+31	Lt.	12'	14.7 S.Y.

INSTALL COMB. CONC. CURB & GUTTER

1145+00	Lt.	to	1145+19 Lt.	19.0 L.F.
1145+00	Rt.	to	1146+19 Rt.	132.1 L.F.
1145+43	Lt.	to	1146+18 Lt.	87.6 L.F.
1146+50	Lt.	to	1149+00 Lt.	261.0 L.F.
1146+49	Rt.	to	1149+00 Rt.	263.5 L.F.
1146+50	Lt.	to	1149+00 Lt.	209.1 L.F.
1146+50	Lt.	to	1149+00 Rt.	209.1 L.F.

INSTALL CONC. SIDEWALK

1148+61	to	1149+00 Lt.	118.3 S.Y.
1148+49.7	to	1149+00 Rt.	124.6 S.Y.
1148+50	to	1149+00 ℄	63.4 S.Y.

ADJUST MANHOLES TO GRADE

1148+11 - 25' Lt. 1-Ea.

INSTALL CONC. RETAINING WALL

1147+00 to 1147+50 Lt.
DWG. NO. 384(10)-1
Conc. Class AE-2 5.7 C.Y.
Reinforcing Steel 284 LBS.
(See sheet 45 for details)

RESET MONUMENT BOXES

1146+48.6 1st St. N. 1-Ea.

INSTALL 2" BIT. FIBER CONDUIT

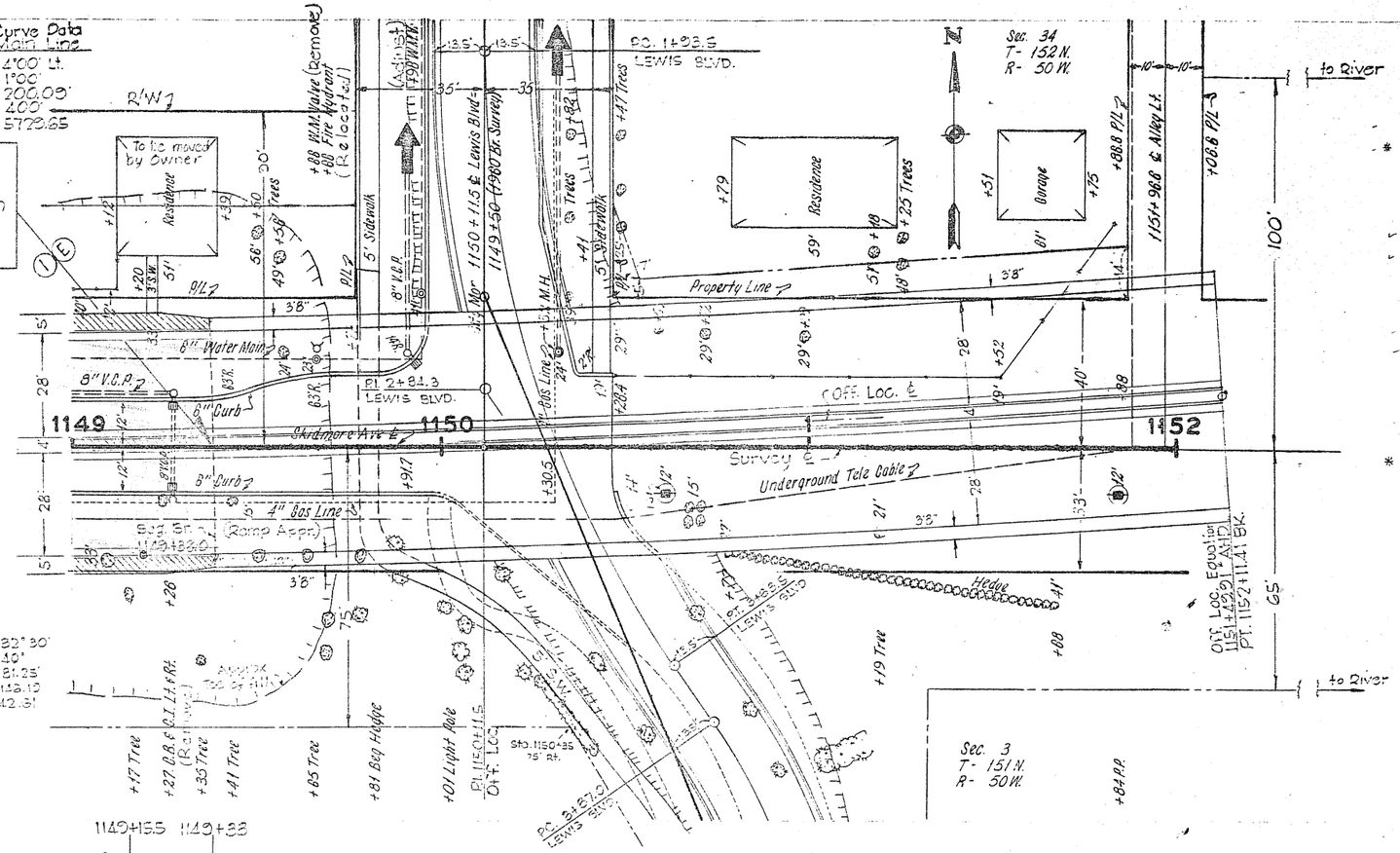
1145+19	to	1145+48 - 28' Lt.	24 LF.
1145+36	to	1146+70 - 30' Lt.	74 LF.
1145+36	to	1146+70 - 30' Rt.	74 LF.
1145+36 - 30' Rt.	to	1146+70 - 30' Lt.	130 LF. (Dbl.)

BR 9090

FED. AID SYY. NO.	STATE	FED. AID PROJECT NO.		SHEET NO.	TOTAL SHEETS
S	N.D.	F-364	(10)	20	102
		F708(1)		1B	43

Curve Data
Main Line
Δ = 4°00' Lt.
D = 1°00'
T = 200.09'
L = 400'
R = 5729.65

END F-364(10)
STA. 1149+38.0 =
STA. 1149+38.0 on
FUT. F-708(1)
(BRIDGE)



CURVE DATA
LEWIS BLVD

Δ = 35°00'	Δ = 32°30'
D = 20°	D = 40°
L = 175'	L = 81.25'
R = 287.94'	R = 143.13'
T = 90.79'	T = 42.31'

INSTALL 8" P.C.C. PAVEMENT

1149+00	to	1149+24	128.0	S.Y.
* 1149+24	to	1149+38.5	77.3	S.Y.

INSTALL COMB. CONC. CURB & GUTTER - TYPE "A"

1149+00	to	1149+38.5	Lt.	38.4 L.F.
1149+00	to	1149+38.5	Rt.	38.6 L.F.

INSTALL COMB. CONC. CURB & GUTTER - TYPE "B"

1149+00	to	1149+38.5	Lt.	38.5 L.F.
1149+00	to	1149+38.5	Rt.	38.5 L.F.

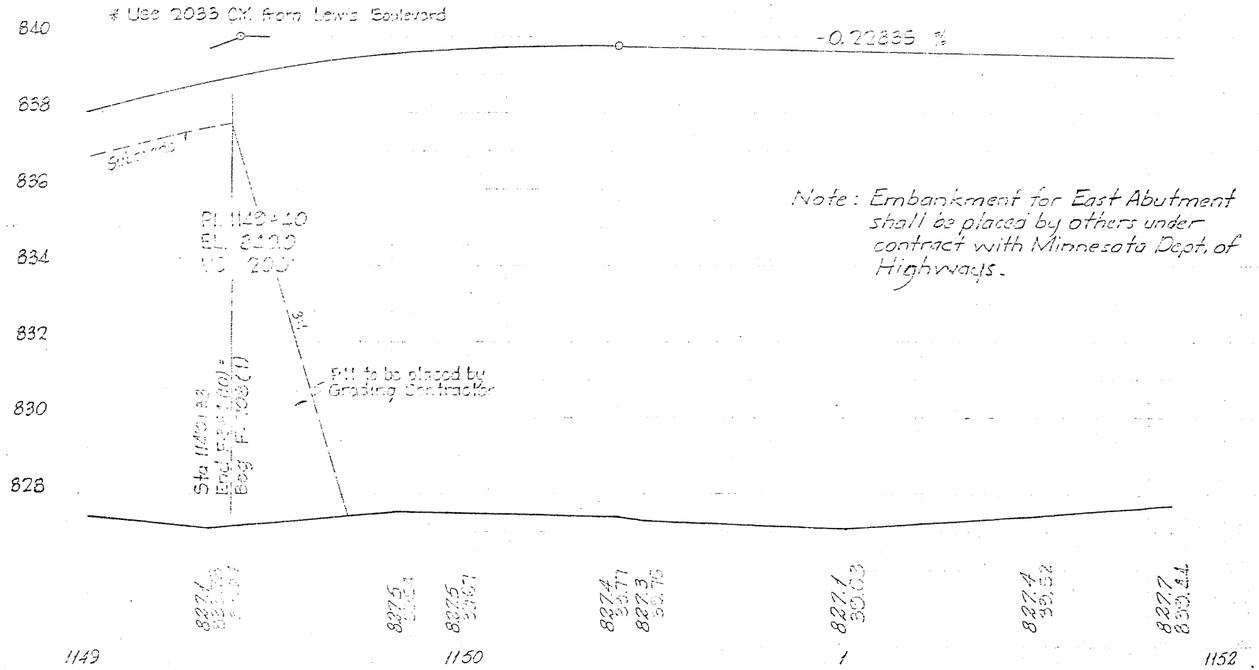
INSTALL CONC. SIDEWALKS

1149+00	to	1149+38.5	Lt.	17.8 S.Y.
1149+00	to	1149+38.5	Rt.	17.7 S.Y.
1149+00	to	1149+38.5	℄	11.8 S.Y.

* See Pavement Detail sheet # 43 for details on bridge approach slab sta. 1149+24 - 1149+38.5

1149+15.5 1149+38

* Exc. 0 C.Y.
Emb. 2038 C.Y.
* Use 2038 C.Y. from Lewis Boulevard

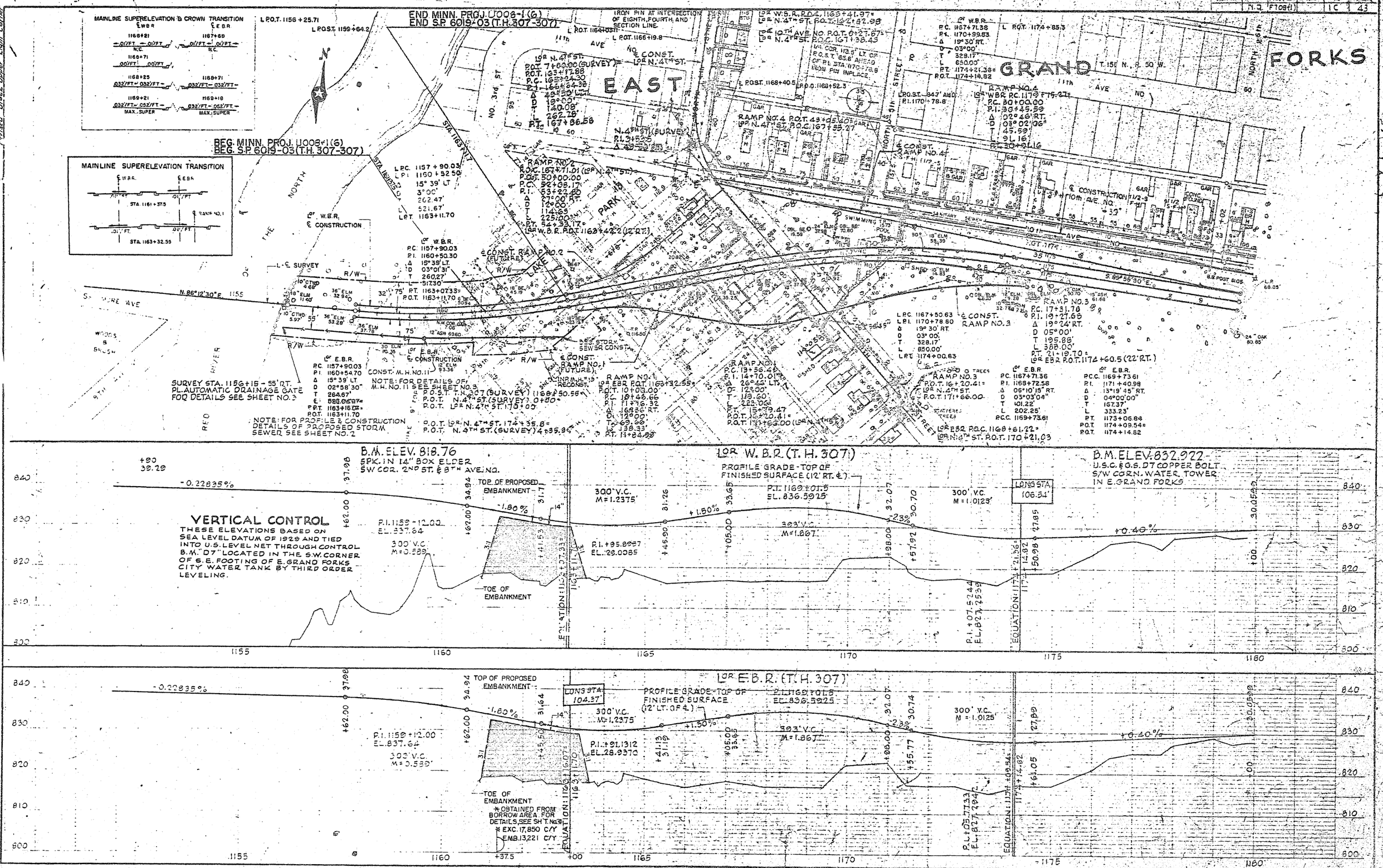


Note: Embankment for East Abutment shall be placed by others under contract with Minnesota Dept. of Highways.

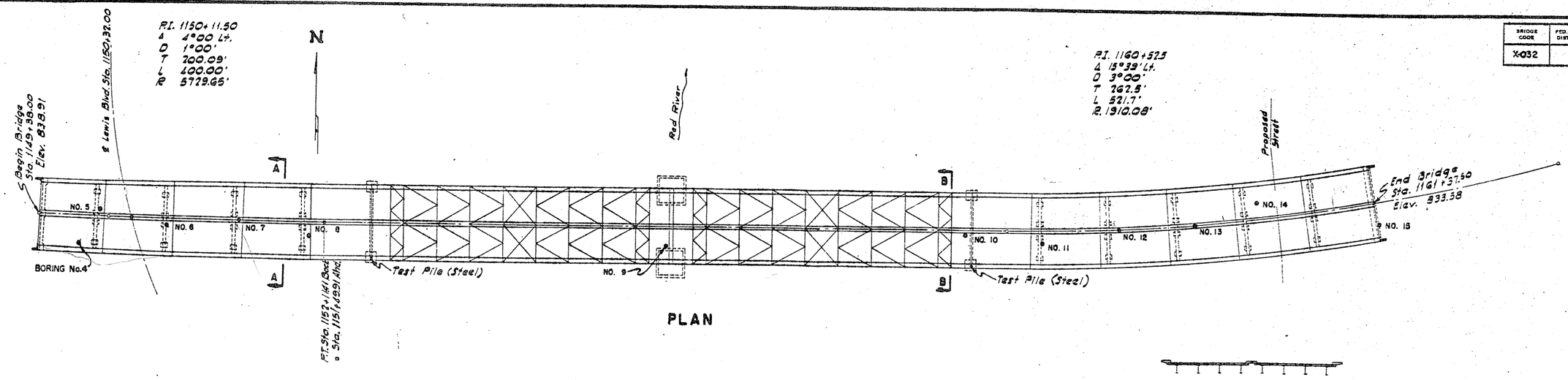
Bench Mark			
No.	Description	Location	Elev.
13	S.W. Cor. of Residence Top Step Conc. Steps	1149+21 - 41' Lt.	829.39

FED. ROAD DIST. No.	STATE	FEDERAL PROJ. No.	FISCAL YEAR	SHEET No.	TOTAL SHEETS
	N.D.	FT08(1)		1C	43

9-17-62
Revised Black Upper Right Corner



BRIDGE CODE	FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
X-032	5	N. D.	F-708(1)		2	43

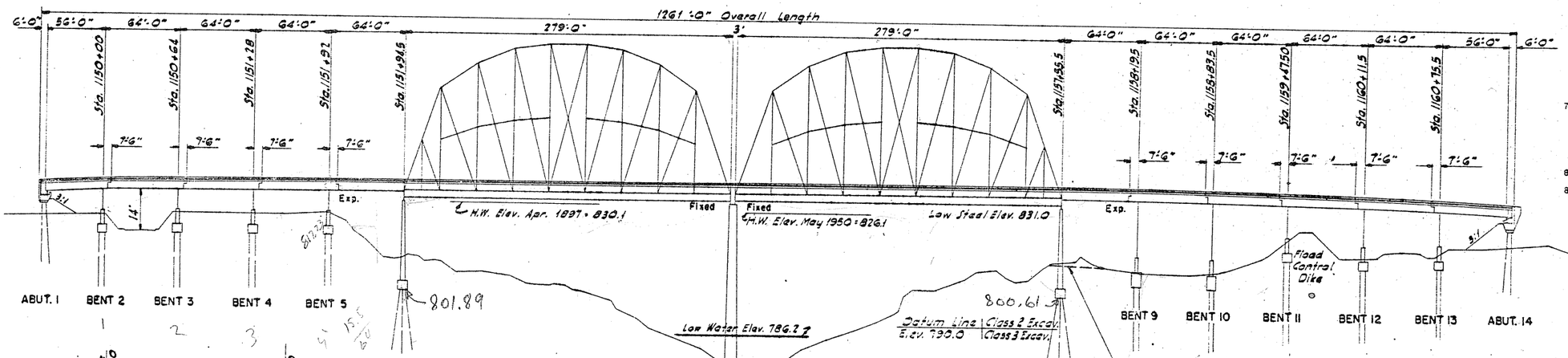


PLAN

A-A
Scale: 1" = 20'

For Construction Notes See Sheet 708(1)-8.

* SPECIAL PROVISIONS
** H20-S16 (1957) AND ALTERNATE LOADING DESIGNATED IN P.P.M. 20-4, SECTION 4C.



ELEVATION

PILE DATA

Type 14BP73

PILE LENGTH	LOCATION
145'	Abutment # 1
140'	Bents # 2 thru # 5
125'	Pier # 6
85'	Pier # 7
120'	Pier # 8
125'	Bents # 9 and # 10
135'	Bent # 11
130'	Bents # 12 and # 13
150'	Abutment # 14

VERTICAL CURVE DATA

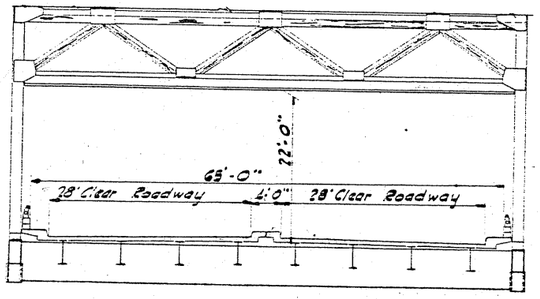
Elevations are to top of finished roadway basics Median Curb
See Sheet 708(1)-22 for Adjusted Profile Grades and Superelevations.

BENCH MARKS

NO.	DESCRIPTION	LOCATION	ELEV.
13	S.W. Corner of Pr. Citizens Top Concrete Step.	Sta. 1148+21 - 41' Lt.	829.39
12	S.E. Corner of loading Platform of Krop Cing Sand Co.	Sta. 1143+47 - 53' Lt.	835.25
11	S.E. Corner in Concrete base of Old Gas Pump Stand owned by Bargain Barn Co.	Sta. 1130+05 - 53' Lt.	830.87

PILE LOADING

LOCATION	DEAD LOAD	LIVE LOAD	EARTH	WIND 50 LB.	WIND 15 LB.	WIND 100 LB. LL.	LONG. FORCE	ICE		DESIGN LOAD	MAXIMUM REQ'D. BEARING
Abut. # 1 & 14	35.78 T.	23.21 T.								59.00 T.	90.0 T.
Bents	48.85 T.	19.53 T.	3.00 T.							[illegible] T.	90.0 T.
Pier # 6 & 13	56.66 T.	8.88 T.	7.66 T.							73.20 T.	90.0 T.
Pier # 7	55.04 T.	4.96 T.	3.89 T.							63.89 T.	90.0 T.



B - B
Scale = 1" = 10'-0"

ESTIMATE OF QUANTITIES

	SPL PROV.	SPEC. NO.	BID ITEM		
		12	REMOVING EXISTING STRUCTURE AT STA.		
		15A	EXCAVATION CLASS 1		CU. YD.
76A	100	15B	" CLASS 2	1200	CU. YD.
	100	15C	" CLASS 3	965	CU. YD.
	106		LINSEED OIL TREATMENT	450	GALS
		60A	CONCRETE CLASS A		CU. YD.
82C	88-A 89	60A	" CLASS AE-2	4001.1	CU. YD.
82C	88-A 89	60B	" CLASS BE-2	162.0	CU. YD.
	90	62A	REINFORCING STEEL (INTERMEDIATE GRADE)	740,415	LB.
*			LIGHTING SYSTEM	1	LUMP SUM
		64A	UNTREATED TIMBER		M.B.M.
		64B	TREATED TIMBER		M.B.M.
		65A	UNTREATED TIMBER PILING — @ — FT.		LIN. FT.
		65B	TREATED TIMBER PILING — @ — FT.		LIN. FT.
	102	65C	STEEL PILING "H" (14BP73)	25,925	LIN. FT.
		65I	UNTREATED TIMBER TEST PILES		EACH
		65J	TREATED TIMBER TEST PILES LONG		EACH
	102	65K	STEEL TEST PILE 135' LONG (14BP73)	2	EACH
*		65T	LOADING TEST - 300 TONS	2	EACH
			BRIDGE BENCH MARKS	1	SET
* 63		63A	STRUCTURAL STEEL	1,485,262	LB.
* 63		63B	STRUCTURAL LOW-ALLOY STEEL	2,327,980	LB.
*			STRUCTURAL QUENCHED & TEMPERED STEEL	70,000	LB.
		81	ORNAMENTAL METAL RAILING	2520.67	LIN. FT.

STRUCTURAL DRAWINGS*

GENERAL DRAWING	708(1) THIS SHEET
DETAIL DRAWINGS	708(1)-1 THRU 708(1)-40
STANDARDS	ST'D. 7.5, H-0401, H-0501

DESIGN LOADING	SCALE
* *	HORIZONTAL 1" = 20 FEET VERTICAL 1" = 5 FEET

NORTH DAKOTA
STATE HIGHWAY DEPARTMENT

SKIDMORE AVENUE
BRIDGE LAYOUT

PROJECT F-708(1) STA. 1154+75.0

GRAND FORKS COUNTY

APPROVED

7/25-62 Joseph R. Kirby
DATE BRIDGE ENGINEER

JOSEPH R. KIRBY REGISTERED PROFESSIONAL ENGINEER NORTH DAKOTA NO.

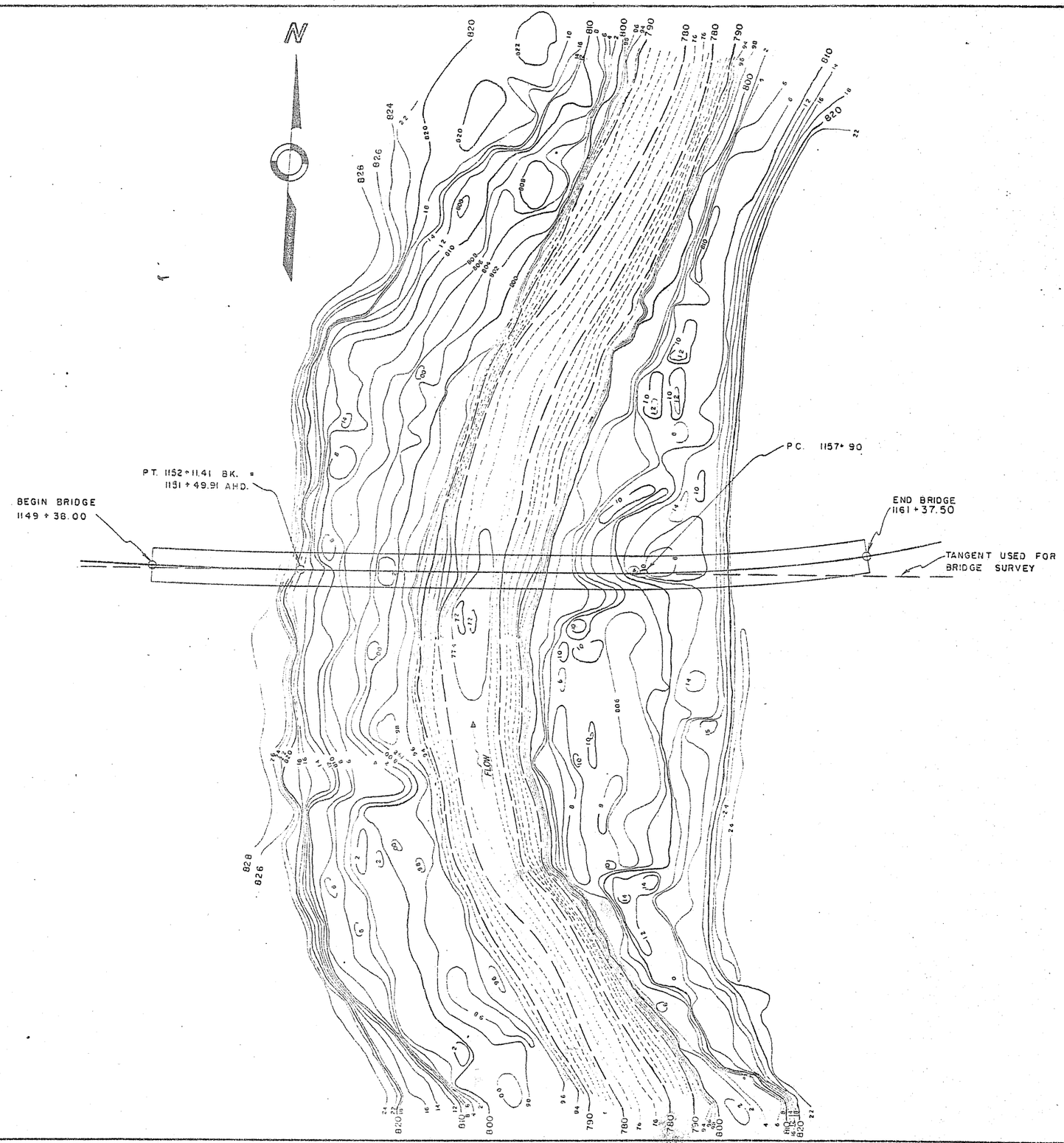
BR. NO. 9090

708(1)

2-33

TRACING | QUANTITIES | CHECKED BY | MADE BY | CHECKED BY | MADE BY | CHECKED BY

FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	F-708(1)		3	43



HYDRAULIC DATA

DRAINAGE AREA	23,100 sq. mi.
DESIGN FREQUENCY	50 - year
MAXIMUM DISCHARGE, 1897	APPROX. 80,000 c.f.s.
ENERGY GRADIENT	0.0000606 ft./ft. (HI. W. PROFILE)
FLOOD STAGE VELOCITY - MAIN CHANNEL	5.40 f.p.s.
TOTAL WATERWAY PROVIDED	APPROX. 29,000 sq. ft.
LOW WATER ELEVATION, MARCH 1960	786.2

BRIDGE LENGTH — 1261'-0"

BRIDGE NO. 2-33

NORTH DAKOTA
STATE HIGHWAY DEPARTMENT

SKIDMORE AVENUE
PROJECT F-708(1)
GRAND FORKS COUNTY

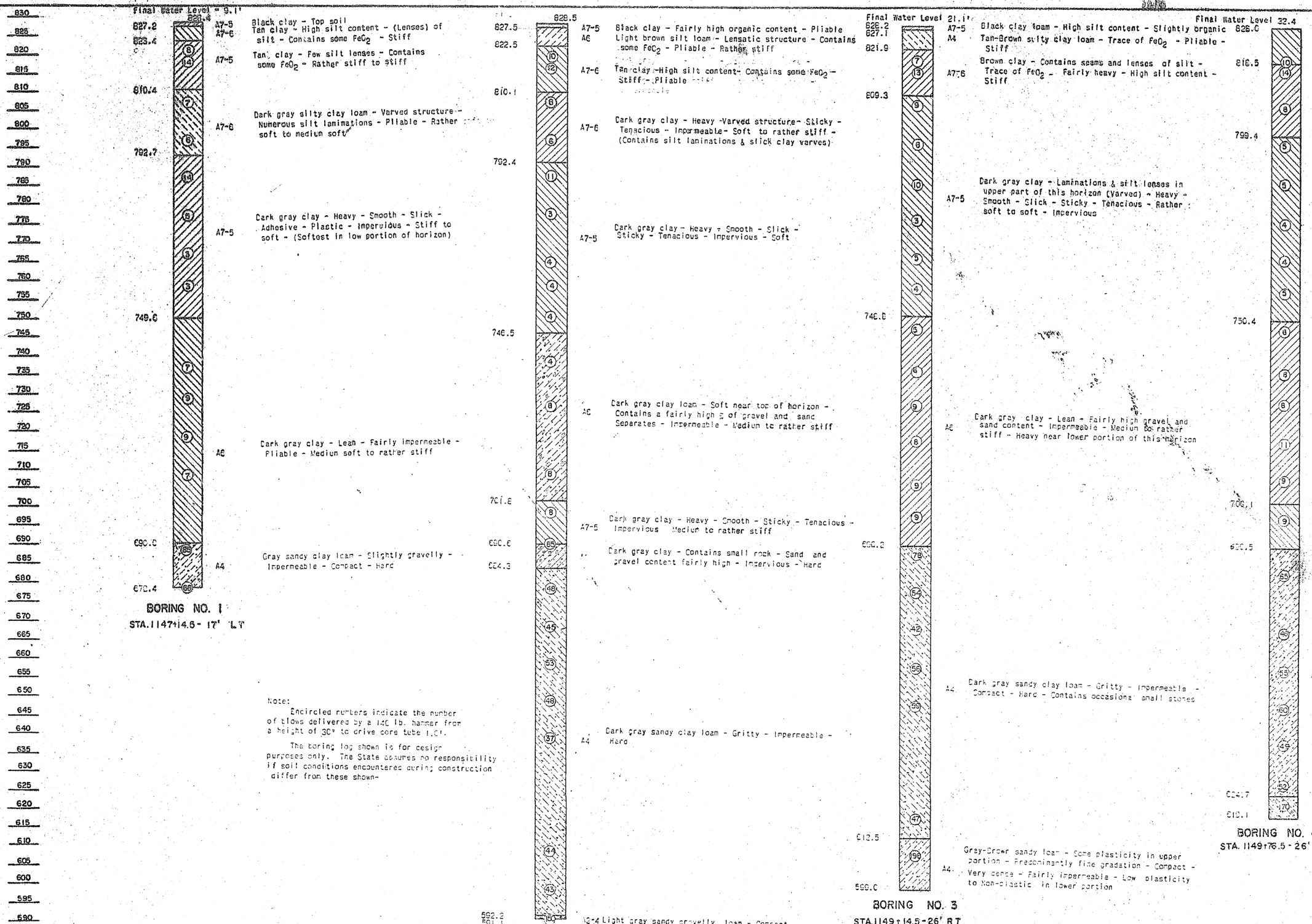
MAIN
(F-708(1))

TRACING — MADE BY — CHECKED BY
QUANTITIES — MADE BY — CHECKED BY

708(1)-1

708(1)-1

FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	F-708(1)		4	43



BORING LOG

F-708(1)

GRAND FORKS COUNTY - ND
POLK COUNTY - MINN.

TRACING
QUANTITIES
NAME BY
CHECKED BY
CHECKED BY
CHECKED BY

708(1)-2

708(1)-2

FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	F-708(1)		5	43

Final Water Level - Not obtained

BORING NO. 5

STA. 1149+94 - 6' RT. of ℄

827.0

Elev.	Soil	Description
826.0	A7-5	Black clay loam - Top soil
821.0	A7-6	Tan silty clay - Contains traces of FeO_2 - Pliable - Rather stiff
813.2	A7-6	Brown clay - Heavy - High silt content - Laminated with silt lenses - Pliable - Medium stiff
804.5	A6	Dark gray silty clay - Varved structure (Contains some slick clay varves) - Pliable - Rather stiff
750.5	A7-5	Dark gray clay - Slightly laminated near top of horizon - Massive structure - Heavy - Smooth - Slick - Adhesive - Tenacious - Impervious - Rather soft to medium stiff (Soft between 40.0 to 60.0')
689.9	A6	Dark gray clay - Lean - Impermeable - Pliable - Rather stiff to stiff
619.9	A2-4	Dark gray sandy loam - Fine gradation - Predominantly with a fairly high % of pebbles and small rock - Variable plasticity - Fairly impermeable - Very dense
606.8	A4	Dark gray sandy loam - Fine gradation - Fairly impermeable - Compact & very dense

BORING NO. 6

STA. 1150+57 - 9' LT.

826.9

Elev.	Soil	Description
825.9	A7-5	Black clay loam - Rather soft - Permeable - Plastic
821.2	A6	Tan silty clay loam - Rather soft - Permeable - Plastic
817.4	A7-6	Tan clay - High silt content - Laminated - Medium stiff - Small iron Oxide content
806.9	A7-6	Gray brown clay - High silt content - Plastic - Medium stiff
795.9	A7-6	Gray clay - Laminated in part & varved structure - Medium stiff - Plastic
751.5	A7-5	Gray clay - Adhesive - Smooth - Tenacious - Plastic - Impervious - Soft to rather soft
690.1	A6	Gray clay - Pockets of silt in upper portion of horizon - Pliable - Slightly permeable - Medium stiff - Fairly high content of fine sand & gravel
592.8	A4	Gray sandy loam - Fine gradation - Very dense
591.5	A4	Gray sandy clay loam - Fine gradation - Very dense & compact - Changes to sandy, gravelly loam

BORING NO. 7

STA. 1151+25 - On ℄

826.8

Elev.	Soil	Description
816.8	A7-6	Brown silty clay - Permeable - Rather soft
810.1	A2-6	Brown clay - Lensatic structure - High silt content in lenses & pockets - Pliable - Medium stiff - Slightly permeable - Small amount of iron Oxide
752.8	A7-6	Dark gray clay - Varved structure from 16.7' to 30.0'
711.8	A7-5	Adhesive - Smooth - Tenacious - Heavy - Plastic - Impervious - Soft to rather soft
690.1	A6	Dark gray clay - Medium to high fine sand content - Lean - Slightly permeable - Medium stiff
672.2	A7-5	Dark gray clay - Adhesive - Smooth - Tenacious - Pliable - Medium stiff to stiff . Impervious
	A4	Dark gray sandy loam - Fine grained sand - Very dense and compact - Fragipan

BORING NO. 8

STA. 1151+90 - 13' RT.

814.3

Elev.	Soil	Description
813.2	A7-5	Black clay loam - Medium stiff - Permeable
811.9	A7-6	Tan silty clay - Pliable - Rather stiff
769.3	A7-5	Gray clay - Lensatic structure - Lenses and pockets of silt - Rather stiff - Fairly impermeable - Heavy - Varved structure in lower 1/3 of horizon - Varves are of various angles
749.3	A7-5	Dark gray clay - Adhesive - Smooth - Tenacious Plastic - Impervious - Soft to rather soft
690.1	A6	Dark gray clay - Fairly high fine sand content - Lean - Rather stiff to stiff - Slightly permeable
644.3	A4	Gray sandy loam - Fine gradation - Compact and very dense - Fragipan

Note:

Encircled numbers indicate the number of blows delivered by a 140 lb. hammer from a height of 30" to drive core tube 1.0'.

The boring log shown is for design purposes only. The State assumes no responsibility if soil conditions encountered during construction differ from these shown.

BRIDGE NO. 2-33

BORING LOG

F-708 (1)

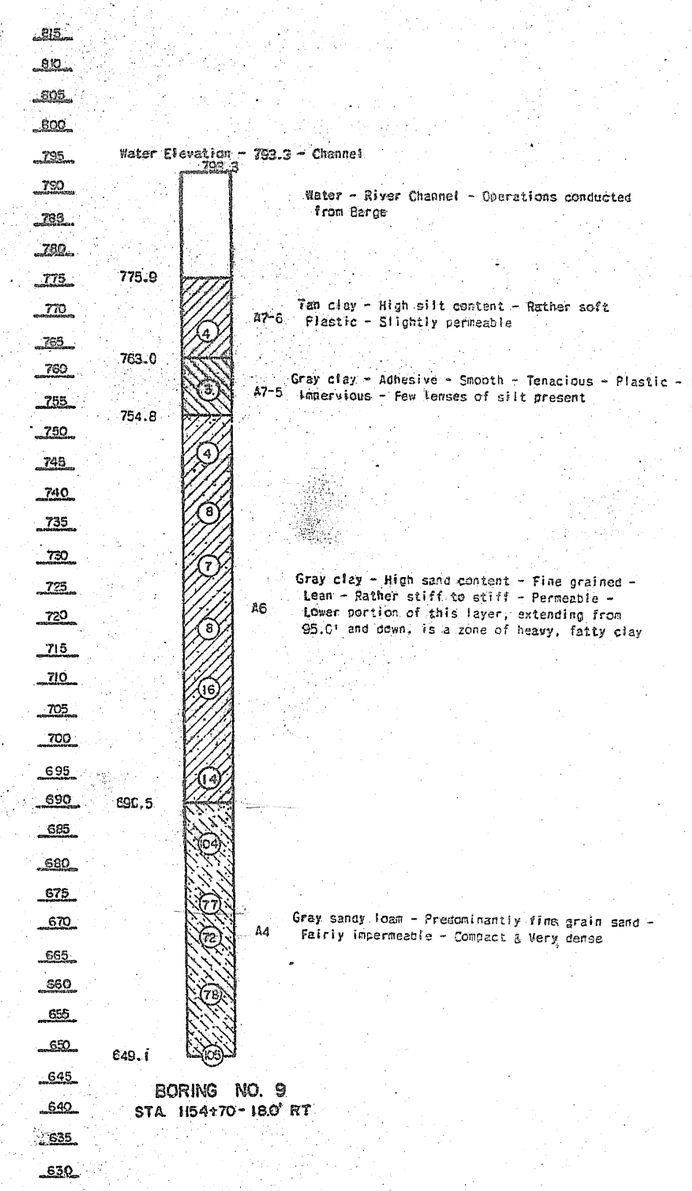
GRAND FORKS COUNTY - N.D.
POLK COUNTY - MINN.

708(1)-3

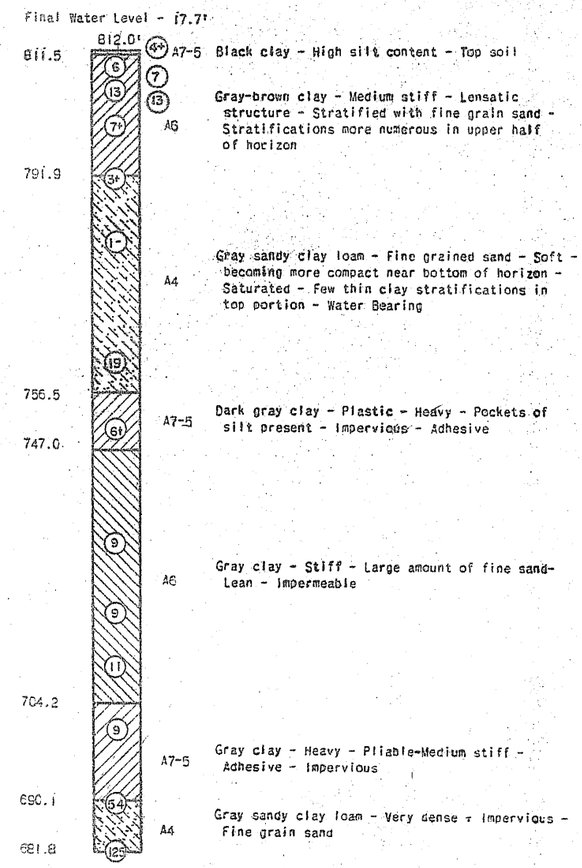
TRACING: INDR BY J.L.S. CHECKED BY
QUANTITIES: MADE BY CHECKED BY

708(1)-3

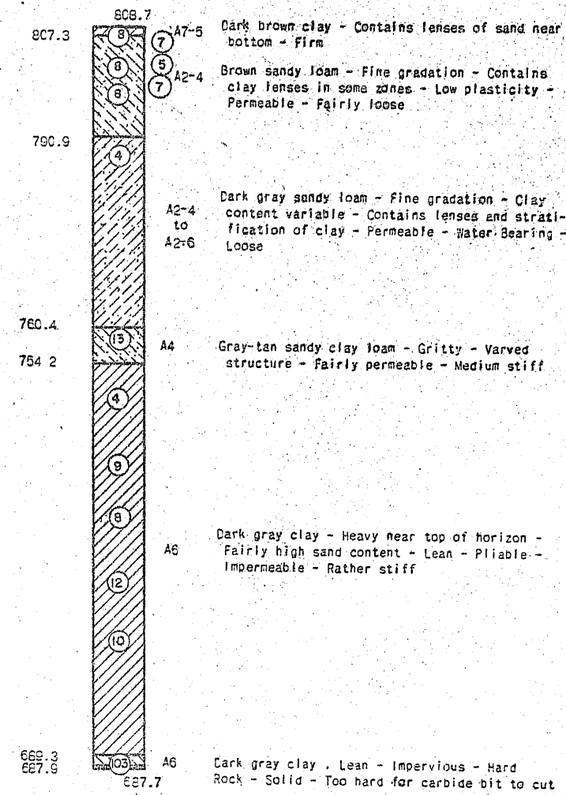
FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	F-708(1)		6	43



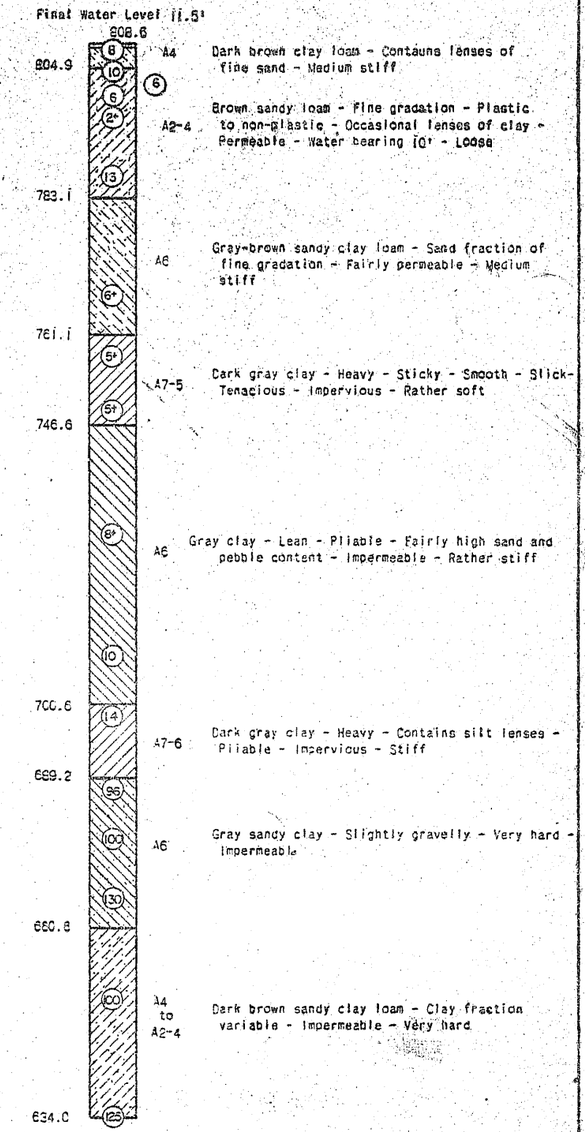
BORING NO. 9
STA. 1154+70 - 18.0' RT



BORING NO. 10
STA. 1157+50 - 4' RT.



BORING NO. 11
STA. 1158+12 - 12' RT. of ℄



BORING NO. 12
STA. 1158+83 - On ℄

Note:

Encircled numbers indicate the number of blows delivered by a 140 lb. hammer from a height of 30" to drive core tube 1.0 feet.

The boring log shown is for design purposes only. The State assumes no responsibility if soil conditions encountered during construction differ from these shown.

BRIDGE NO. 2-33

BORING LOG

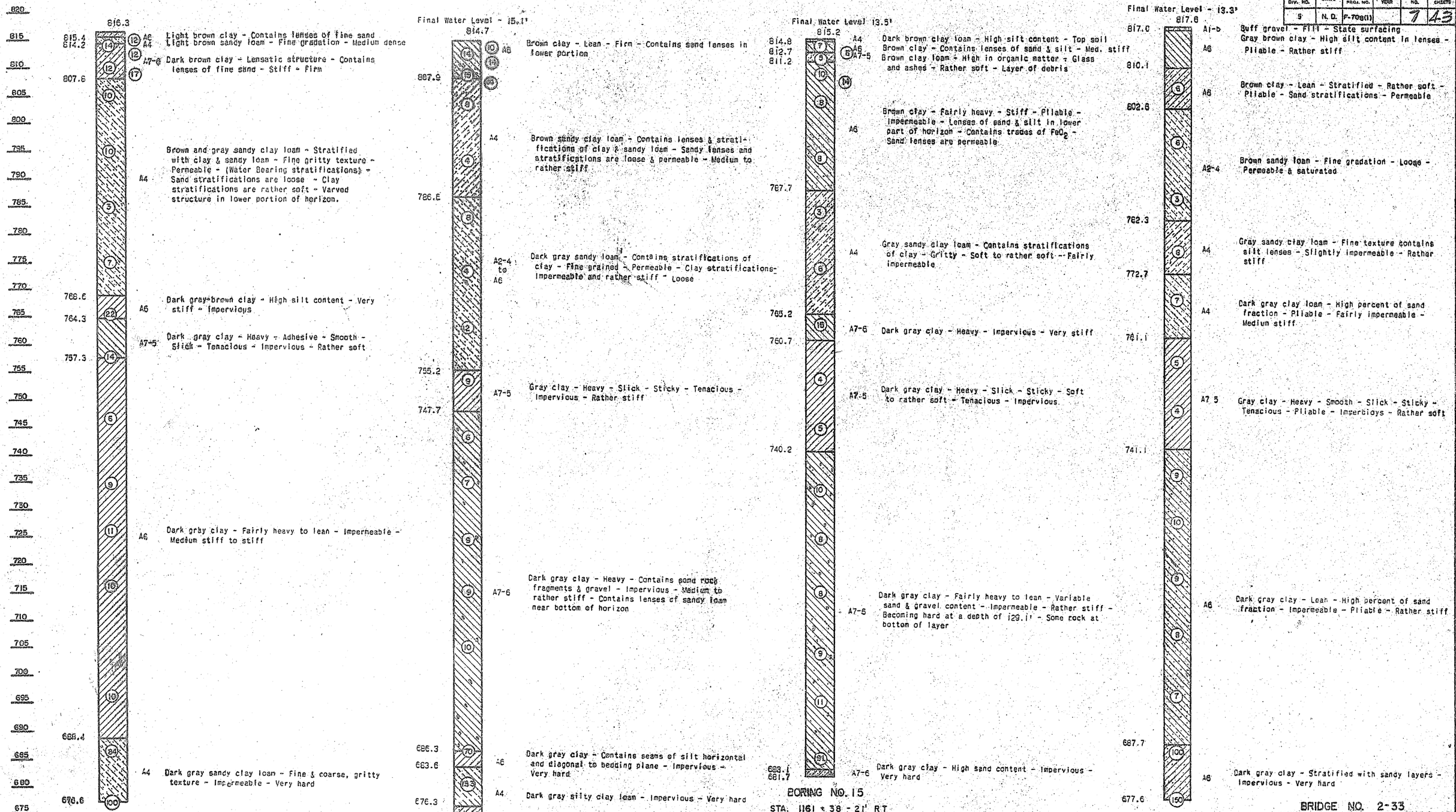
F-708(1)

GRAND FORKS COUNTY - ND
POLK COUNTY - MINN.

708(1)-4

708(1)-4

FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	F-708(1)		7	43



Note:

Encircled numbers indicate the number of blows delivered by a 140 lb. hammer from a height of 30" to drive core tube 1.0'.

The boring log shown is for design purposes only. The State assumes no responsibility if soil conditions encountered during construction differ from these shown.

BRIDGE NO. 2-33

BORING LOG

F-708(1)

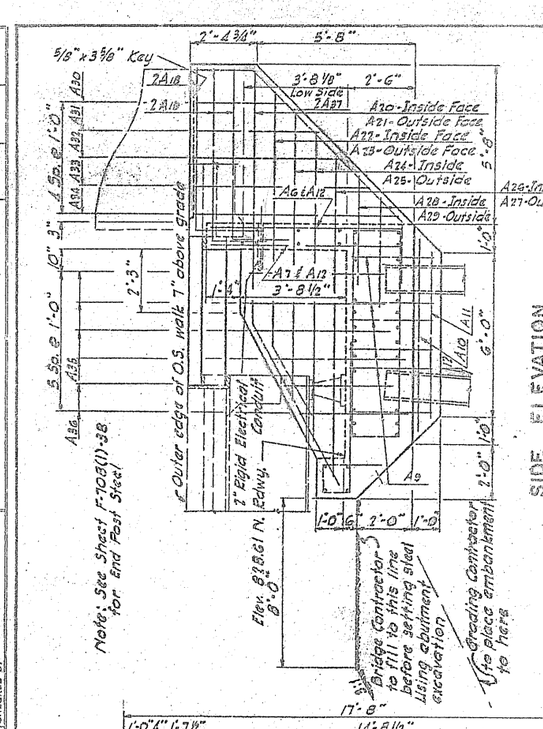
GRAND FORKS COUNTY - ND
POLK COUNTY - MINN.

TRACING CHECKED BY
QUANTITIES MADE BY CHECKED BY

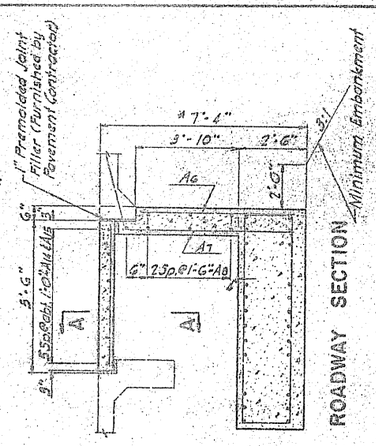
708(1)-5

708(1) 5

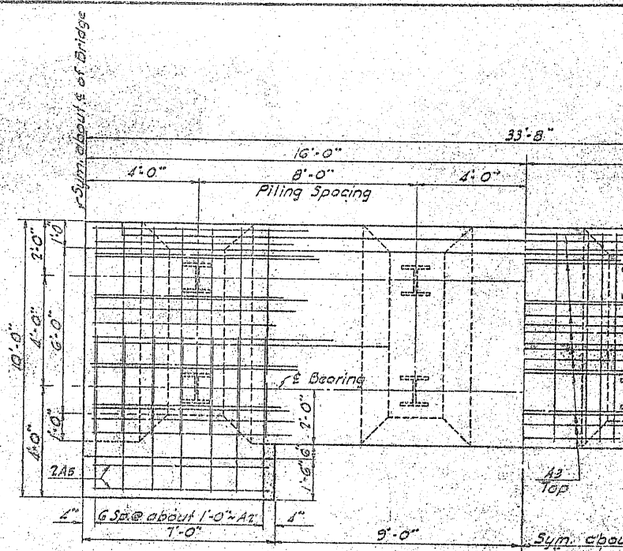
FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(1)		8	43



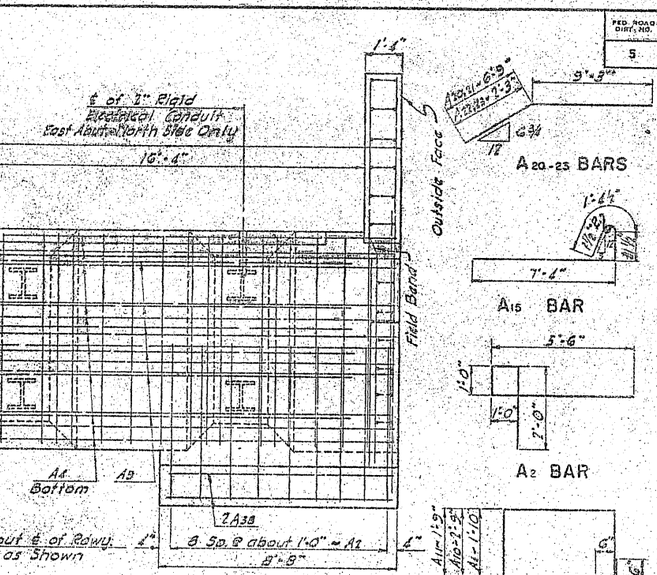
SIDE ELEVATION



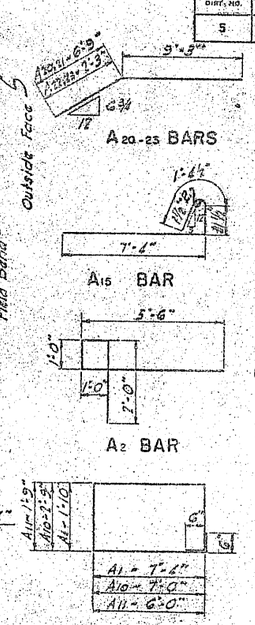
℄ ROADWAY SECTION



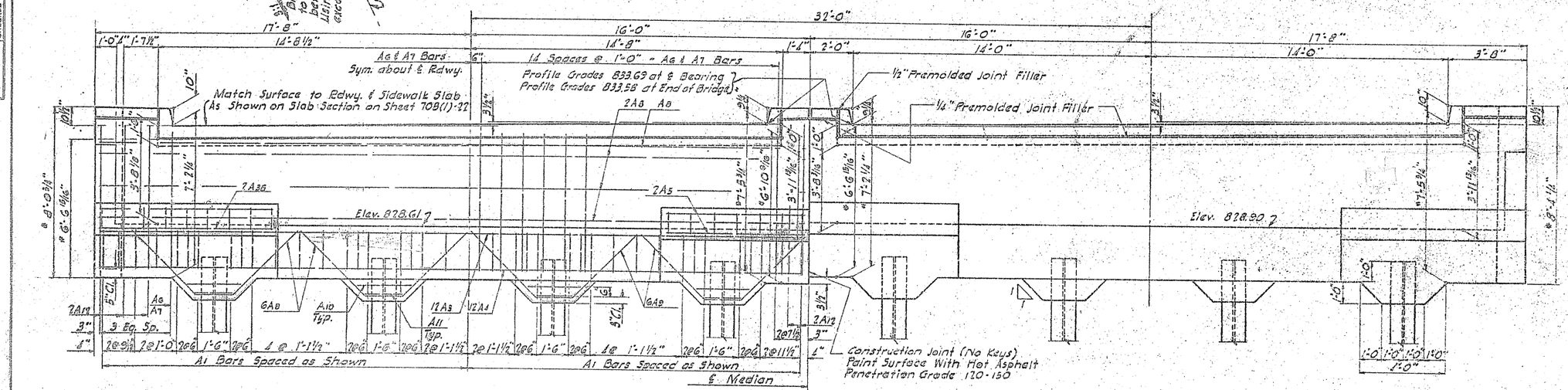
QUARTER PLAN
Showing Dimensions



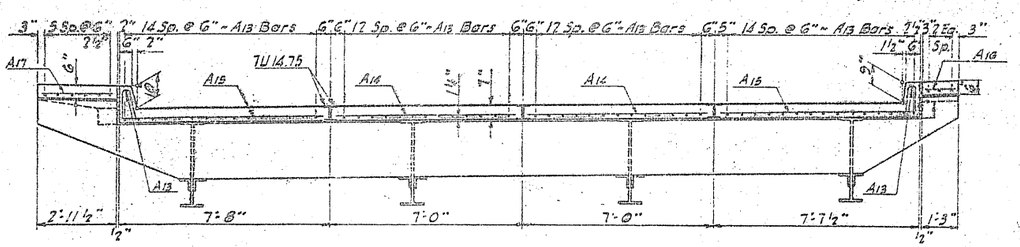
QUARTER PLAN
Showing Reinforcing



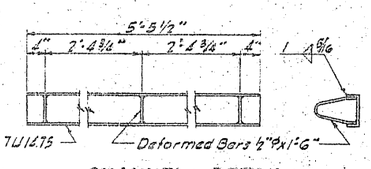
BAR LIST				
MARK	NO.	SIZE	LENGTH	SHAPE
A1	18	7	19'-2"	Bent
A2	32	5	9'-6"	"
A3	24	8	33'-3"	Str.
A4	24	9	33'-3"	"
A5	9	4	6'-6"	"
A6	60	5	5'-6"	"
A7	60	5	5'-6"	Bent
A8	14	4	33'-3"	Str.
A9	48	4	10'-3"	Bent
A10	8	4	20'-6"	"
A11	8	4	16'-6"	"
A12	20	5	7'-3"	Str.
A13	134	5	5'-2"	"
A14	24	5	6'-9"	"
A15	24	5	9'-8"	Bent
A16	12	4	1'-0"	Str.
A17	12	4	2'-9"	"
A18	4	6	8'-3"	Bent
A19	8	5	8'-3"	"
A20	2	5	16'-6"	"
A21	2	4	16'-0"	"
A22	2	5	17'-6"	"
A23	2	4	17'-6"	"
A24	4	5	10'-6"	Str.
A25	4	4	10'-6"	"
A26	2	5	11'-9"	"
A27	2	4	11'-9"	"
A28	2	5	4'-0"	"
A29	2	4	4'-0"	"
A30	2	4	7'-6"	Bent
A31	2	4	9'-6"	"
A32	2	4	11'-6"	"
A33	2	4	13'-6"	"
A34	2	4	15'-6"	"
A35	8	4	10'-6"	"
A36	4	4	8'-6"	"
A37	4	4	12'-0"	"
A38	8	4	8'-3"	Str.



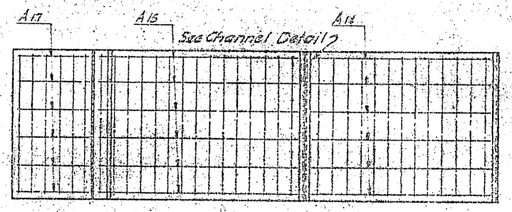
ELEVATION
All Steel Sym. about ℄ of Median & Dimensions as Shown



SECTION A-A
Showing Removeable Slab



CHANNEL DETAIL



HALF PLAN ~ REMOVABLE SLAB

* Dimensions are at end of bridge.

QUANTITIES	
Concrete Class A-3	848 C.Y.
Reinforcing Steel	11,944 Lbs.
Structural Steel	1,000 Lbs.
Piling (See Layout)	

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.- E. GRAND FORKS, MINN.

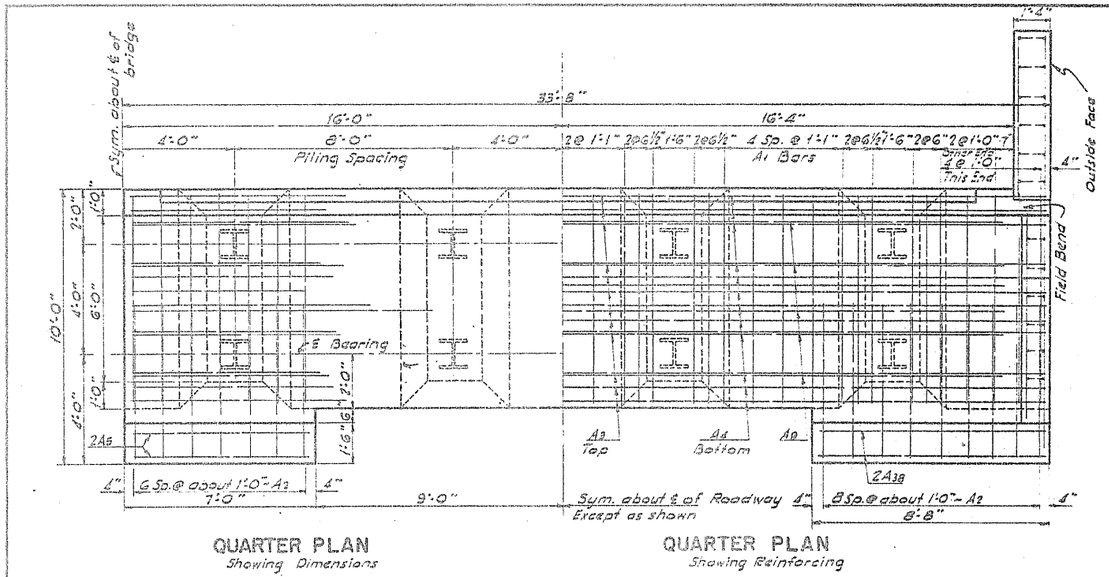
EAST ABUTMENT

CHECKED BY
MADE BY
CHECKED BY
QUANTITIES

708(1)-6

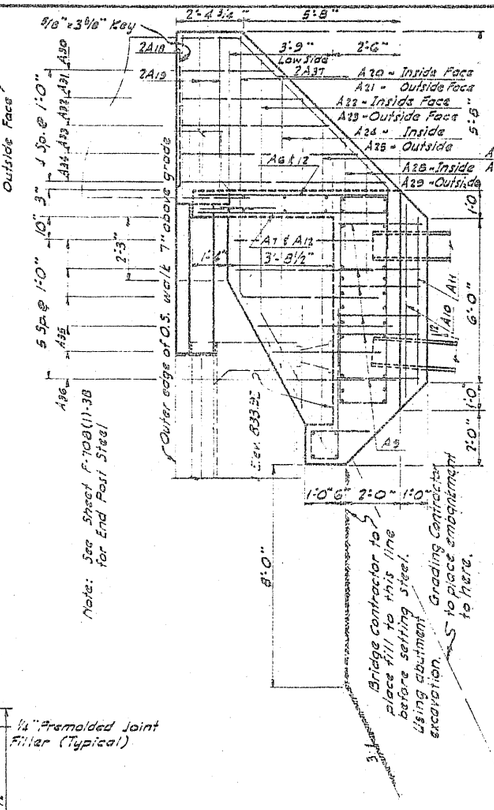
708(1)-6

FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(1)		9	43

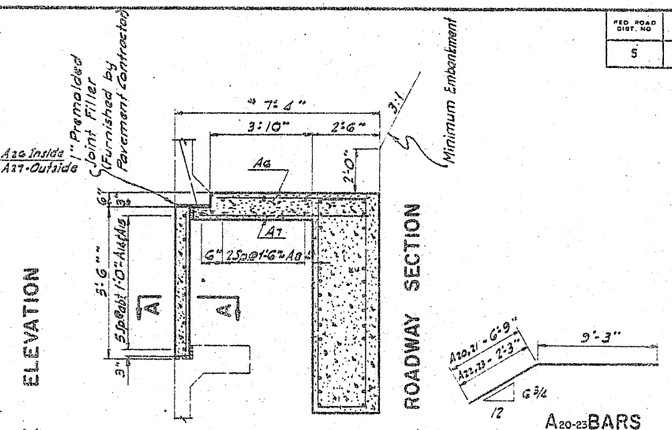


QUARTER PLAN
Showing Dimensions

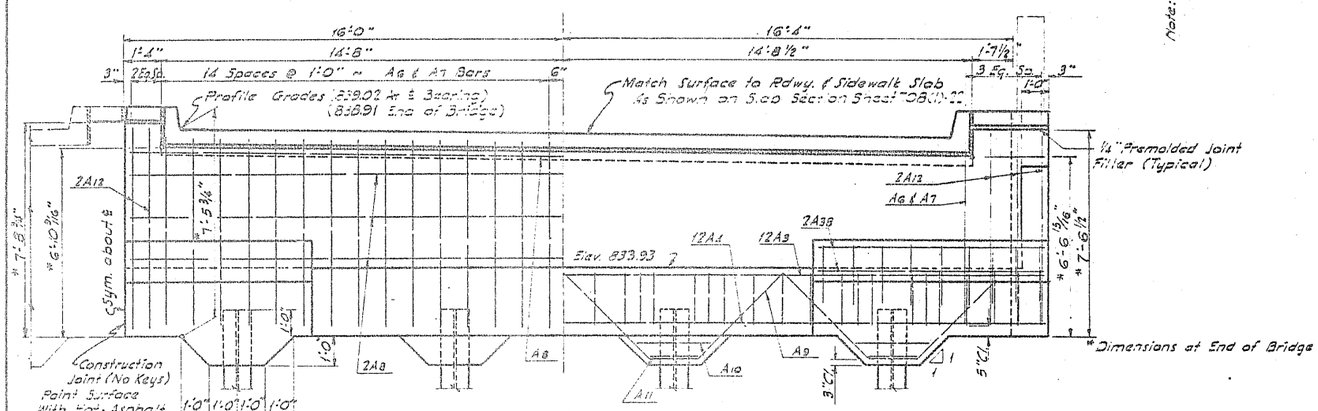
QUARTER PLAN
Showing Reinforcing



SIDE ELEVATION

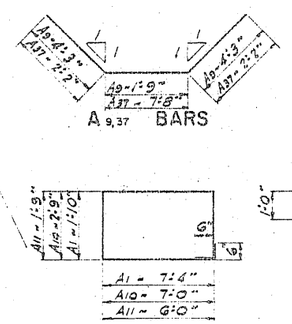


ROADWAY SECTION



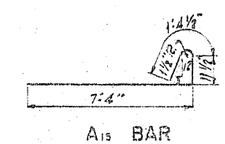
QUARTER ELEVATION
Quarter Showing Wall Steel

QUARTER ELEVATION
Quarter Showing Footing Steel

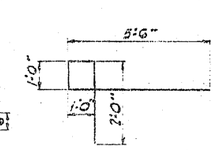


A 9,37 BARS

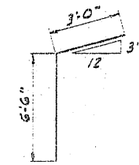
A1, A10 & A11 BARS



A15 BAR

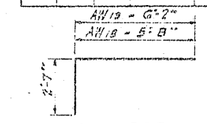


A2 BAR

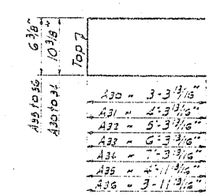


A7 BAR

BENT BAR DETAILS

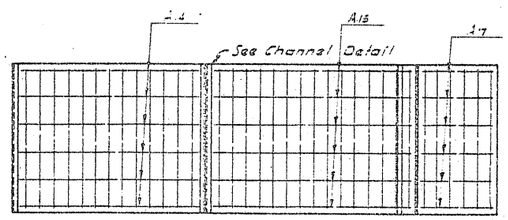


A16,18 BAR

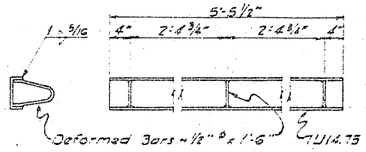


A30-36 BARS

SECTION A-A
Showing Removable Slab



HALF PLAN - REMOVABLE SLAB



CHANNEL DETAIL

BAR LIST

MARK	NO.	SIZE	LENGTH	SHAPE
A1	78	7	19'-4"	Bent
A2	32	5	9'-6"	"
A3	24	8	33'-3"	Str.
A4	24	9	33'-3"	"
A5	8	4	6'-6"	"
A6	60	5	5'-6"	"
A7	60	5	9'-6"	Bent
A8	14	4	33'-3"	Str.
A9	48	4	10'-3"	Bent
A10	8	4	20'-6"	"
A11	8	4	16'-6"	"
A12	20	5	7'-3"	Str.
A13	134	5	5'-2"	"
A14	24	5	6'-9"	"
A15	24	5	9'-8"	Bent
A16	12	4	1'-0"	Str.
A17	12	4	2'-9"	"
A18	4	6	8'-3"	Bent
A19	8	5	8'-9"	"
A20	2	5	16'-0"	"
A21	2	4	16'-0"	"
A22	2	5	11'-6"	"
A23	2	4	11'-6"	"
A24	2	5	10'-6"	Str.
A25	2	4	10'-6"	"
A26	2	5	11'-9"	"
A27	2	4	11'-9"	"
A28	2	5	4'-0"	"
A29	2	4	4'-0"	"
A30	2	5	7'-6"	Bent
A31	2	4	9'-6"	"
A32	2	4	11'-6"	"
A33	2	4	3'-6"	"
A34	2	4	5'-6"	"
A35	8	4	10'-6"	"
A36	4	4	8'-6"	"
A37	2	4	12'-0"	"
A38	8	4	8'-3"	Str.

QUANTITIES

Concrete Class AE-2	84.3	C.Y.
Reinforcing Steel	11,944	Lbs.
Structural Steel	1,000	Lbs.
Piling (See Layout)		

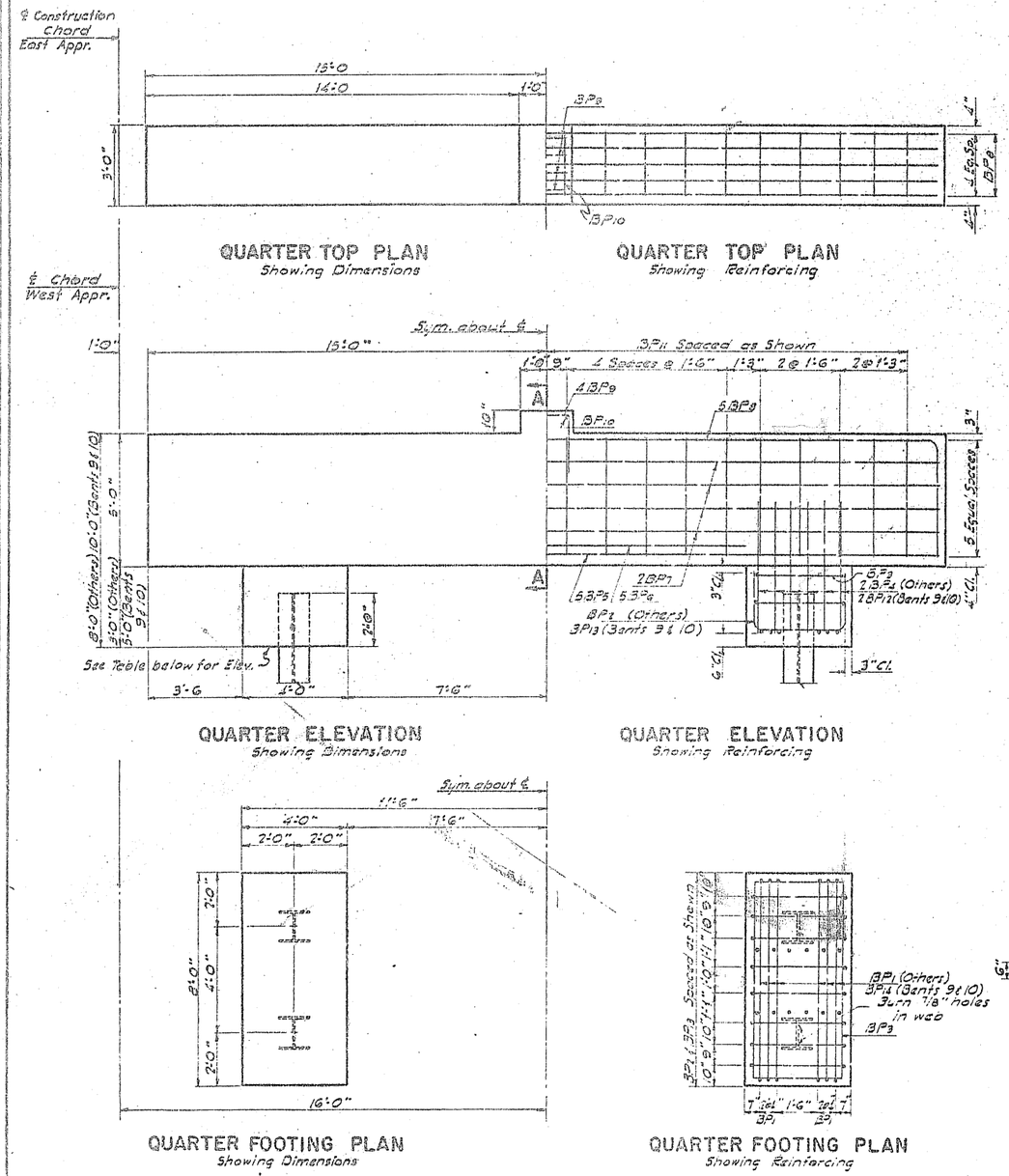
RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

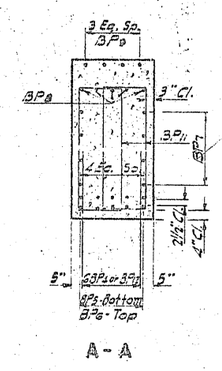
WEST ABUTMENT

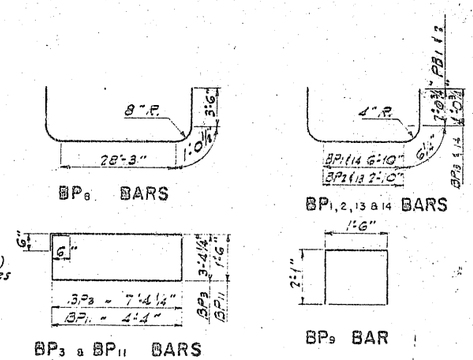
TRACING
QUANTITIES
CHECKED BY
MADE BY
CHECKED BY

708(1)-7

708(1)-7







BAR LIST FOR 18 BENT FOOTINGS

MARK	NO	SIZE	LENGTH	SHAPE
BP1	168	7	12'-0"	Bent
BP2	224	5	8'-0"	"
BP3	72	4	22'-5"	"
BP4	336	6	5'-0"	Str.
BP5	90	8	29'-6"	"
BP6	90	8	15'-0"	"
BP7	144	4	29'-6"	"
BP8	90	9	37'-4"	Bent
BP9	72	5	5'-8"	"
BP10	36	4	2'-8"	Str.
BP11	720	4	12'-8"	Bent
BP12	96	6	7'-0"	Str.
BP13	64	5	12'-0"	Bent
BP14	48	7	16'-0"	"

NOTES:

STRUCTURAL STEEL

Unless designated as Low Alloy (L.A.), Structural Quenched and Tempered Steel (Q & T) or A36 Steel material shall be Structural Carbon Steel which conforms to the latest ASTM A-7 Specifications. Low alloy steel shall conform to ASTM A441 Specifications. Cast steel shall conform to AASHO M-192 Class 70-36. Open holes shall be 15/16" ⌀ except as noted.

Field and shop rivets may be interchanged wherever the contractor so desires.

High tensile strength bolts may replace 7/8" field rivets.

All holes for field connections of main truss members shall be sub-punched or sub-drilled, and reamed with parts assembled and members in correct relative position. Assembled parts shall be taken apart for removal of burrs caused by drilling or reaming.

Truss shop assemblies shall consist of units of at least three panels with one panel overlapping the next assembly.

The Contractor shall submit his method of erection to the Engineer for approval (in writing) before shop drawings are prepared.

Paint and painting shall conform to the North Dakota Standard Specifications for Road and Bridge Construction, sections 80, 132.11 & 132.17. All exposed steel surfaces shall be given one shop coat of red lead paint, one spot coat of red lead paint after erection and concrete work is completed and two field coats of aluminum paint (first field coat tinted with prussian blue). Non-exposed surfaces of floor beams, stringers, expansion joints and deck joints shall receive the regular shop coat, field spot coat and, in addition, one full field coat of red lead, tinted.

Welding will not be paid for directly but shall be included in the unit price bid for structural steel.

The metal-arc process shall be used for all shop and field welding. All welding shall conform to the current "Standard Specifications for Welded Highway and Railway Bridges-Design, Construction and Repair" of the American Welding Society. Only low hydrogen electrodes shall be used.

All rivets shall be 7/8" ⌀.

A36 steel shall conform to ASTM-A36 specifications and shall be paid for as Structural Steel.

Weld inspection will be required where necessary in accordance with the special provisions.

GENERAL:

Reinforcing steel bar dimensions for bent bars are given center to center unless noted.

The fabricator shall add a prefix to each bar designation to differentiate between structures and/or units of a structure on the project.

The cost of incidental items shown on the plans but not listed in the estimate of quantities shall be included in the unit price bid for the various pay items.

A single or double-acting steam, diesel, or air hammer will be required for all piling for this project. The hammer shall develop not less than 22,000 foot pounds of energy per blow. Minimum weight of ram shall be 4800 pounds.

The concrete railing components, curbs, outside of slab and exposed wing surfaces shall be given the "Rubbed Surface Finish".

The cost of clearing and grubbing shall be included in the unit price bid for other items.

No piling shall be driven in the abutment locations until the embankment at each end of the bridge has been placed up to the berm elevations.

DESIGN STRESSES:

Concrete	1,200 #/sq. in.
Carbon Steel	18,000 #/sq. in.
Low Alloy Steel	
3/4" and under	27,000 #/sq. in.
3/4" to 1 1/2"	24,000 #/sq. in.
1 1/2" to 4"	22,000 #/sq. in.
Q. & T. Steel	55,000 #/sq. in.
Reinforcing Steel	20,000 #/sq. in.

FOOTING	ELEVATION
Bent #2S	821.00
Bent #2N	821.01
" #4	820.[illegible]
" #5	820.73
" #8N	820.87
" #9S	805.17
" #10N	804.37
" #10S	804.66
" #11N	818.70
" #11S	818.99
" #12N	818.81
" #12S	811.20
" #13N	811.95
" #13S	811.26
" #14N	811.00
" #14S	811.03

For Staking Details See Sheets 708(1)-10 & 708(1)-11

Fast & Wash Approach Spans

QUANTITIES FOR ALL BENT FOOTINGS

Concrete Class AE-2	4503 C.Y.
Reinforcing Steel	41,055 Lbs.

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

BENT FOOTING

TRACING CHECKED BY
QUANTITIES MADE BY CHECKED BY

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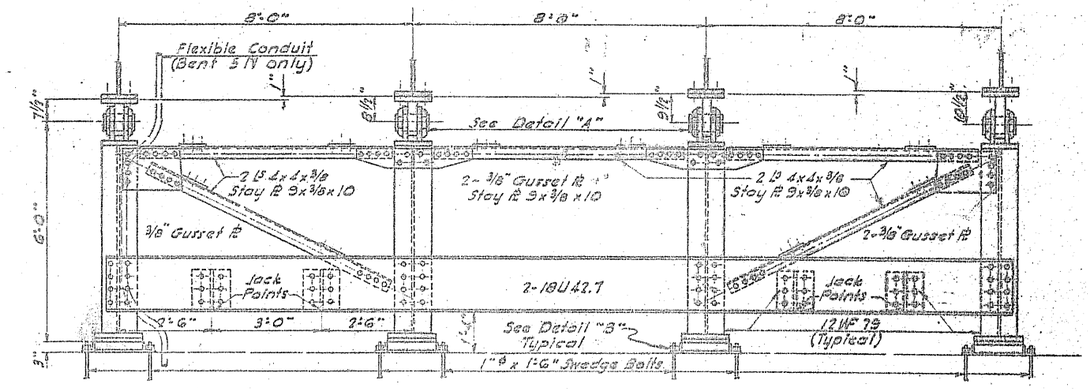
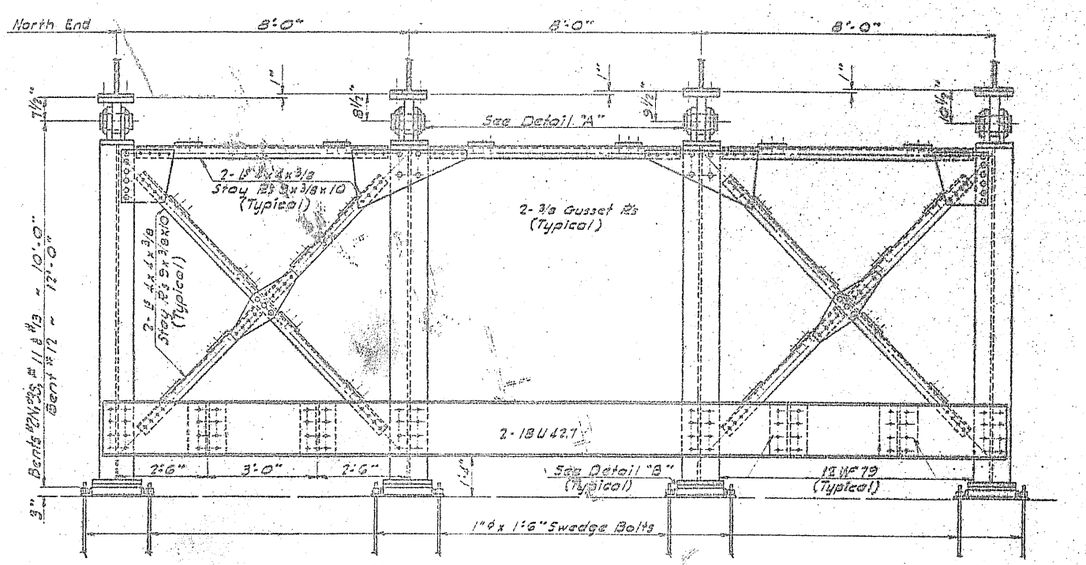
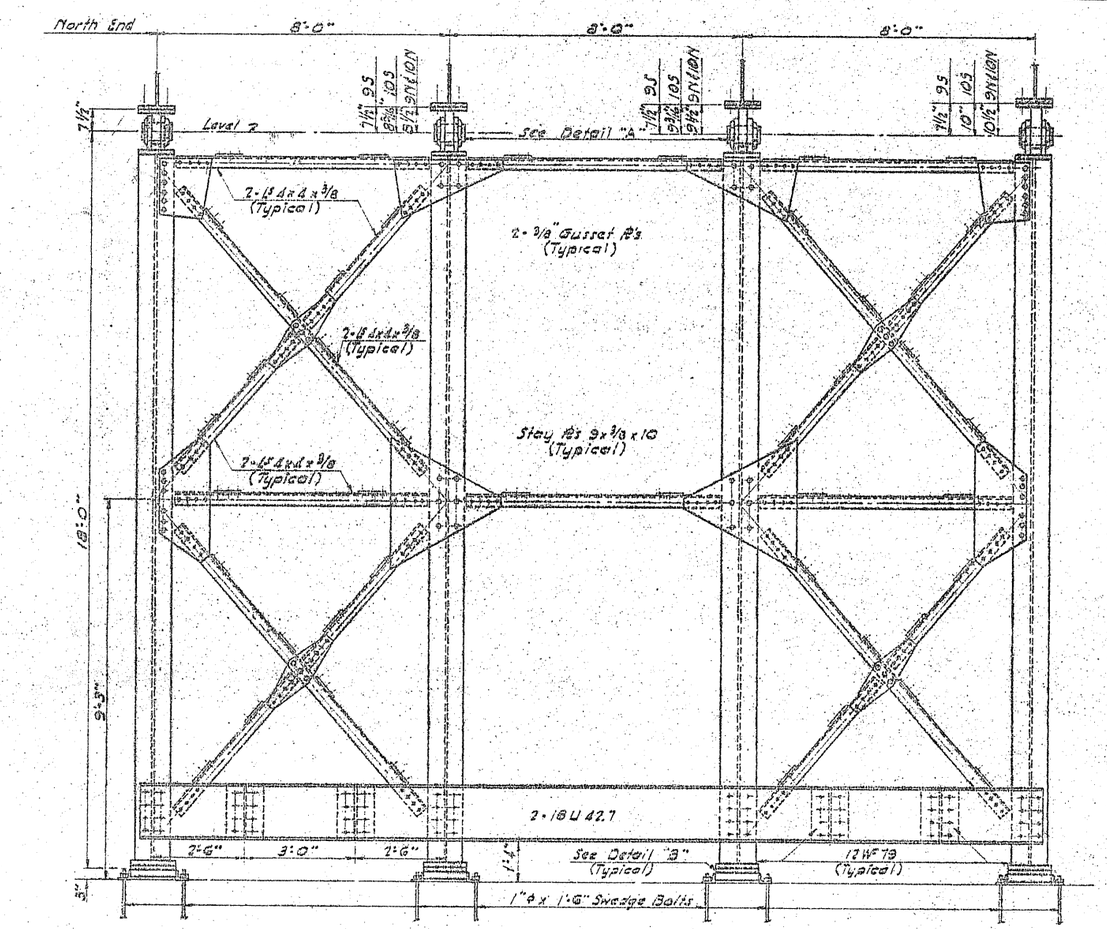
FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(1)		11	43



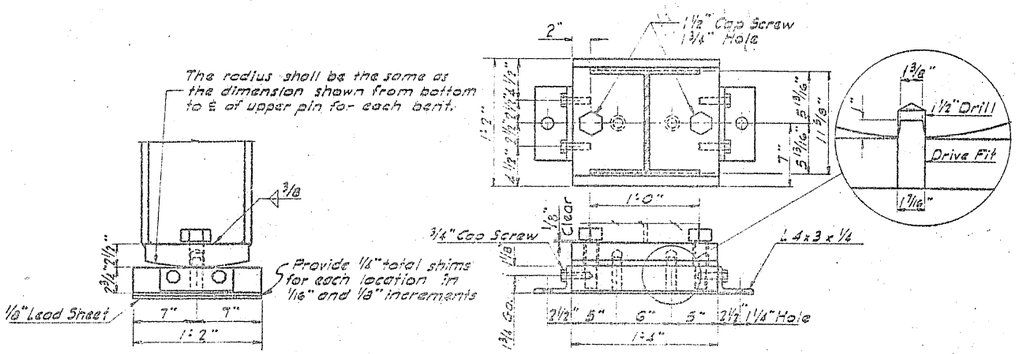
BENTS 2S, 3N, 4N, 4S, 5N & 5S



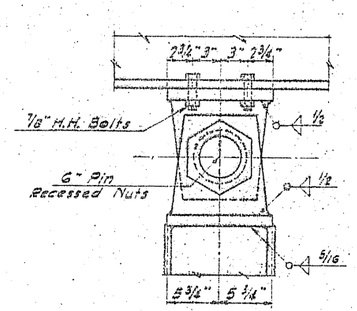
BENTS 2N, 3S, 11N, 11S, 12N, 12S, 13N & 13S

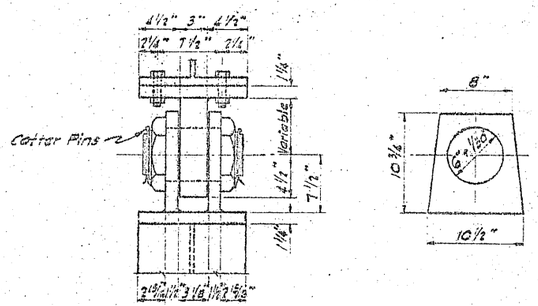


BENTS 9N, 9S, 10N & 10S



DETAIL "B"





DETAIL "A"

These bents have been designed to withstand jacking stresses of dead load only, using four jacks located as shown. All contact surfaces in detail "A" and "B" shall be milled before the parts are welded together.
The upper half of the bearing in Detail "A" shall be stress-relieved after welding in accordance with the A.W.S. Specification 506(b).

QUANTITIES FOR ALL BENTS	
Structural Steel	190,000 Lbs.

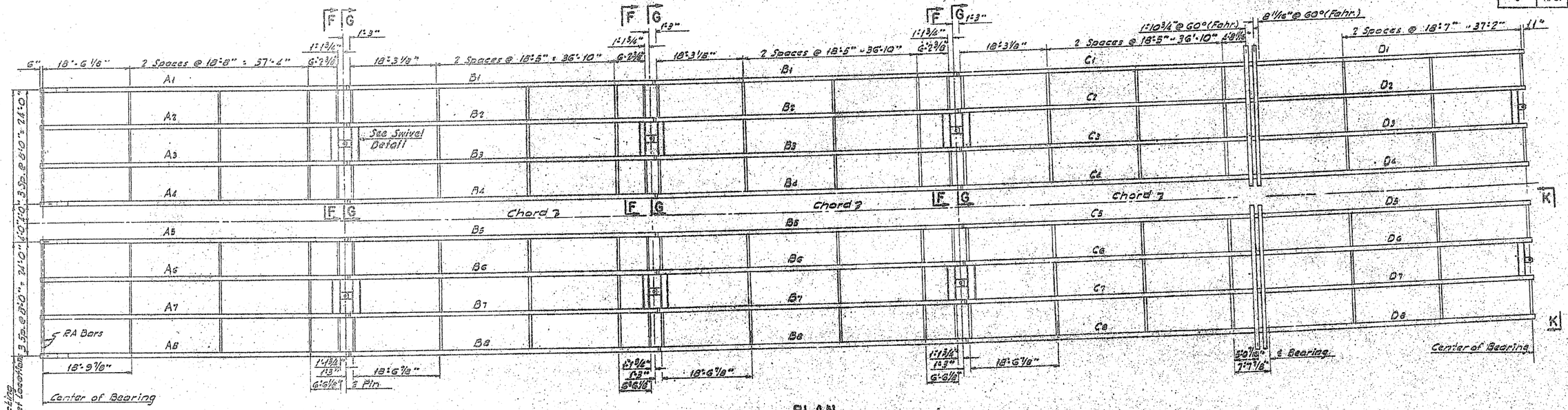
RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

BENT DETAILS

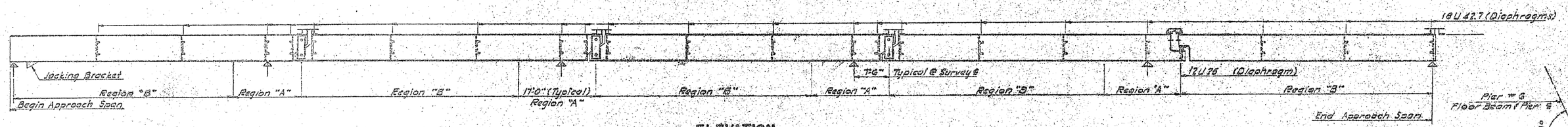
TRACING	
INKED BY	
CHECKED BY	
QUANTITIES	
MADE BY	
CHECKED BY	

708(1)-9

FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	F-7081		12	43



PLAN



ELEVATION

Beam No.	Center of bearing to center of pin
A1	63'-2¼"
A2	63'-3 5/16"
A3	63'-4 3/16"
A4	63'-6 7/16"
A5	63'-8 9/16"
A6	63'-7 3/8"
A7	63'-8 11/16"
A8	63'-9 3/4"

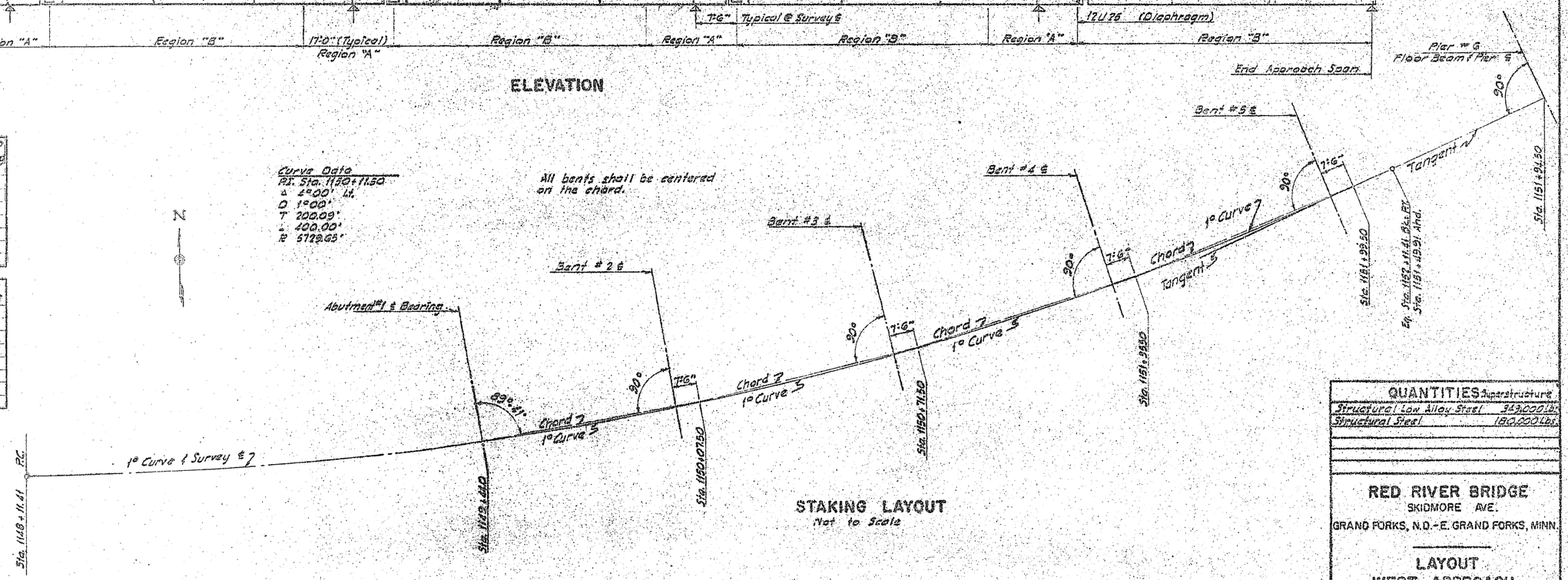
Beam No.	Center of pin to center of bearing
C1	63'-8 1/4"
C2	63'-9 9/16"
C3	63'-10 3/8"
C4	63'-11 7/16"
C5	64'-0 9/16"
C6	64'-1 5/8"
C7	64'-2 11/16"
C8	64'-3 3/4"

Beam No.	Center of pin to center of pin
B1	63'-8 1/4"
B2	63'-9 9/16"
B3	63'-10 3/8"
B4	63'-11 7/16"
B5	64'-0 9/16"
B6	64'-1 5/8"
B7	64'-2 11/16"
B8	64'-3 3/4"

Beam No.	Center of bearing to center of bearing
D1	55'-5 3/16"
D2	55'-5 1/2"
D3	55'-5 11/16"
D4	55'-5 7/8"
D5	55'-6 1/8"
D6	55'-6 5/16"
D7	55'-6 1/2"
D8	55'-6 11/16"

Curve Data
P.I. Sta. 1130+11.50
Δ 4°00' Lt.
D 1°00'
T 200.09'
L 400.00'
R 5729.65'

All bents shall be centered on the chord.



STAKING LAYOUT
Not to Scale

QUANTITIES Superstructure	
Structural Low Alloy Steel	343,000 Lb.
Structural Steel	180,000 Lb.

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

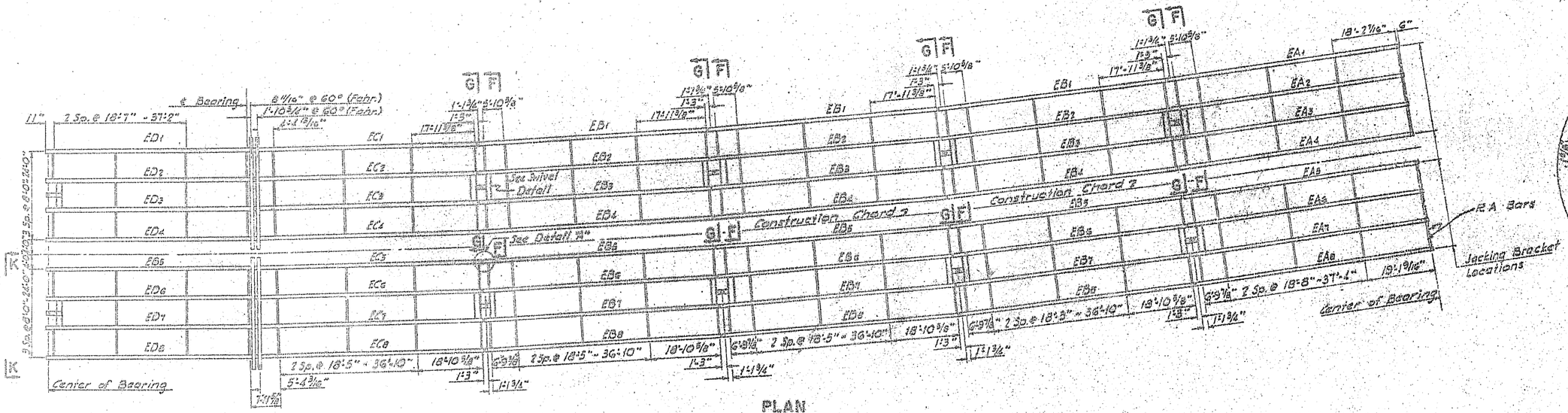
LAYOUT
WEST APPROACH

708(i)-10

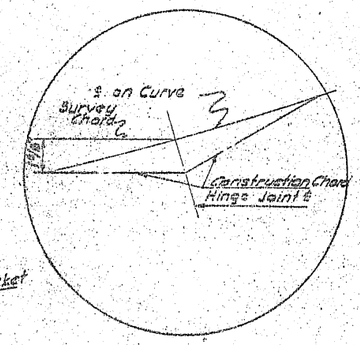
TRACINGS CHECKED BY
QUANTITIES MADE BY CHECKED BY

708(i)-10

FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEARS	SHEET NO.	TOTAL SHEETS
5	N. D.	P-7080		18	43

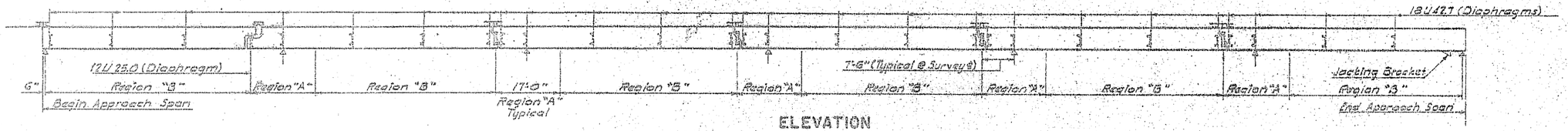


PLAN



DETAIL "A"

Typical, except @ Sta. 1157+55.6 where centerlines are coincident



ELEVATION

Beam No.	Center of bearing to center of pin
EA1	62'-6 13/16"
EA2	62'-10"
EA3	63'-1 3/16"
EA4	63'-4 3/8"
EA5	63'-7 5/8"
EA6	63'-10 13/16"
EA7	64'-2"
EA8	64'-5 3/16"

Beam No.	Center of pin to center of bearing
EC1	63'-0 3/4"
EC2	63'-3 15/16"
EC3	63'-7 3/16"
EC4	63'-10 3/8"
EC5	64'-1 3/8"
EC6	64'-4 12/16"
EC7	64'-8 1/16"
EC8	64'-11 1/2"

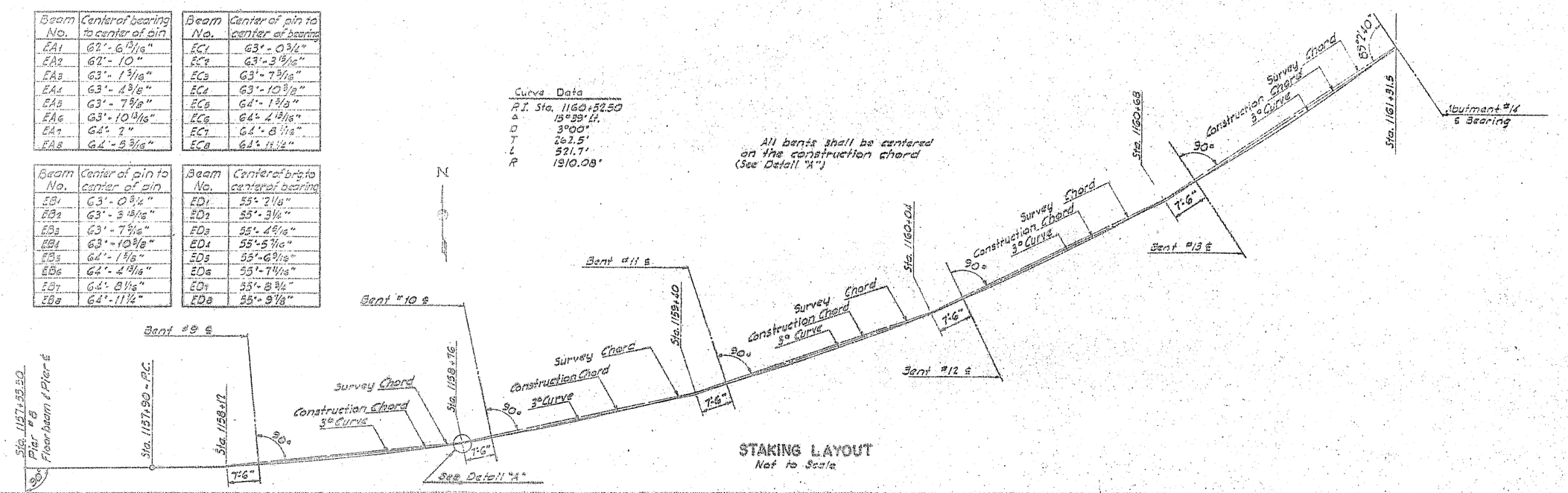
Beam No.	Center of pin to center of pin
EB1	63'-0 3/4"
EB2	63'-3 15/16"
EB3	63'-7 3/16"
EB4	63'-10 3/8"
EB5	64'-1 3/8"
EB6	64'-4 13/16"
EB7	64'-8 1/16"
EB8	64'-11 1/2"

Beam No.	Center of brg. to center of bearing
ED1	55'-2 1/8"
ED2	55'-3 1/16"
ED3	55'-4 5/16"
ED4	55'-5 7/16"
ED5	55'-6 9/16"
ED6	55'-7 11/16"
ED7	55'-8 3/4"
ED8	55'-9 7/8"

Curve Data

P.I. Sta. 1160+52.50
Δ 15°39' Lt.
D 3°00'
T 262.5'
L 521.7'
R 1910.08'

All bents shall be centered on the construction chord (See Detail "A")



STAKING LAYOUT

Not to Scale

QUANTITIES	
Structural Low Alloy Steel	214,000 LBS
Structural Steel	216,000 LBS

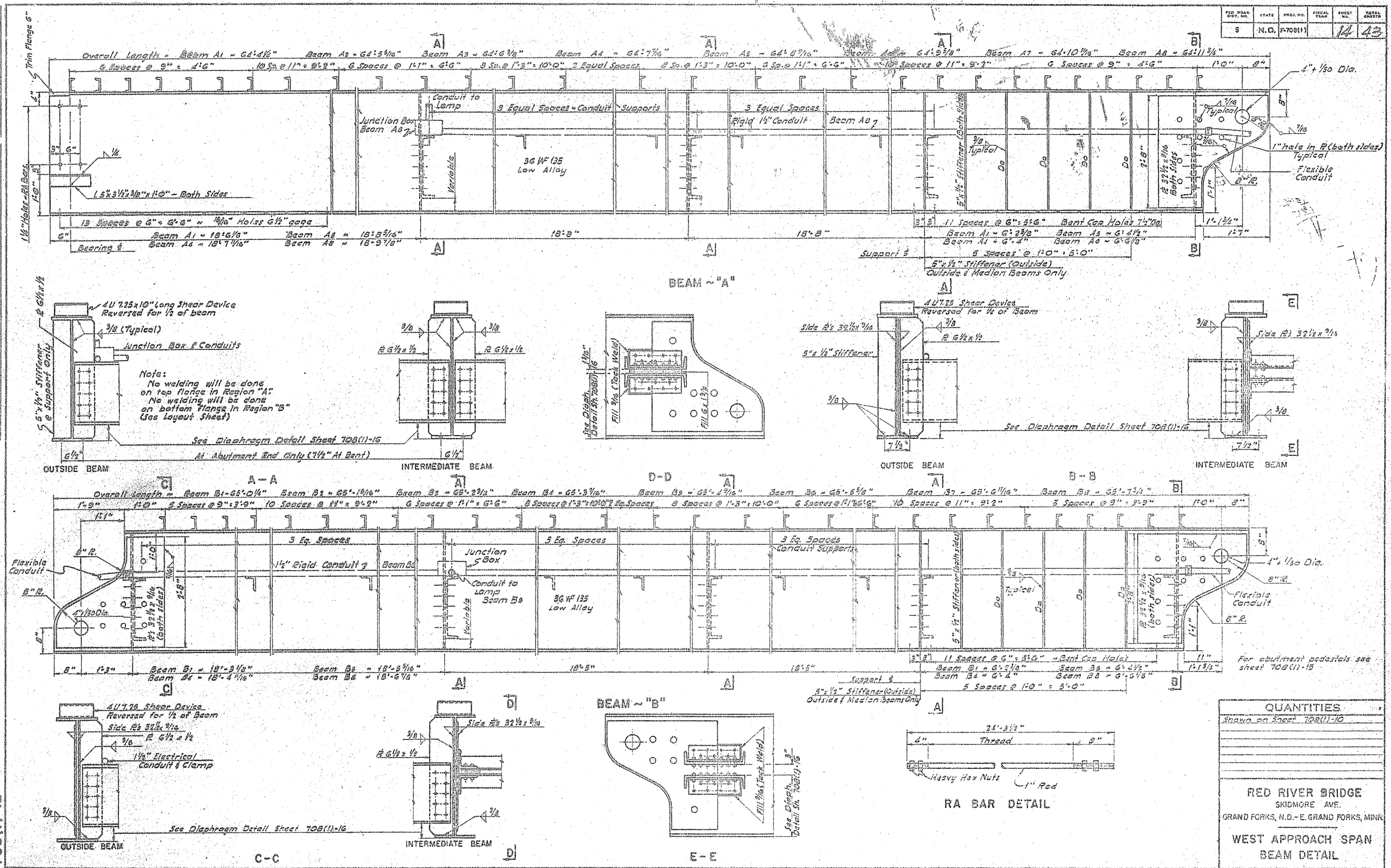
RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

LAYOUT
EAST APPROACH

708(I)-11

708(I)-11

FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(11)		14	43



QUANTITIES

Shown on Sheet 708(1)-10

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.–E. GRAND FORKS, MINN.

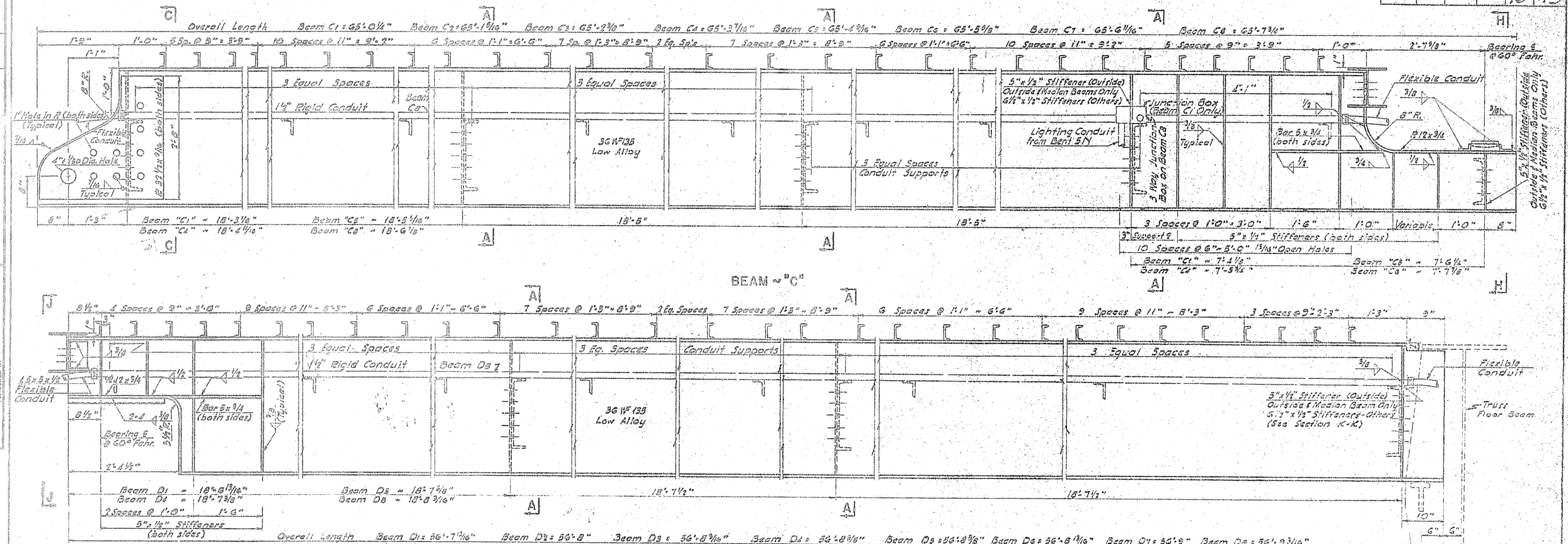
WEST APPROACH SPAN
BEAM DETAIL

708(1)-12

TRACING | CHECKED BY | MADE BY | QUANTITIES | CHECKED BY

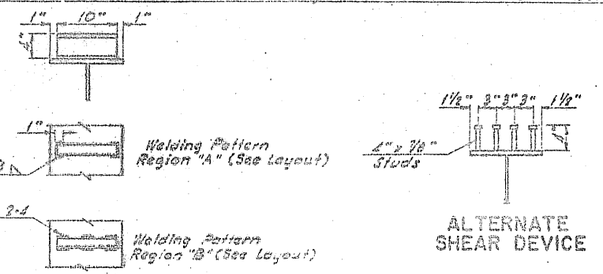
708(1)-12

FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(1)		15	43



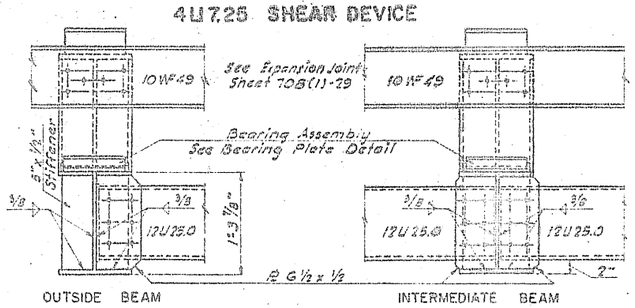
BEAM ~ "C"

BEAM ~ "D"

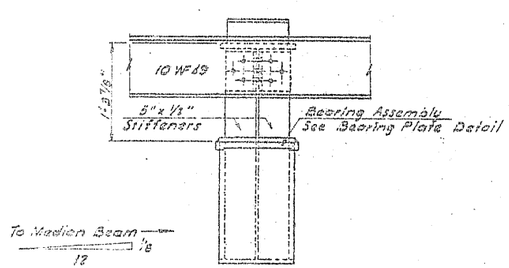


4U7.25 SHEAR DEVICE

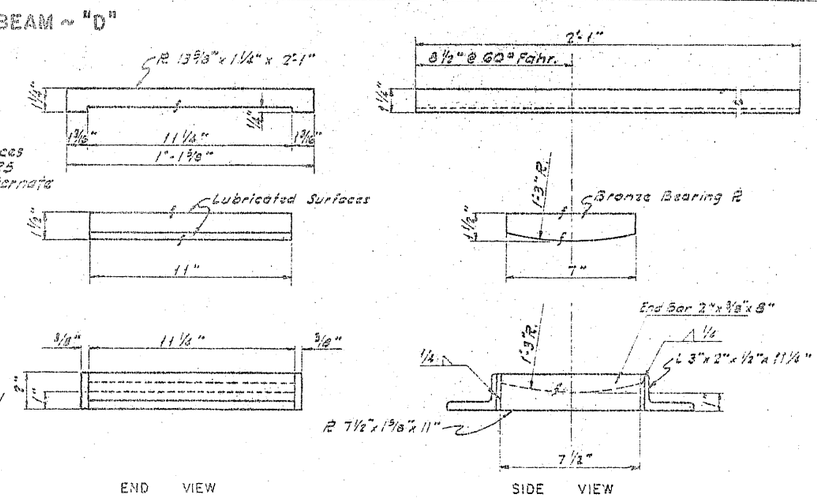
The pay weight for the shear devices will be computed using the 4U7.25 even though the studs of the alternate shear device are used.



H - H



J - J



BEARING PLATE DETAILS

NOTE:

The Bronze bearing plates shall conform to ASTM B100-55, Alloy 1. Contact surfaces shall be finished in the direction of motion to ASA B46.1-47 No. 125. Where a lubricated surface is indicated, it shall be bored in a geometric pattern of recesses to receive a lubricating material suitable for long-life service of the bearing face. The lubricated area shall comprise approximately 25 percent of the bearing face to provide a coefficient of friction for bronze on steel of approximately 10 percent for this loading.

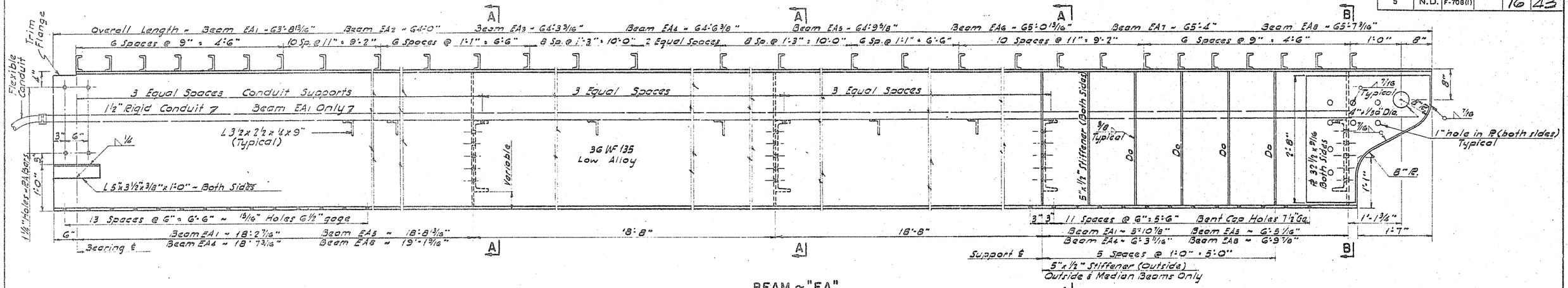
QUANTITIES

Shown on Sheet 708(1)-10

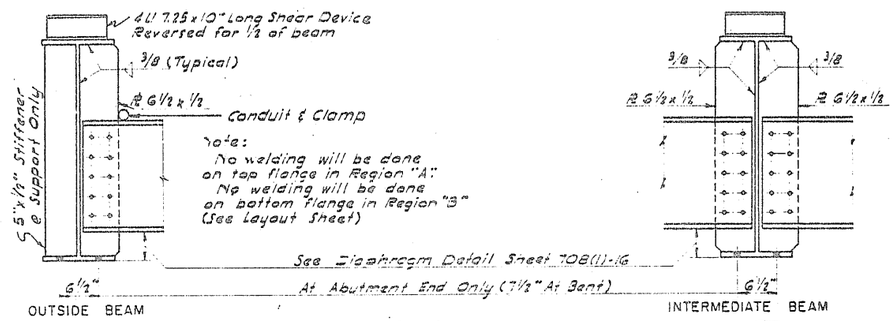
RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

WEST APPROACH SPAN
BEAM DETAIL

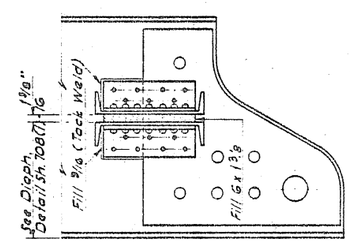
FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(1)		16	43



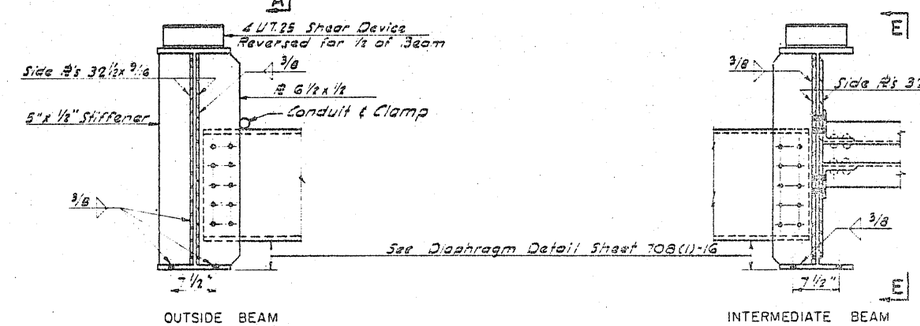
BEAM ~ "EA"



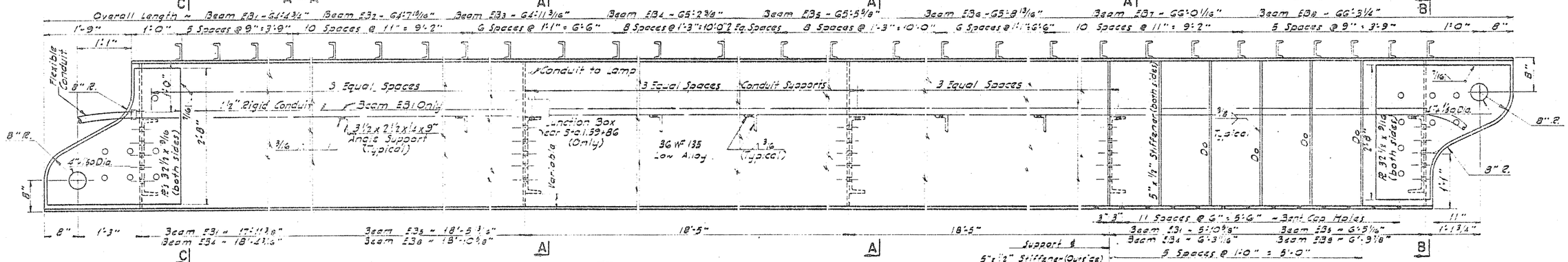
A – A

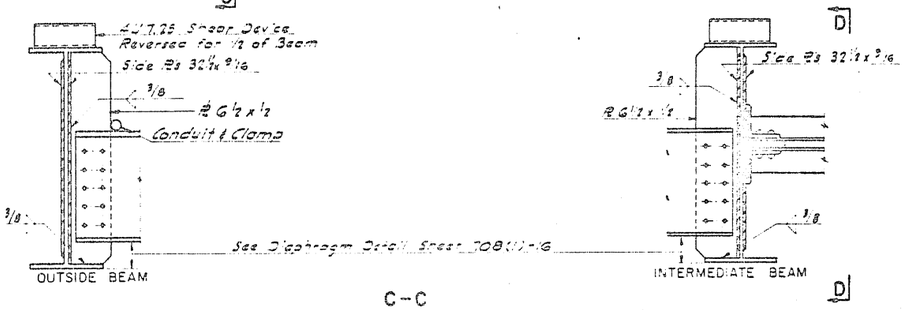


D-D



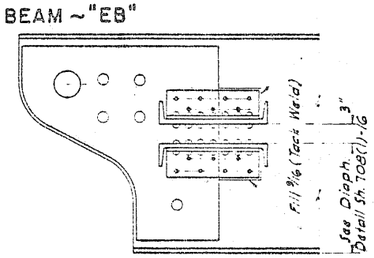
B – B



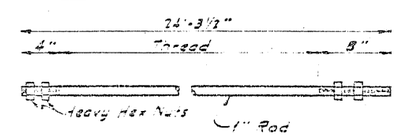


C – C

BEAM ~ "EB"



E – E



RA BAR DETAIL

QUANTITIES
Shown on Sheet 708(1)-11

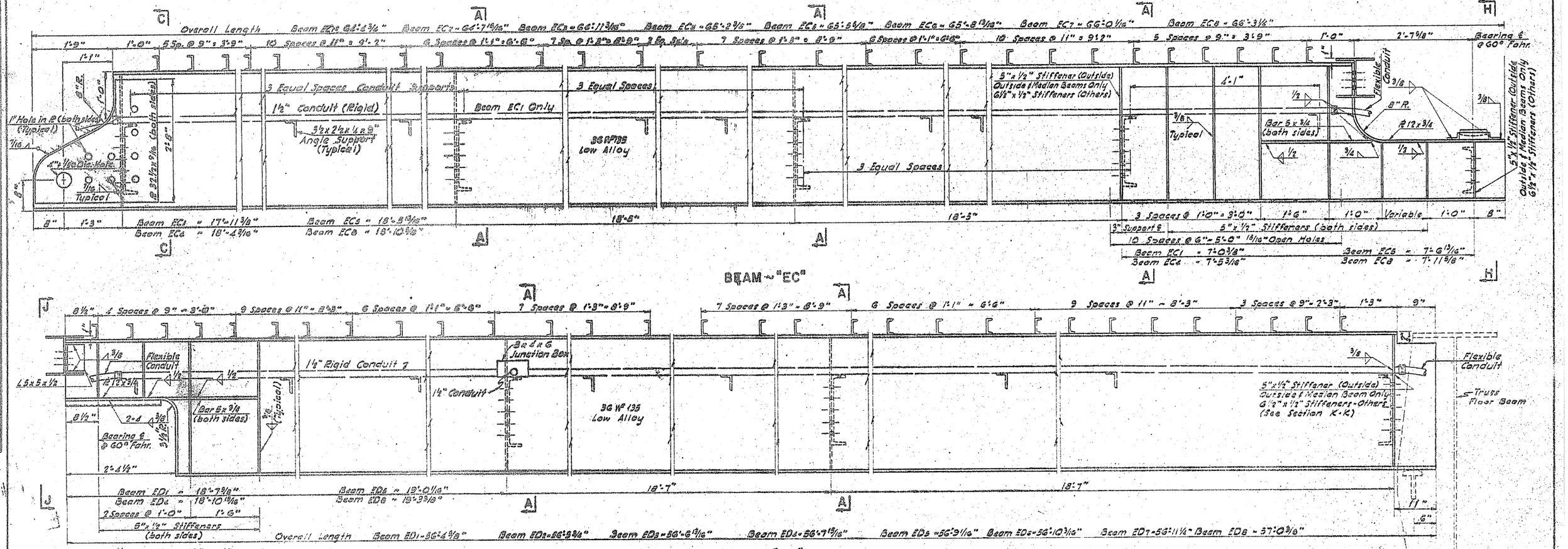
RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D. – E. GRAND FORKS, MINN.

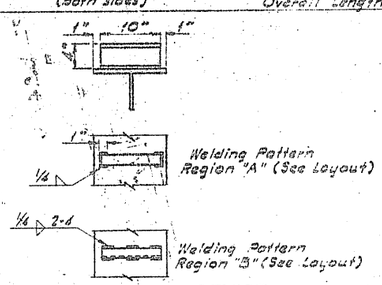
EAST APPROACH SPAN
BEAM DETAIL

TRACING MADE BY 2-21-
CHECKED BY 2-23-
QUANTITIES MADE BY
CHECKED BY

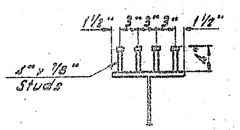
708(1)-14

FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(1)		17	43

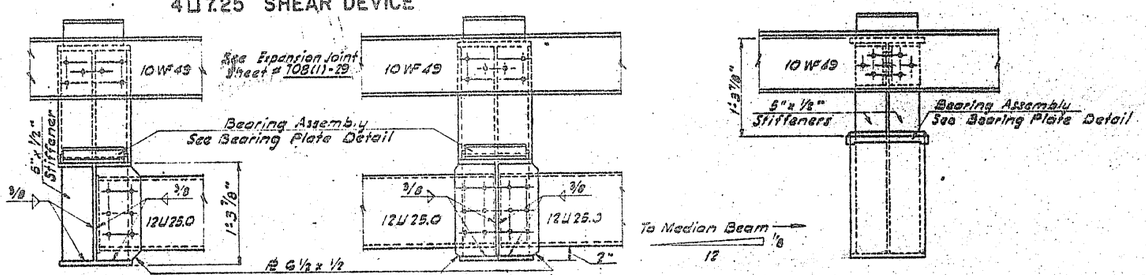




4 U 7.25 SHEAR DEVICE

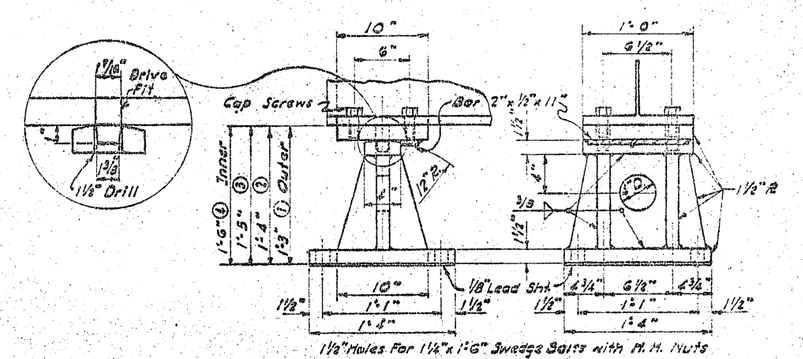


ALTERNATE SHEAR DEVICE



H - H

J - J



ABUTMENT PEDESTALS
(Cast Steel or Welded Plates)

For bearing plate details see sheet 708(1)-13

QUANTITIES
Shown on Sheet 708(1)-11

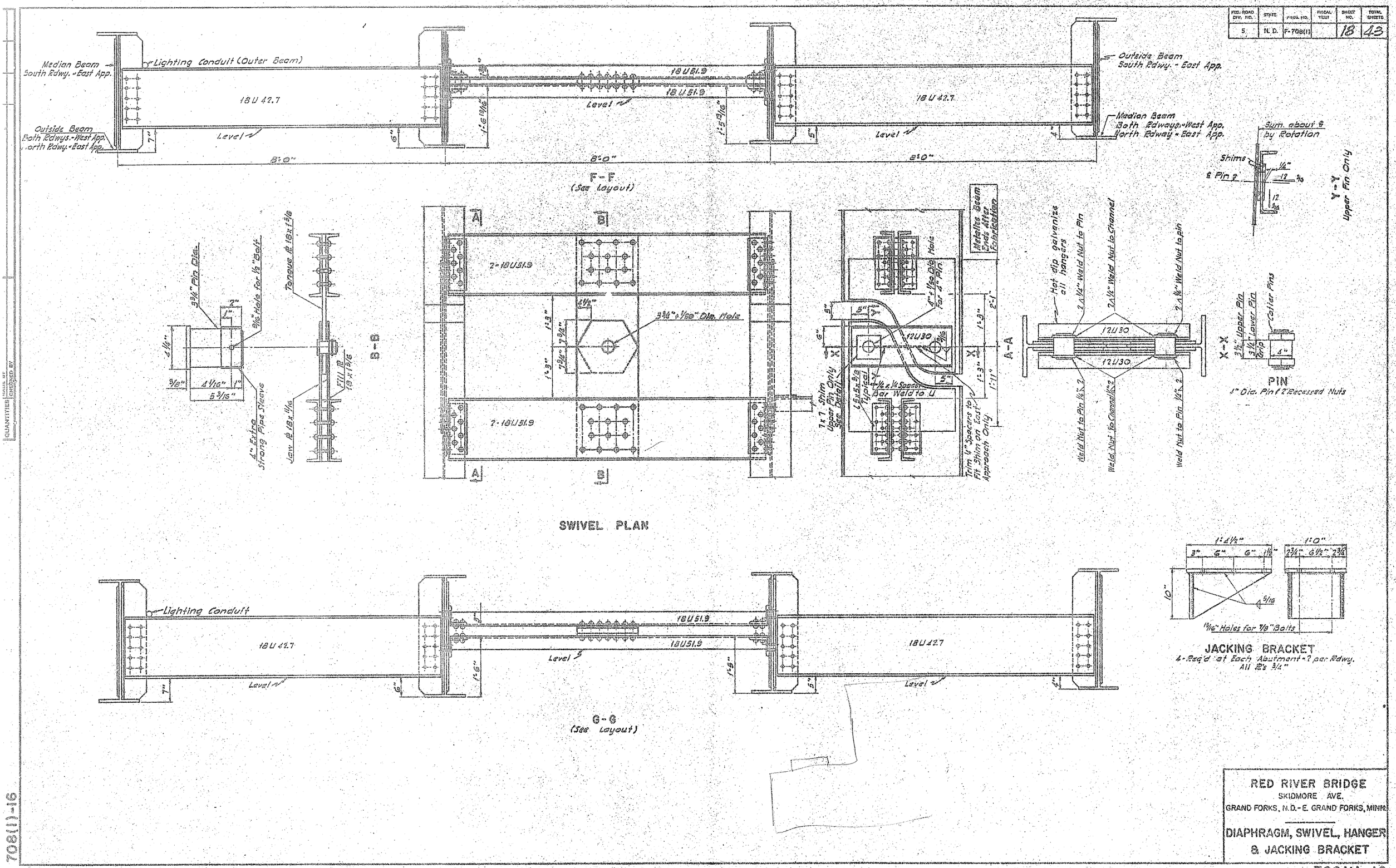
RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

EAST APPROACH SPAN
BEAM DETAIL

TRACING MADE BY 12/21
CHECKED BY 12/21
QUANTITIES MADE BY
CHECKED BY

708(1)-15

FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	F-708(1)		18	43



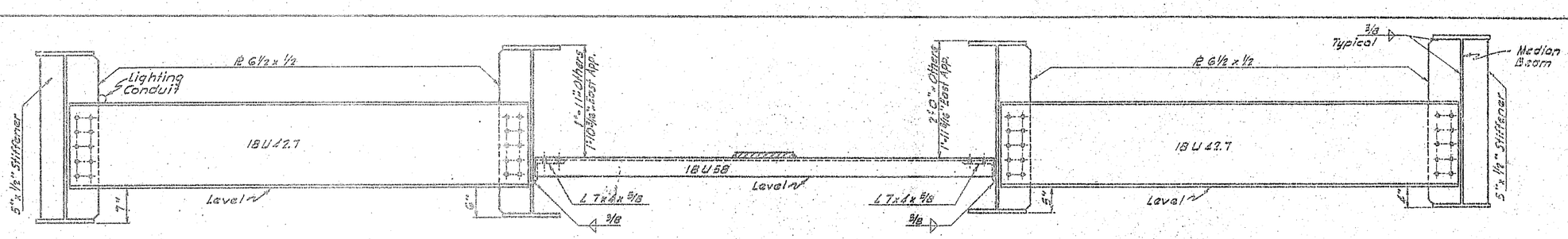
RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

DIAPHRAGM, SWIVEL, HANGER & JACKING BRACKET

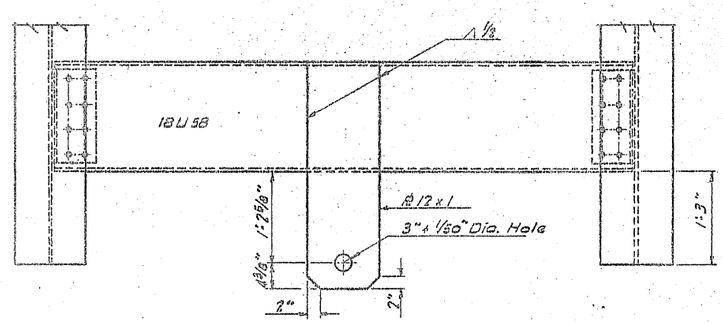
708(1)-16

708(1)-16

FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	F-708(1)		19	43

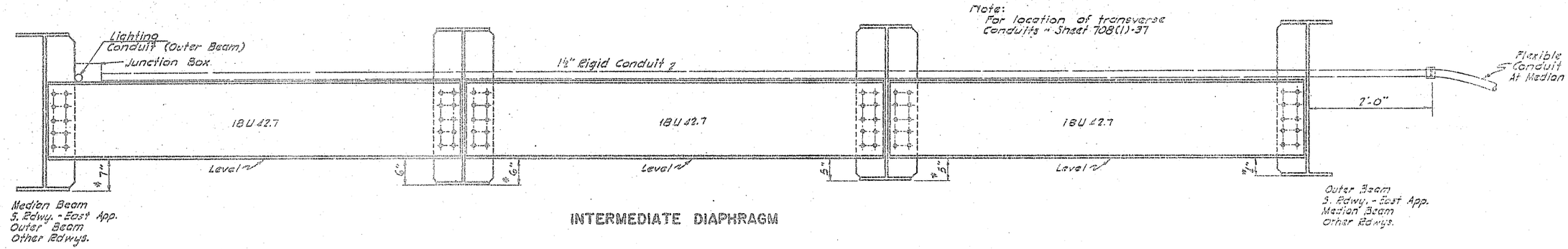


K-K
(See Layout)



PLAN
For Connection to Truss Span
See Floor Beam @ Lo
(Sheet 708(1)-31)

Note:
For location of transverse conduits - Sheet 708(1)-37



INTERMEDIATE DIAPHRAGM

* Dimensions are variable for Panels 6 & 7, South Roadway East Approach Max. = 7" Min. 4"

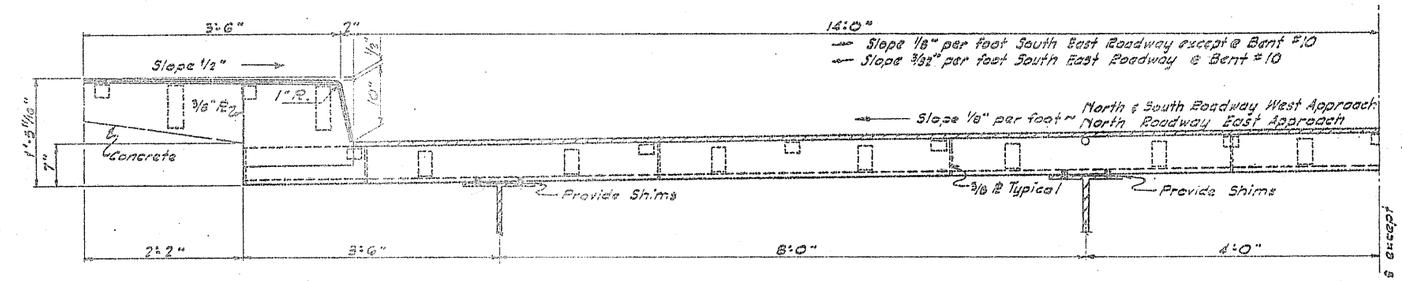
RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D. - E. GRAND FORKS, MINN.

INTERMEDIATE DIAPHRAGM AND SWIVEL DETAILS

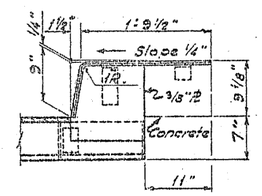
CHECKED BY

708(1)-17

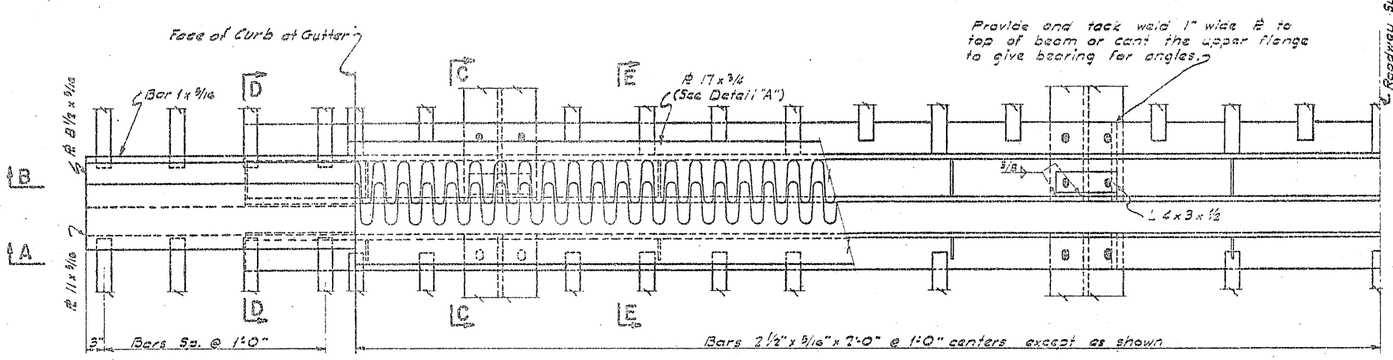
FED. ROAD DIST. NO	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(1)		20	43



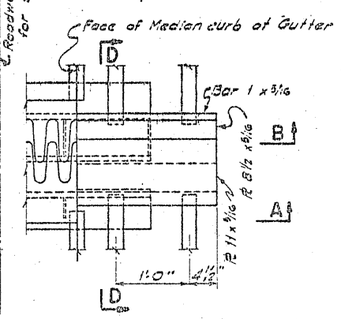
B - B



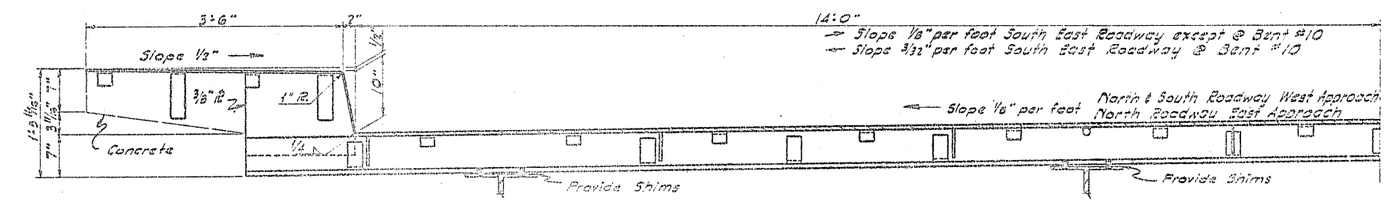
MEDIAN



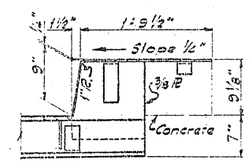
HALF PLAN



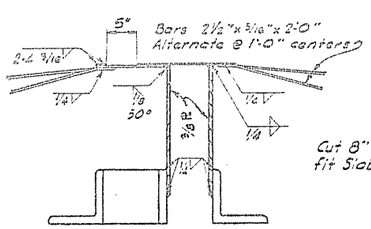
MEDIAN



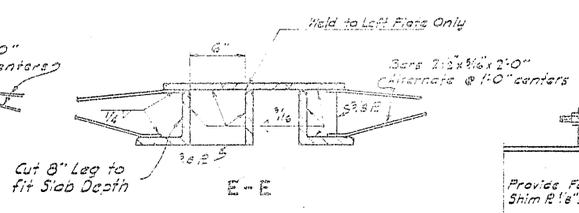
A - A



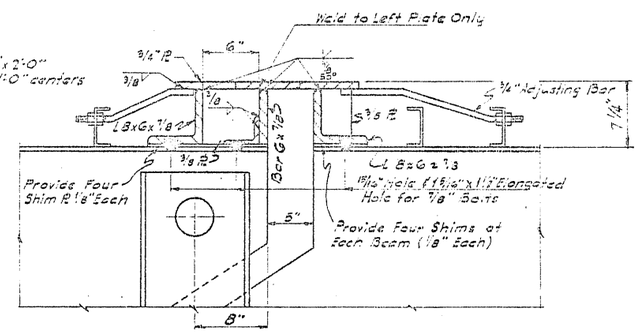
MEDIAN



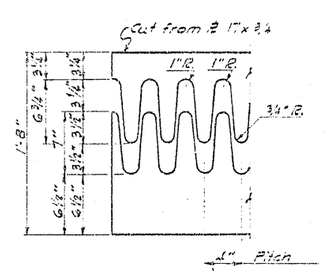
D - D



E - E



C - C



DETAIL "A"

Note:
Hot dip galvanize or metallize all expansion devices (and slab joint devices) after fabrication.

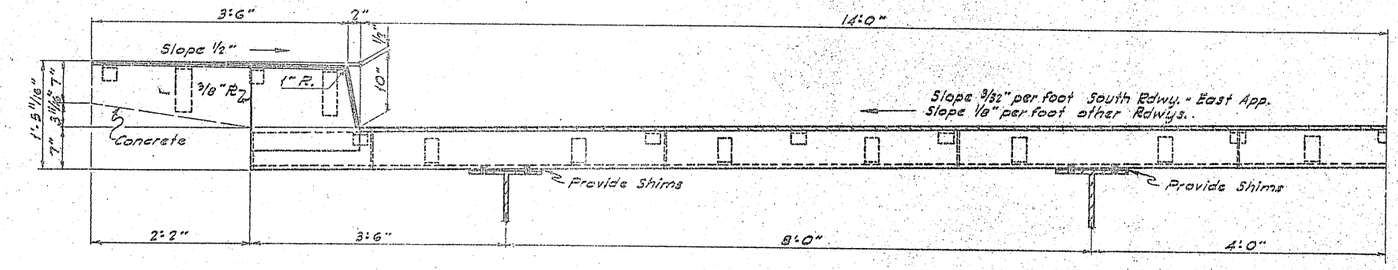
QUANTITIES

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

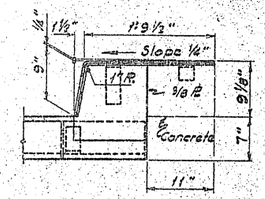
SLAB JOINT
AT SWIVEL
(APPROACH SPANS)

708(1)-18

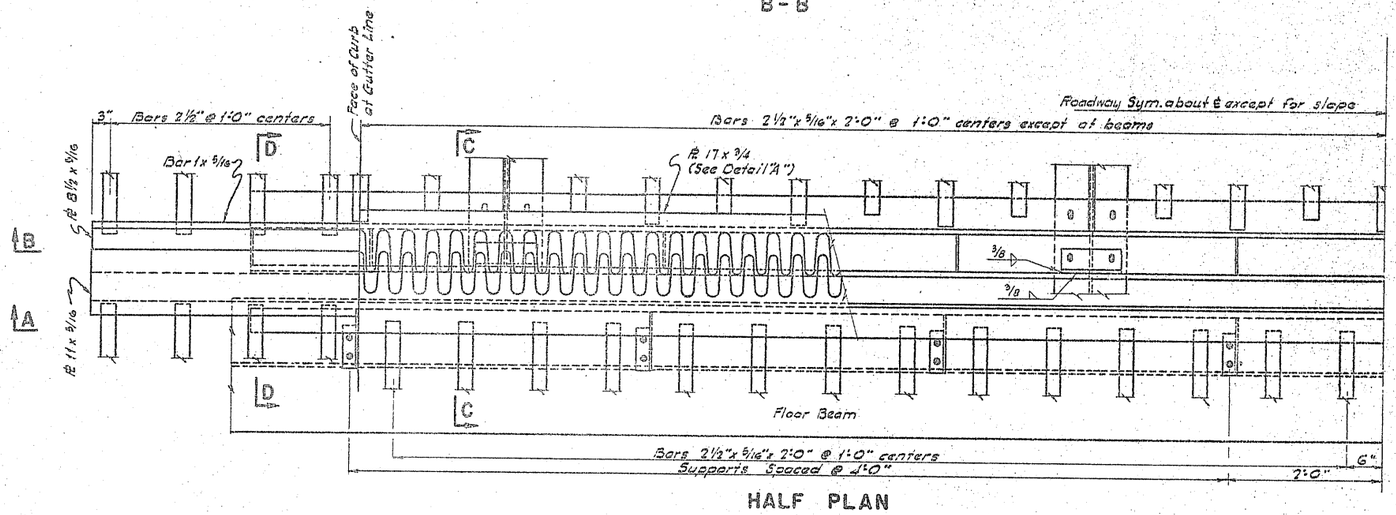
FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(1)		21	43



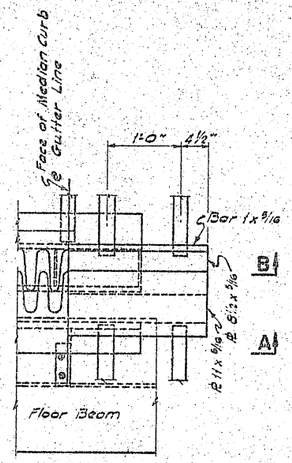
B - B



MEDIAN



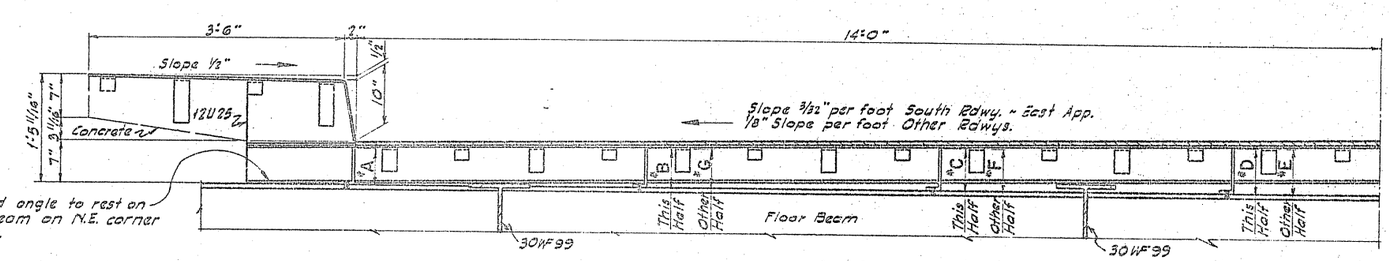
HALF PLAN



MEDIAN

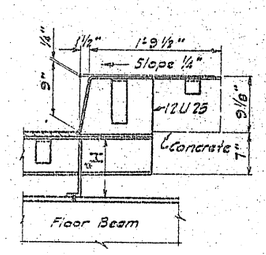
LOCATION	A	B	C	D	E	F	G	H
N.W. Rdwy.	7 5/16	7 1/8	7 3/16	8 3/8	8 11/16	9 1/2	10"	10 1/2
S.W. Rdwy.	7 1/16	7 1/8	7 3/16	8 3/8	8 11/16	9 1/2	10"	10 1/2
N.E. Rdwy.	6 3/8	7 1/8	6 11/16	7 7/8	8 1/16	8 11/16	9 3/16	9 11/16
S.E. Rdwy.	7 11/16	7 3/4	7 1/2"	8"	8 1/16	8 7/8	9 1/4"	9 11/16

Note: Deduct 1/8" in all of the above dimensions to allow for shimming.

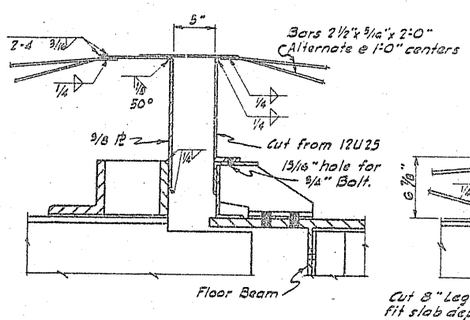


A - A

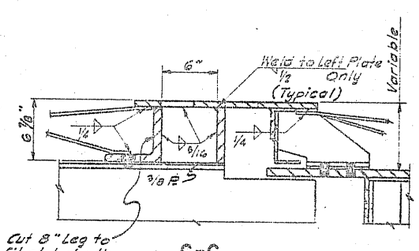
* See Table Above



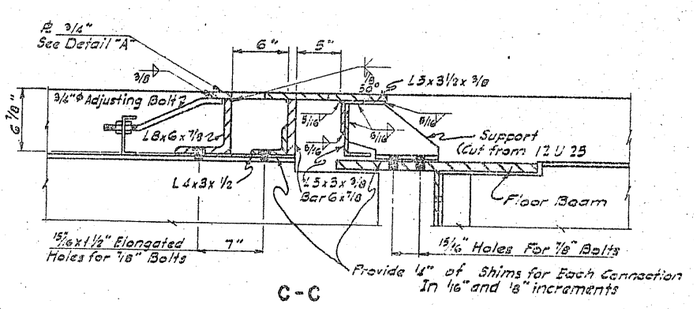
MEDIAN



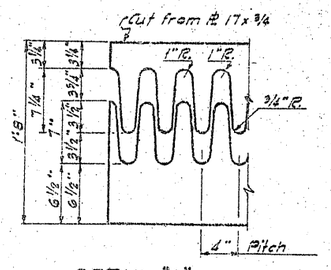
D-D



C-C
Showing 2 1/2" Bars



C-C



DETAIL "A"

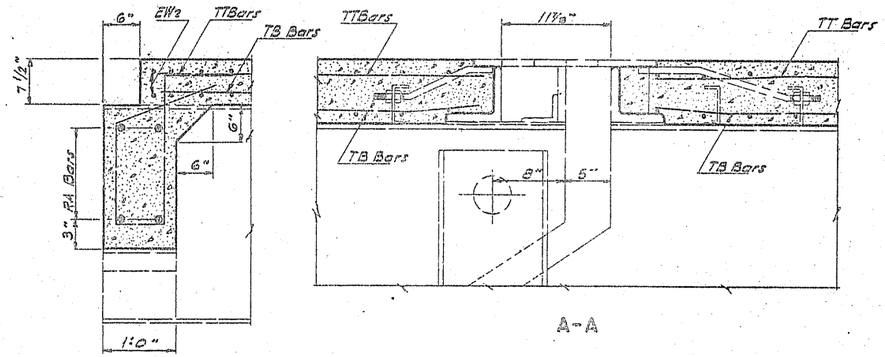
Note:
Hot dip galvanize or metalize all expansion devices (and slab joint devices) after fabrication.

QUANTITIES

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

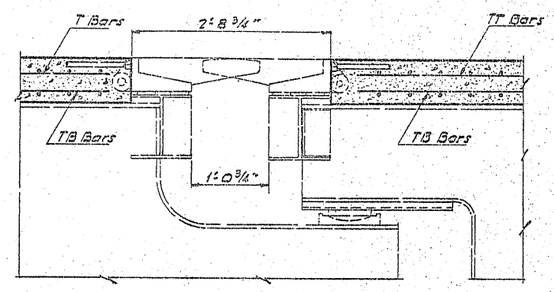
SLAB JOINT AT SWIVEL
(END OF TRUSS ONLY)

FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(1)		22	43

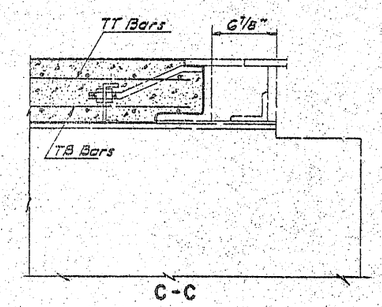


A-A

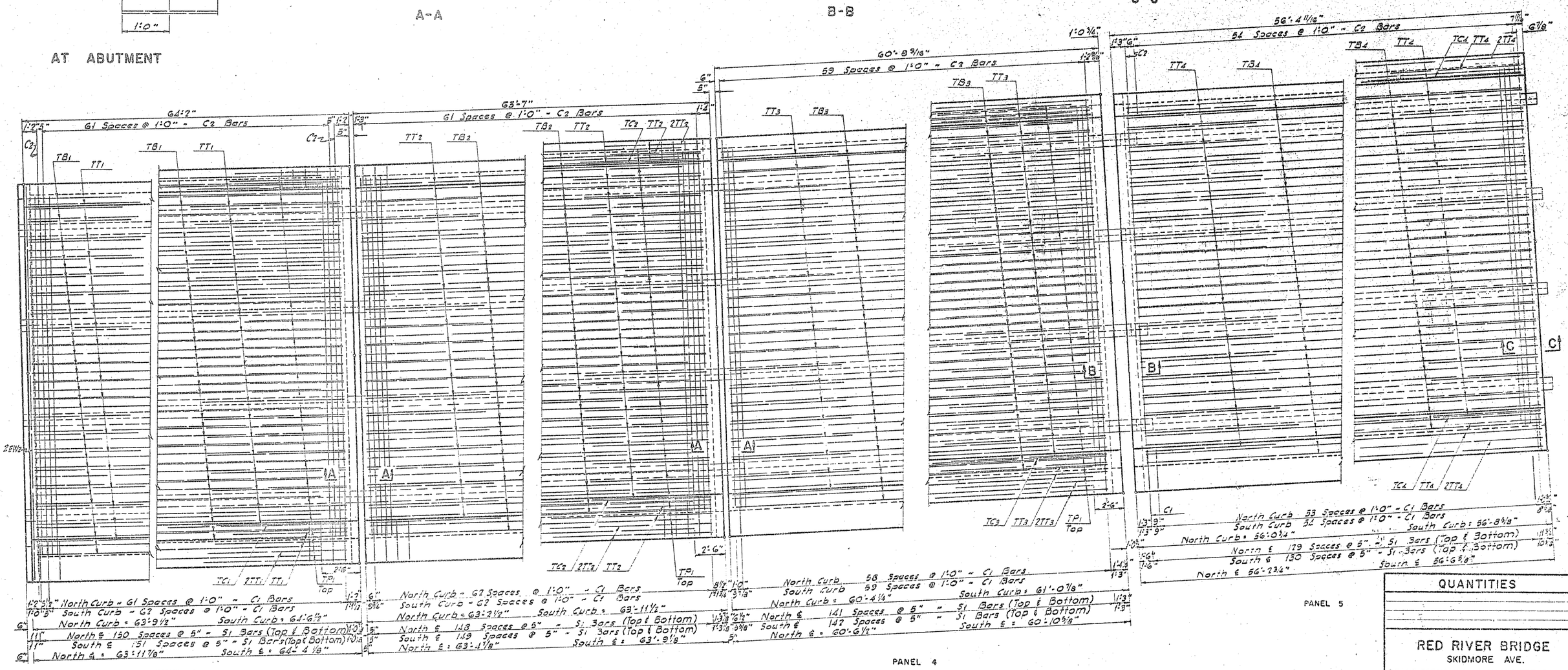
AT ABUTMENT



B-B



C-C



PART PLAN - APPROACH SPANS

For deck drain locations see sheet 708(1)-21

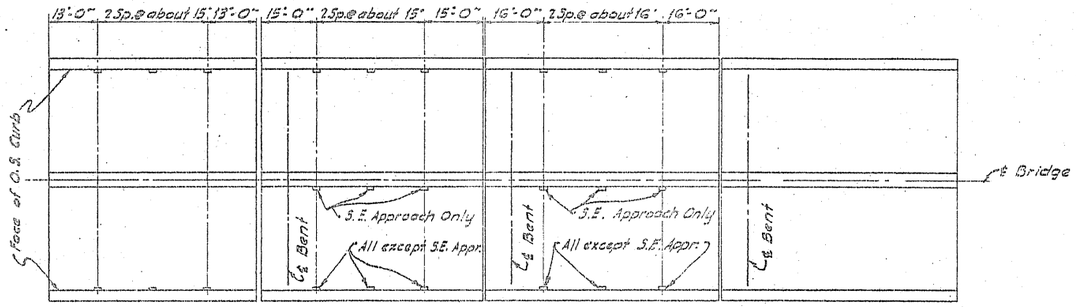
QUANTITIES

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

WEST APPROACH SLAB

708 (1)-20

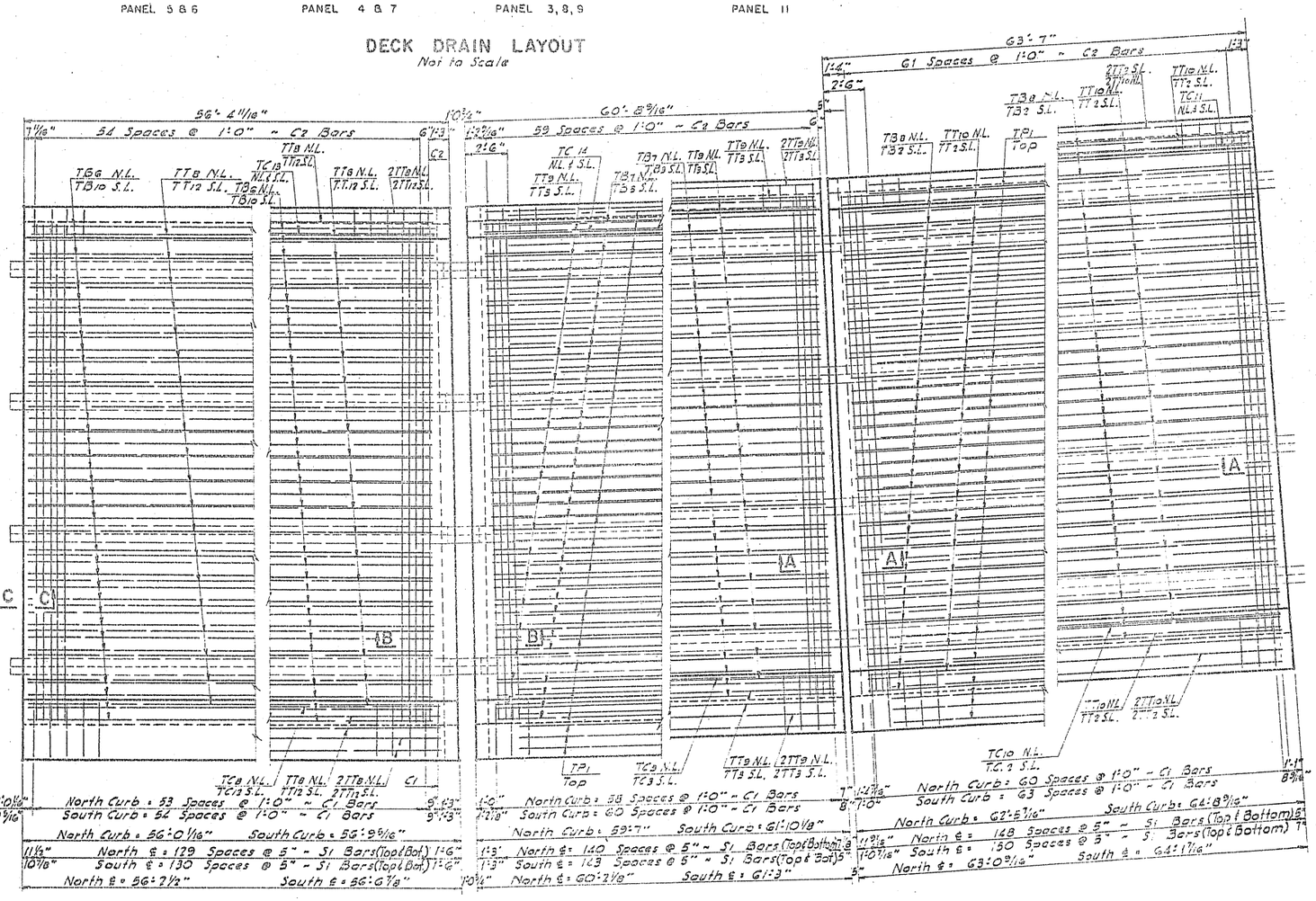
FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(1)		23	43



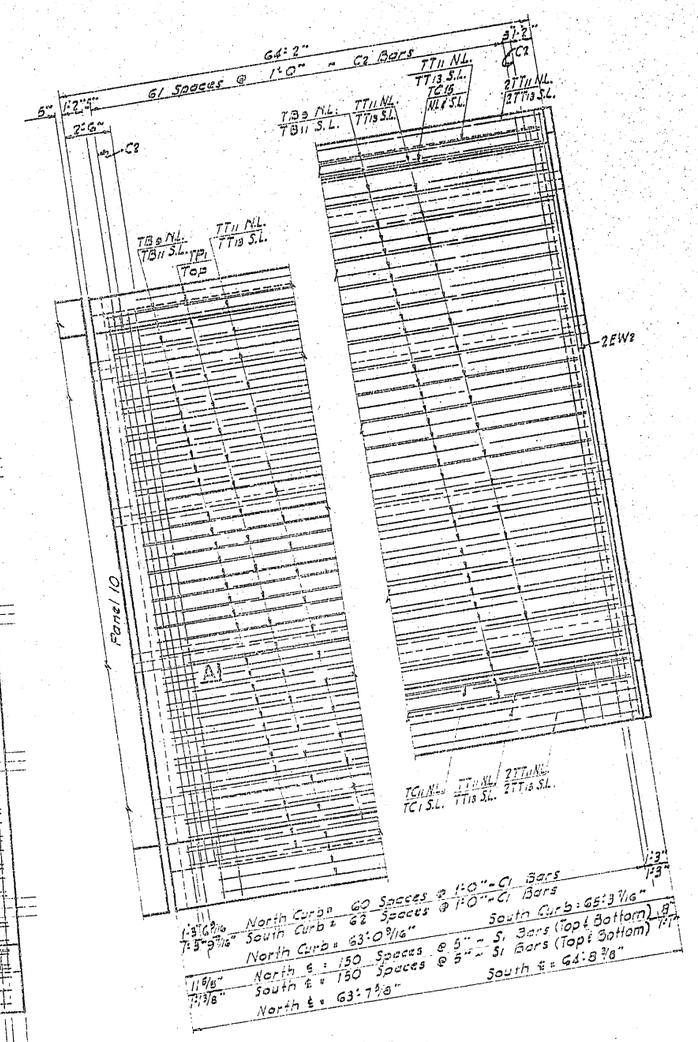
NOTE:
No Deck Drains shall be placed in panels 1, 2, 10 & 11. East panels as shown, west panels opposite.

DECK DRAIN LAYOUT

Not to Scale



HALF PLAN – APPROACH SPANS



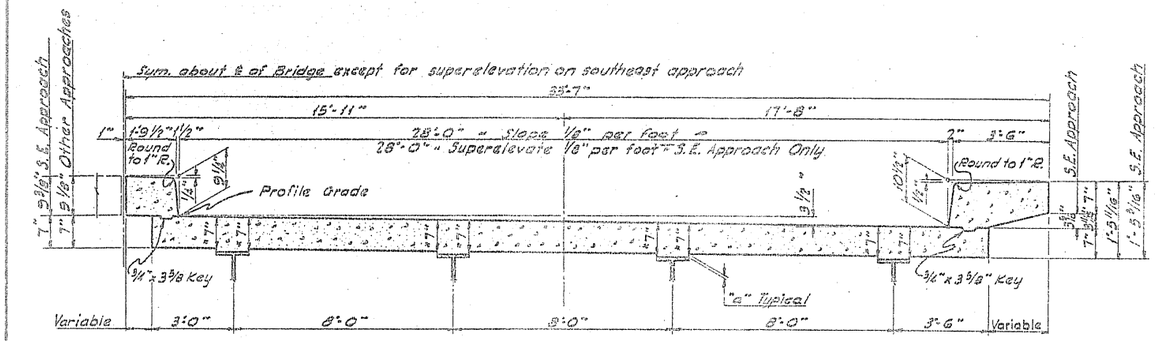
Note:
For sections of edgebeam, A-A, B-B & C-C see sheet # 708(1)-22 & sheet # 708(1)-20

QUANTITIES

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.–E. GRAND FORKS, MINN.

EAST APPROACH SLAB

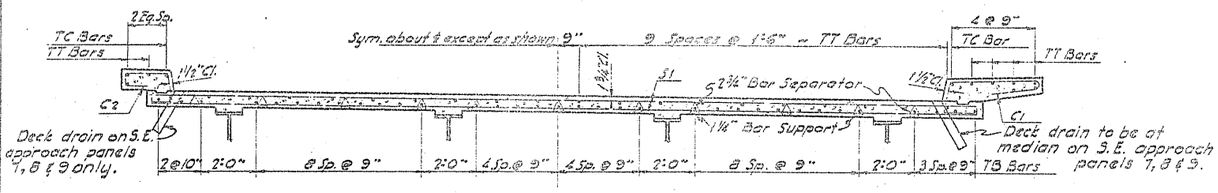
FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(1)		24	43



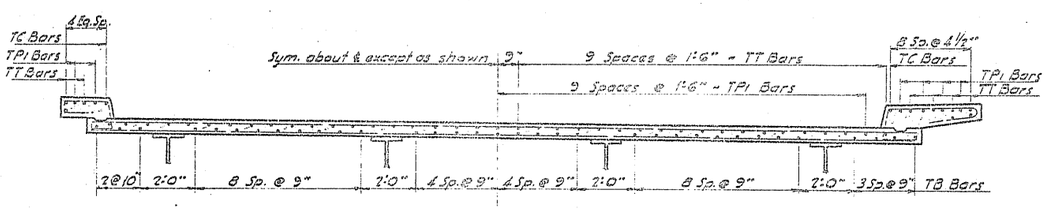
The S.E. approach slab shall be warped to give a superelevation of 1/8" per foot. The profile grade and south gutter grades to give this surface are shown this Sheet.

SLAB SECTION
Showing Dimensions

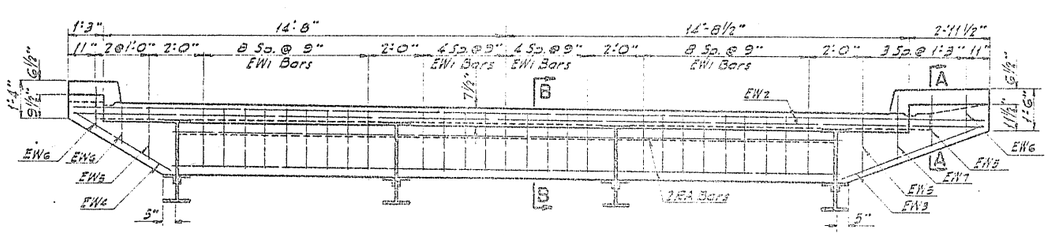
"a" The "T" dimension shown must be maintained by varying dimension "a" to compensate for deflections and vertical curve corrections.



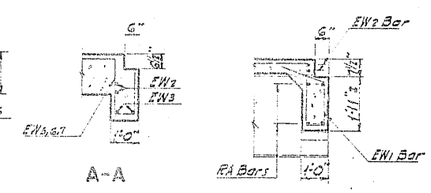
SLAB SECTION
Showing Reinforcing Steel

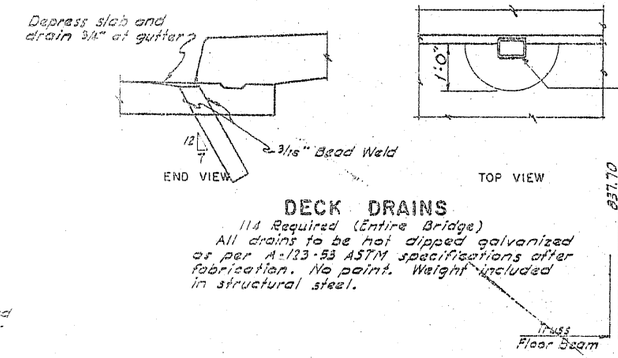


SLAB SECTION
Showing Reinforcing Over Bents

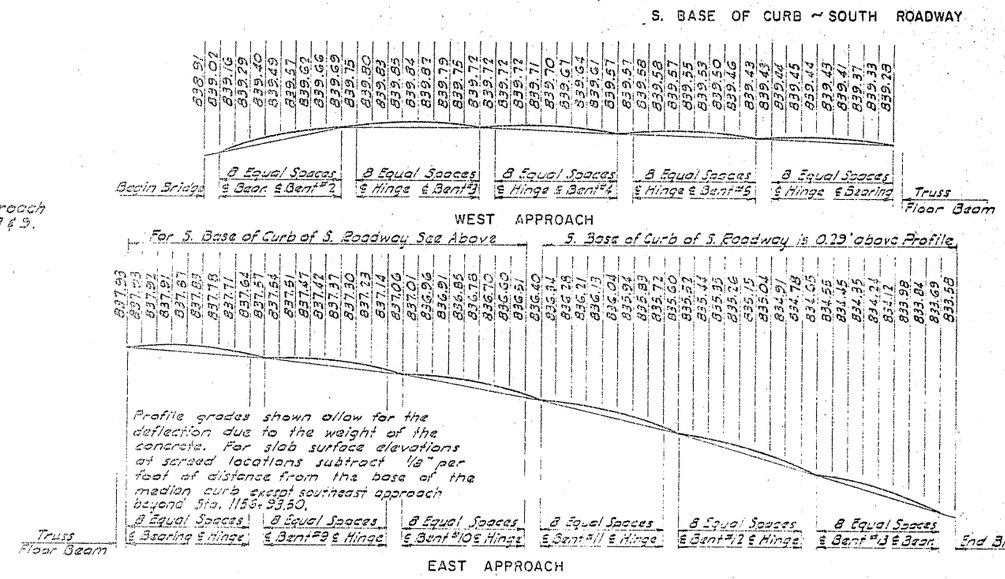


EDGE BEAM





DECK DRAINS
114 Required (Entire Bridge)
All drains to be hot dipped galvanized as per A-123-53 ASTM specifications after fabrication. No paint. Weight included in structural steel.



Profile grades shown allow for the deflection due to the weight of the concrete. For slab surface elevations at screed locations subtract 1/8" per foot of distance from the base of the median curb except southeast approach beyond Sta. 1156+93.50.

ADJUSTED PROFILE GRADES

BAR LIST

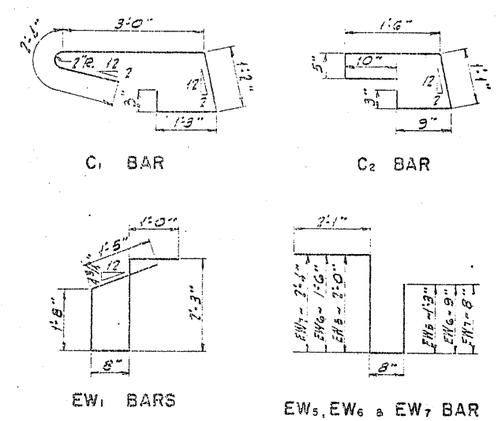
MARK	NO	SIZE	LENGTH	SHAPE
C1	1364	5	8'-0"	Bent
C2	1280	4	4'-10"	"
EW1	108	4	7'-0"	Bent
EW2	8	6	33'-4"	Str.
EW3	8	4	5'-9"	"
EW4	8	4	4'-3"	"
EW5	12	4	6'-0"	Bent
EW6	12	4	5'-0"	"
EW7	4	4	5'-9"	"
S1	6390	5	30'-7"	Str.
TB1	136	5	33'-0"	Str.
TB2	476	5	37'-9"	"
TB3	204	5	31'-3"	"
TB4	136	5	29'-0"	"
TB6	68	5	28'-3"	"
TB7	68	5	30'-9"	"
TB8	204	5	32'-3"	"
TB9	68	5	32'-6"	"
TB10	68	5	28'-6"	"
TB11	68	5	33'-3"	"
TC1	10	6	33'-3"	Str.
TC2	22	6	33'-0"	"
TC3	10	6	31'-6"	"
TC4	8	6	29'-3"	"
TC8	2	6	28'-3"	"
TC9	2	6	30'-3"	"
TC10	6	6	31'-9"	"
TC11	14	6	32'-3"	"
TC12	2	6	28'-9"	"
TC13	4	6	28'-6"	"
TC14	4	6	31'-0"	"
TC15	4	6	32'-9"	"
TT1	116	4	32'-9"	Str.
TT2	206	4	37'-6"	"
TT3	174	4	31'-0"	"
TT4	116	4	28'-9"	"
TT8	58	4	28'-0"	"
TT9	58	4	30'-6"	"
TT10	174	4	32'-0"	"
TT11	58	4	32'-3"	"
TT12	58	4	28'-3"	"
TT13	58	4	33'-0"	"
TP1	445	5	10'-0"	Str.

See lighting details Sheet 708(1)-37 for special details at light posts.

Approach spans shall be poured in sequence starting at the truss and working toward the abutment. Each span, however, shall be poured from the suspended to the cantilever end.

Sidewalks or medians shall not be poured until the slabs in adjacent spans have been poured.

The vertical faces of the sidewalk and median shall be constructed parallel to the center line. The vertical edges of slabs may be constructed parallel to the beam



BENT BAR DETAILS

QUANTITIES

Concrete Class AE-2	1114.6	C.Y.
Reinforcing Steel	302,992	Lbs.
Structural Steel	3520	Lbs.

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

APPROACH SLAB DETAILS

CHECKED BY R.E.S.
MADE BY M.A.E.
CHECKED BY D.D.B.
TRACING
QUANTITIES MADE BY
CHECKED BY

708(1)-22

FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5.	N. D.	F-708(1)		25	43

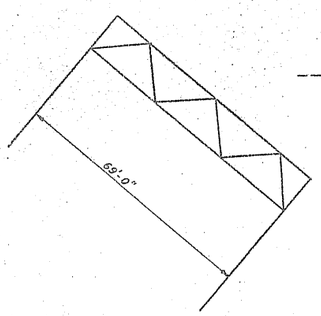
STRESSES AND SECTIONS

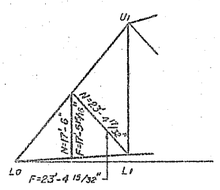
LINE	MEMBER		BOTTOM CHORD						TOP CHORD							DIAGONALS									VERTICALS								LINE
			L_0-L_1	L_1-L_2	$L_2'-L_3'$ / L_2-L_3	$L_3'-L_4'$ / L_3-L_4	L_4-L_4'	$L_2'-L_0'$	L_0-U_1	$U_1'-U_2'$ / U_1-U_2	$U_2'-U_3'$ / U_2-U_3	$U_3'-U_4'$ / U_3-U_4	U_4-U_4'	$U_1'-L_0'$	$U_{1/2}-L_1$	U_1-L_2	U_2-L_3	U_3-L_4	U_4-L_4'	$U_4'-L_4$	$U_3'-L_4'$	$U_2'-L_3'$	$U_1'-L_2'$	$U_{1/2}-L_{1/2}$	U_1-L_1	U_2-L_2	U_3-L_3	$U_4'-L_4'$ / U_4-L_4	$U_3'-L_3'$	$U_2'-L_2'$	$U_1'-L_1'$	
1	DEAD LOAD (Kips)		668.4T.	698.4T.	944.9T.	1,089.1T.	1,169.8T.	668.4T.	1008.1C.	978.4C.	1,103.1C.	1,171.4C.	1,169.8C.	1008.1C.	668.4T	590.6T	356.6T	225.4T	65.3T	0.0	149.4T	248.0T	419.7T	1000.1C.	310.7C	253.5C.	103.1C.	62.9T.	13.0C	123.3C.	188.7T.	1
2	LIVE LOAD (Kips)	REGULAR	143.6T.	143.6T.	203.0T.	234.0T.	251.3T.	143.6T.	232.7C.	210.2C.	237.0C.	251.6C.	251.3C.	232.7C.	159.6T	149.0T	110.0T	93.6T.	74.5T.	67.2T.	83.2T.	91.7T.	114.5T	238.7C.	99.0C	88.0C.	68.0C.	66.6T	55.7C.	66.6C.	77.9T.	2
3		REVERSE														14.4C.	31.4C.	42.8C.	58.8C.		51.6C.	42.0C.	26.0C.		41.3T	36.9T.	50.0T.	49.5C.	60.3T	49.4T		3
4	IMPACT	REGULAR	17.8T.	17.8T.	25.2T.	29.0T.	31.2T.	17.8T.	28.9C.	26.1C.	29.4C	31.2C.	31.2C.	28.9C.	45.0T.	41.7T.	30.8T	26.2T.	20.8T	8.3T.	10.3T.	11.4T.	14.2T	66.8C.	27.7C.	24.6C	19.0C.	8.3T	6.9C.	8.3C.	9.7T.	4
5		REVERSE														4.0C.	8.8C.	12.0C.	16.5C.		6.4C.	5.2C.	3.2C.		11.6T	10.3T.	14.0T.	6.1C	7.5T	6.1T.		5
6	ADD FOR REVERSAL																												33.9T.C.			6
7	LINES 1+2+4+6		829.8T.	829.8T.	1,173.0T.	1,352.1T.	1,452.3T.	829.8T.	1269.7C.	1,214.7C.	1,369.5C.	1,454.2C	1,452.3C.	1,269.7C	873.0T	781.5T.	499.3T	346.2T	160.6T.	75.5T.	242.9T.	351.1T.	548.4T.	1306.6C.	437.4C	366.1C.	190.1C.	137.8T.	109.5C.	198.2C.	276.3T	7
8	DESIGN FOR (Kips)		829.8T	829.8T.	1,173.0T.	1,352.1T.	1,452.3T.	829.8T.	1,269.7C	1,214.7C.	1,369.5C.	1,454.2C	1,452.3C.	1,269.7C.	△ 873.0T.	△ 781.5T.	△ 499.3T	△ 346.2T	△ 160.6T.	75.5T.	242.9T.	351.1T.	548.4T.	△ 1306.6C.	△ 437.4C	△ 366.1C.	△ 190.1C.	137.8T.	109.5C.	198.2C.	276.3T	8
9	SECTION		BOX SIDE PL 7/16x24 COVER PL 2-7/8x17 WEB-1/4x24 Q&T	BOX SIDE PL 7/16x24 COVER PL 5/8x17 Q&T, L.A.	BOX SIDE PL 7/16x24 COVER PL 5/8x17 LA	BOX SIDE PL 1 1/16x24 COVER PL 5/8x17 LA	BOX SIDE PL 1 3/16x24 COVER PL 5/8x17 LA	BOX SIDE PL 7/16x24 COVER PL 5/8x17 CARBON	BOX SIDE PL 1 3/16x24 COVER PL 5/8x17 LA	BOX SIDE PL 1 1/16x24 COVER PL 5/8x17 LA	BOX SIDE PL 1 3/16x24 COVER PL 5/8x17 LA	BOX SIDE PL 1 3/16x24 COVER PL 5/8x17 LA	BOX SIDE PL 1 3/16x24 COVER PL 5/8x17 LA	BOX SIDE PL 7/16x24 COVER PL 5/8x17 LA	WELDED E GIRDER FL 14x13 WEB 1/16x36 5/8 LA	18WF114 E 3/8x5x1'-6" LA	18WF77 LA	18WF64 LA	18WF64 CARBON	18WF64 CARBON	18WF64 CARBON	18WF64 LA	18WF96 LA	WELDED E GIRDER FL 14x14 WEB 1/16x15 1/8 LA	18WF96 LA	18WF96 CARBON	18WF96 CARBON	18WF96 CARBON	18WF96 CARBON	18WF96 CARBON	18WF96 CARBON	9
10	GROSS AREA (Sq. In.)		114.50	58.25	58.25	67.25	73.25	58.25	73.25	67.25	73.25	79.25	79.25	73.25	39.25	41.01	22.53	18.80	18.80	18.80	18.80	18.80	28.22	64.68	28.22	28.22	28.22	28.22	28.22	28.22	28.22	10
11	NET AREA (Sq. In.)		107.50	49.38	49.38	56.88	61.88	49.38	73.25	67.25	73.25	79.25	79.25	73.25	33.25	31.60	19.31	16.06	16.06	16.06	16.06	16.06	24.90	64.68	28.22	28.22	28.22	24.90	28.22	28.22	24.90	11
12	LENGTH (INCHES)		372.00	372.00	372.00	372.00	372.00	744.00	561.00	385.22	376.81	372.53	372.00	561.00	281.00	561.06	639.36	689.04	706.00	706.00	689.04	639.36	561.06	210.00	420.00	520.00	580.00	600.00	580.00	520.00	420.00	12
13	PLANE PARALLEL TO TRUSS	UNSUPPORTED LENGTH	186.00	372.00	372.00	372.00	372.00	372.00	280.50	385.22	376.81	372.53	372.00	561.00	281.00	280.53	319.68	344.52	353.00	353.50	344.52	319.68	280.50	210.00	210.00	260.00	290.00	300.00	290.00	260.00	210.00	13
14		RADIUS OF GYRATION	9.45	8.70	8.70	8.50	8.38	8.70	8.38	8.50	8.38	8.29	8.29	8.38	2.65	2.76	1.98	1.93	1.93	1.93	1.93	1.93	2.71	2.98	2.71	2.71	2.71	2.71	2.71	2.71	2.71	14
15		L/r	19.70	42.70	42.80	43.80	44.40	42.80	33.50	45.32	44.97	44.94	44.87	66.95	106.04	101.64	161.50	178.50	182.90	182.90	178.50	165.60	103.51	70.50	77.50	95.94	107.01	110.70	107.00	95.94	77.50	15
16	PLANE PERP. TO TRUSS	UNSUPPORTED LENGTH	372.00	372.00	372.00	372.00	372.00	372.00	415.69	385.22	376.81	372.53	372.00	415.69	281.00	561.06	639.36	689.04	706.00	706.00	689.04	639.36	561.06	210.00	325.13	325.13	325.13	325.13	310.00	324.75	324.75	16
17		RADIUS OF GYRATION	6.02	7.88	7.88	8.02	7.91	7.88	7.91	8.02	7.91	7.98	7.98	7.91	7.00	7.79	7.54	7.46	7.46	7.46	7.46	7.46	7.70	7.18	7.70	7.70	7.70	7.70	7.70	7.70	7.70	17
18		L/r	61.79	47.20	47.20	46.40	47.00	47.20	52.60	48.03	47.63	46.68	46.62	52.60	40.14	72.00	84.80	92.40	94.60	94.60	92.40	85.70	72.86	29.20	42.20	42.20	42.20	42.20	40.30	42.20	42.20	18
19	BASIC UNIT STRESS (Kips/Sq. In.)	ALLOWABLE*	53.5	23.4	23.4	23.3	23.3	17.4	18.4	18.4	18.4	18.5	18.5	17.6	⊕ 25.8	⊕ 24.5	⊕ 25.6	26.1	17.1	17.1	17.1	26.2	23.6	⊕ 19.5	17.2	12.7	12.1	18.0	12.1	12.7	18.0	19
20		ACTUAL	7.3	16.8	23.8	23.8	23.5	16.8	17.3	18.1	18.7	18.4	18.3	17.3	26.3	24.6	25.9	21.6	10.0	4.6	15.1	21.9	22.0	20.2	15.5	13.0	6.7	5.5	3.9	7.0	11.1	20
21	CAPACITY (Kips)		6,130.0T.	1,153.5T.	1,153.5T.	1,326.5T.	1,443.0T.	857.2T.	1,344.9C.	1,237.4C.	1,347.8C.	1,462.2C	1,462.2C.	1,287.0C	857.9T.	778.8T	494.0T.	419.5T	274.6T	274.6T.	275.0T.	420.5T.	587.6T.	1,261.3C.	486.5C.	358.4C.	342.6C.	448.2T	341.5C.	358.4C	448.2T	21
22	DESIGN CONNECTION FOR		998.7	998.7	1,173.0	1,352.1	1,443.0	843.8	1,307.3	1,226.1	1,369.5	1,458.2	1,457.2	1,278.4	873.0	781.5	499.3	382.9	217.6	206.0	258.9	385.8	568.0	1306.6	462.0	366.2	266.3	336.2	256.1	278.3	362.3	22

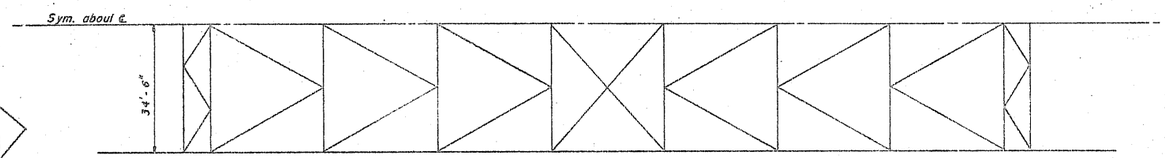
*ADJUSTED TO COMPENSATE FOR FLEXURAL STRESSES DUE TO THE WEIGHT OF THE MEMBER

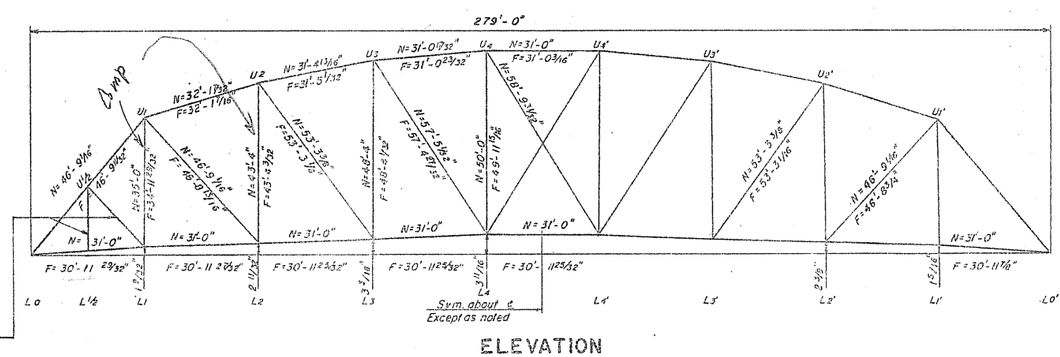
△ MAXIMUM WHEN END BEARING MOVED TO L1/2

⊕ ALLOWING 10% OVERSTRESS



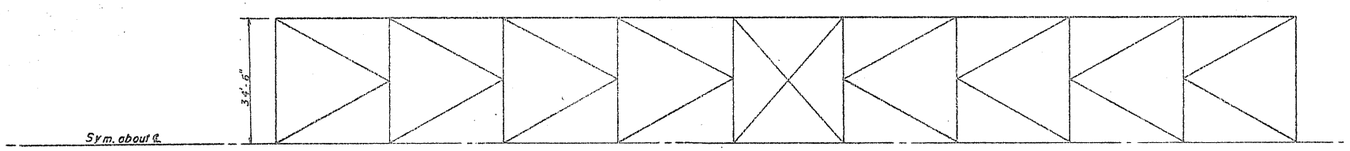






THESE TWO MEMBERS, ONE END OF TRUSS ONLY

ELEVATION



NOTES:

L0' end of each truss shall be located on Pier No. 7.

The top figure on each truss member is its normal length, with no stress and bottom chord horizontal.

The bottom figure is the adjusted length to which the member shall be fabricated to account for dead load deflection, except that a deduction of 1/16" for draw shall be made in the center diagonals.

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

279 FT. TRUSSES

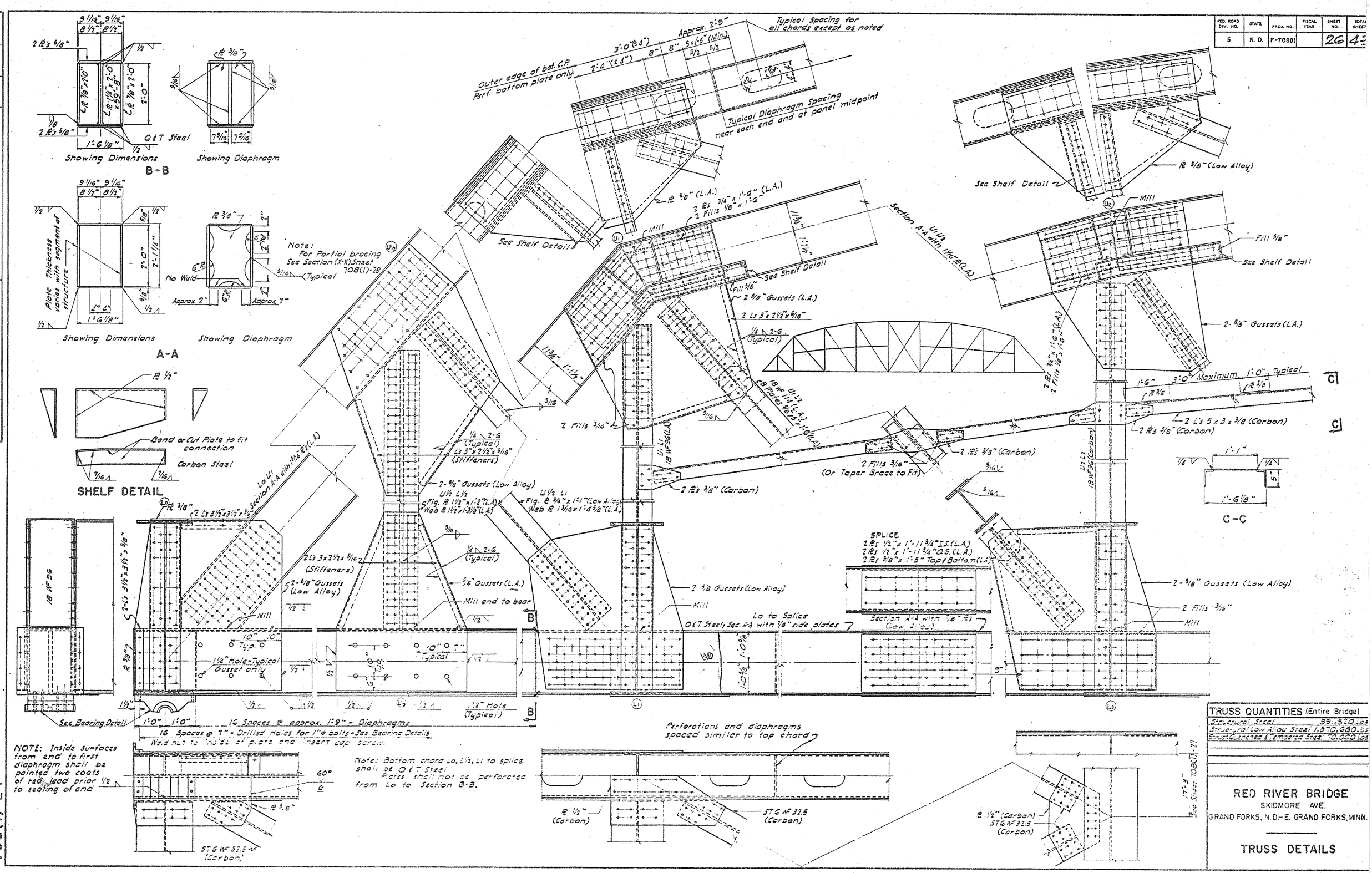
STRESSES AND CAMBER

TRACING CHECKED BY / MADE BY / CHECKED BY
QUANTITIES MADE BY / CHECKED BY

708(1)-23

708(1)-23

FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	F-7080)		26	4[illegible]



TRUSS QUANTITIES (Entire Bridge)	
Structural Steel	89,370 Lbs
Structural Low Alloy Steel	1,370,680 Lbs
Structural Quenched & Tempered Steel	70,000 Lbs

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

TRUSS DETAILS

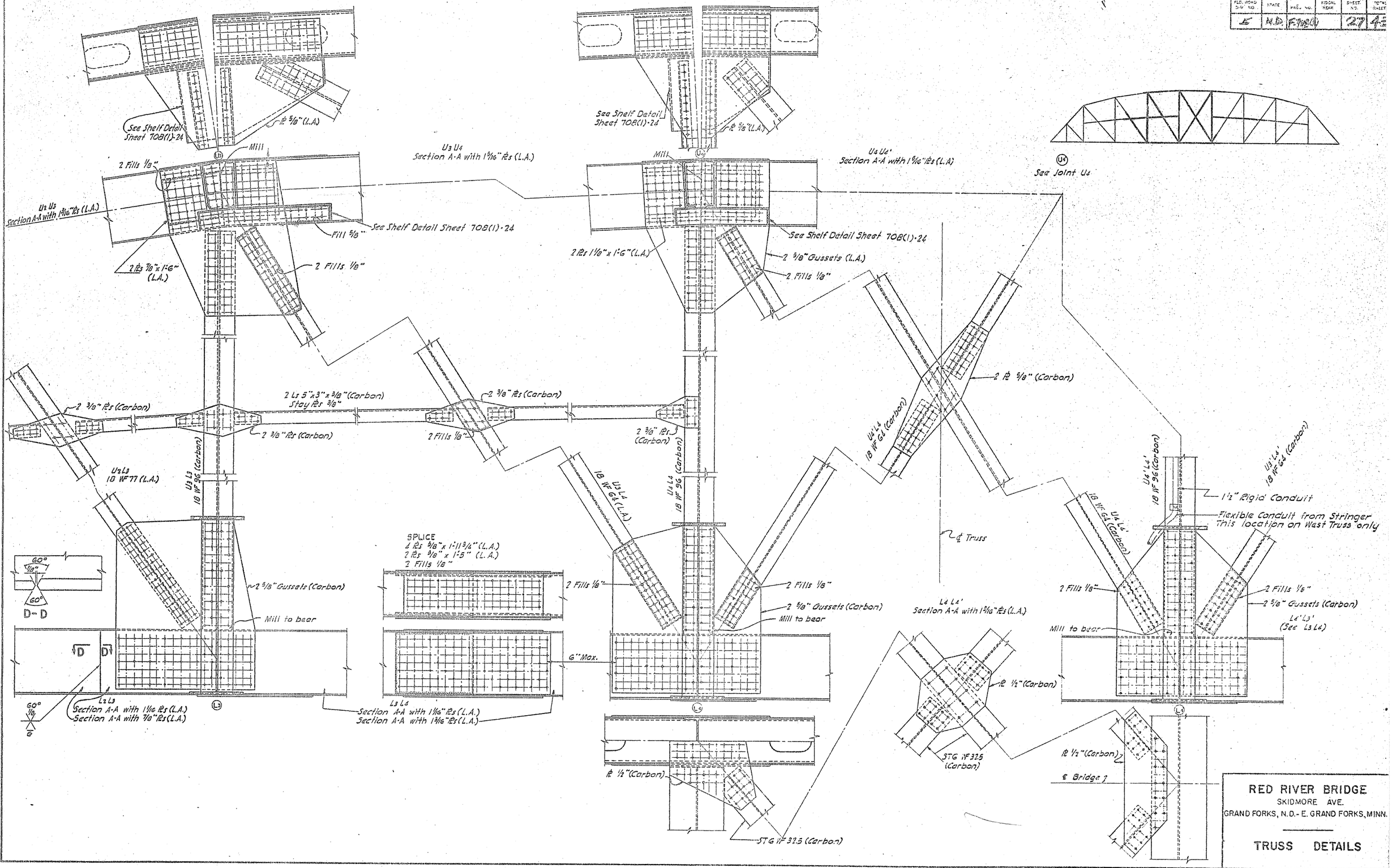
DETAILS	MADE BY	F.D.D.
	CHECKED BY	D.R.L.
TRACING	CHECKED BY	R.R.L.
QUANTITIES	MADE BY	F.D.D.
	CHECKED BY	

708(1)-24

708(1)-24

FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-108(1)		27	42

DETAILS	MADE BY	FDD
	CHECKED BY	DDB
TRACING	MADE BY	RHK & RRL
	CHECKED BY	
QUANTITIES	MADE BY	
	CHECKED BY	



RED RIVER BRIDGE

SKIDMORE AVE.

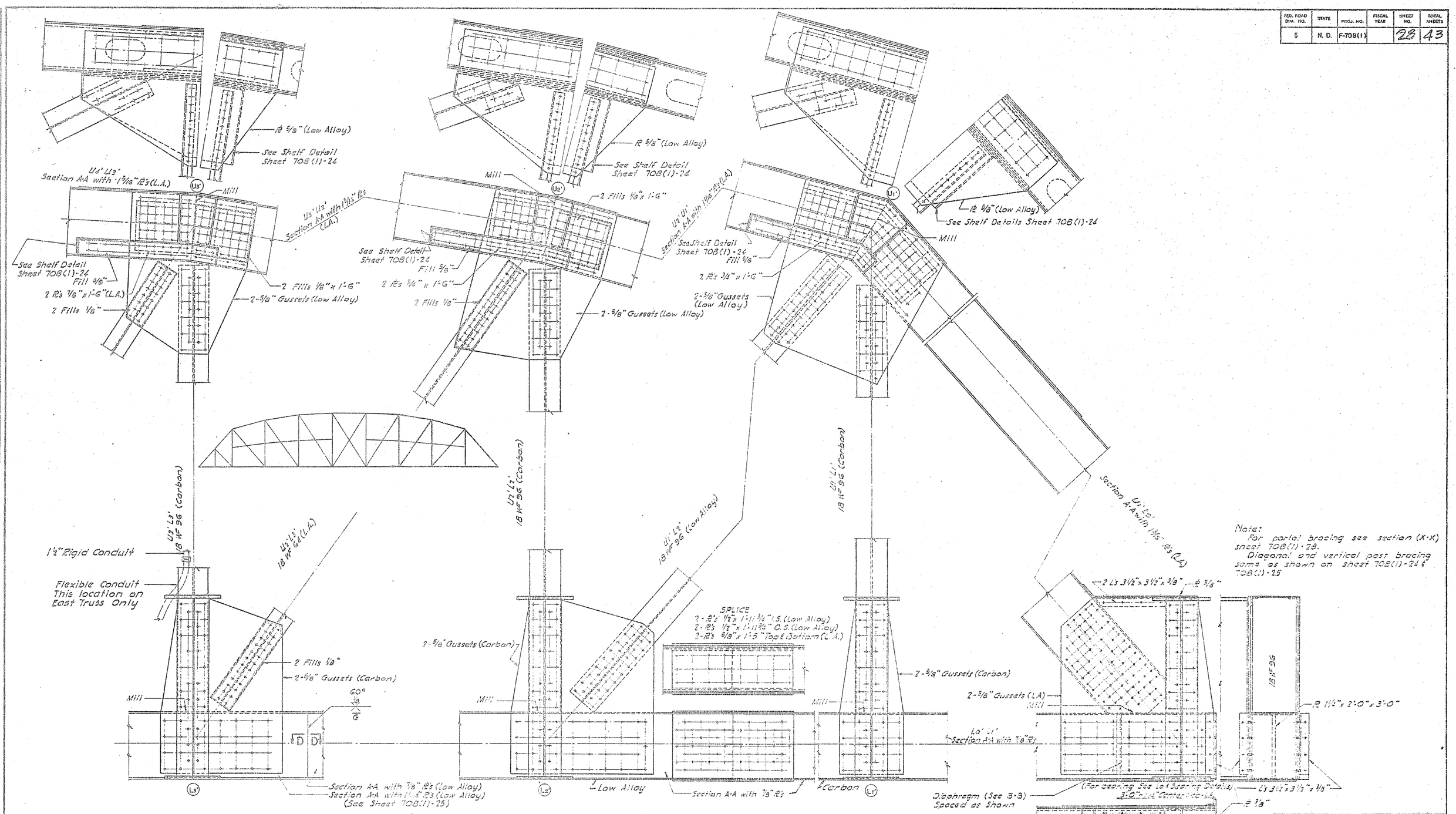
GRAND FORKS, N.D.- E. GRAND FORKS, MINN.

TRUSS DETAILS

708(1)-25

708(1)-25

FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	F-708(1)		28	43



TRACING	CHECKED BY
MADE BY	
CHECKED BY	
QUANTITIES	MADE BY
	CHECKED BY

RED RIVER BRIDGE

SKIDMORE AVE.

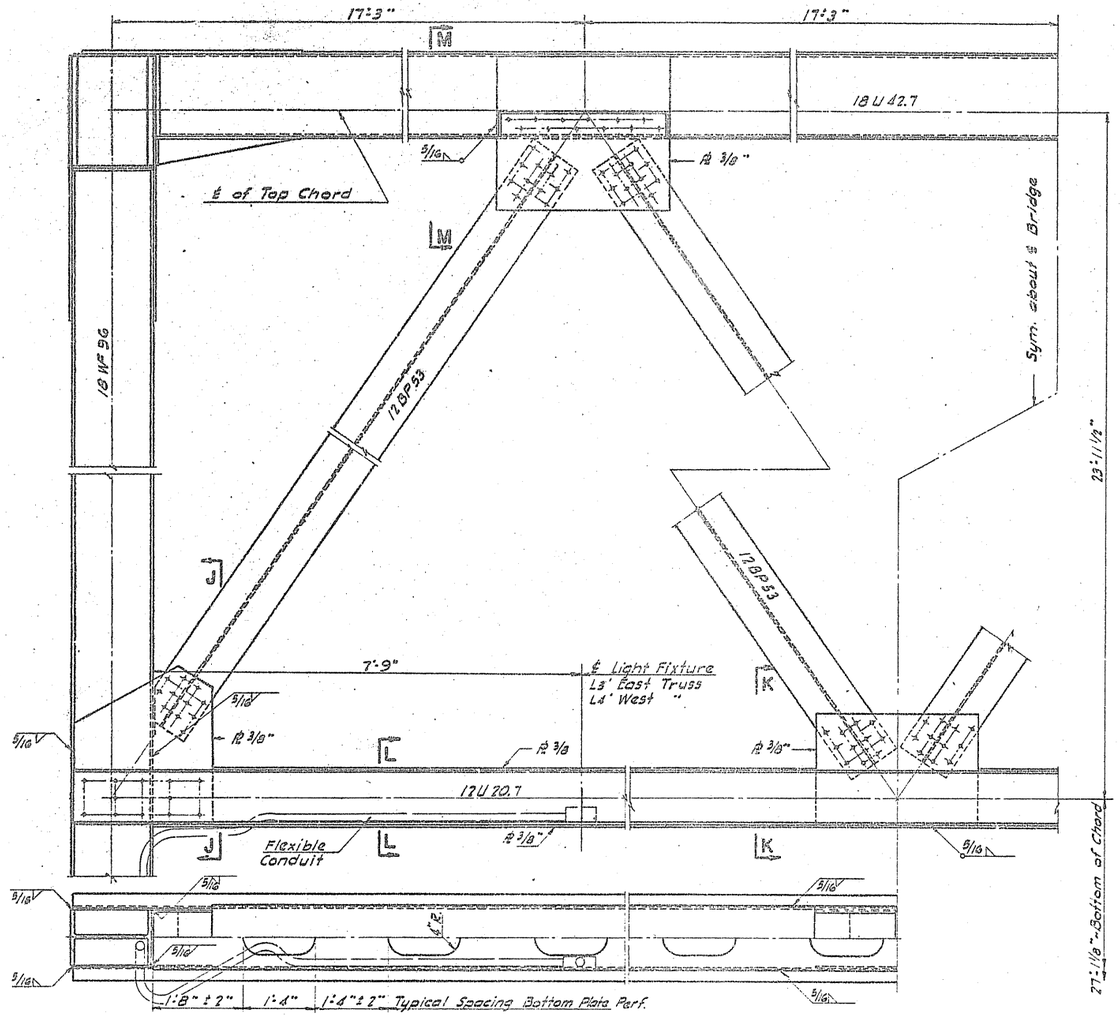
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

TRUSS DETAILS

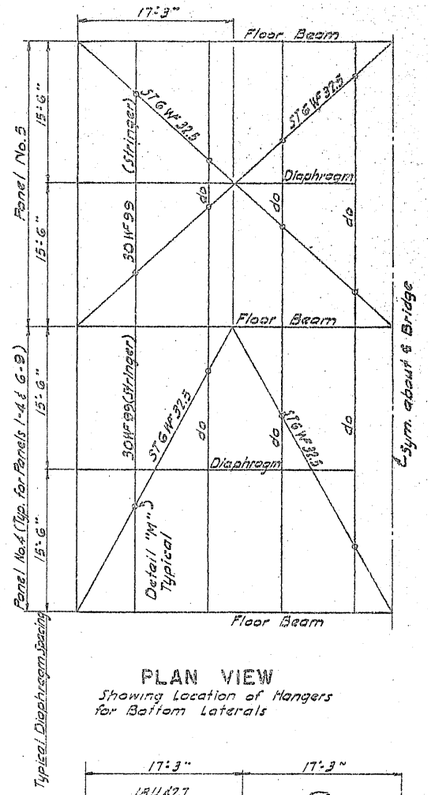
708(1)-26

708(1)-26

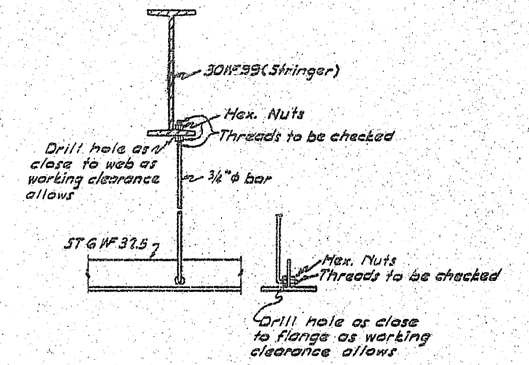
FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	F-708(1)		29	43



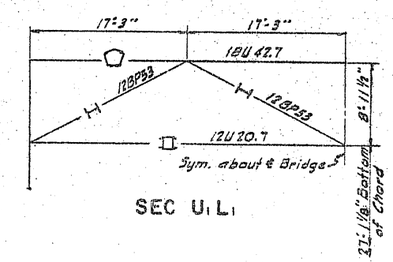
SWAY BRACING
(Section @ U_4L_4 Shown - Connections Typical)



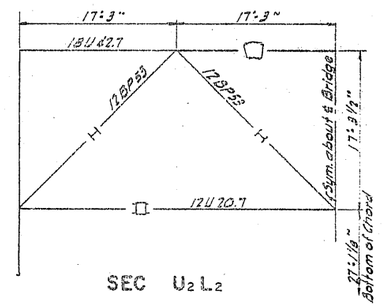
PLAN VIEW
Showing Location of Hangers for Bottom Laterals



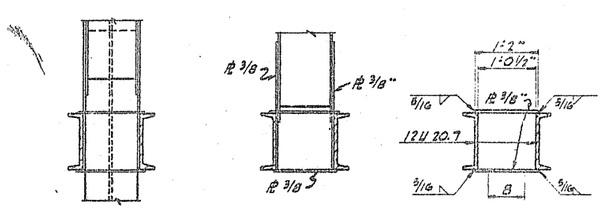
DETAIL "M"



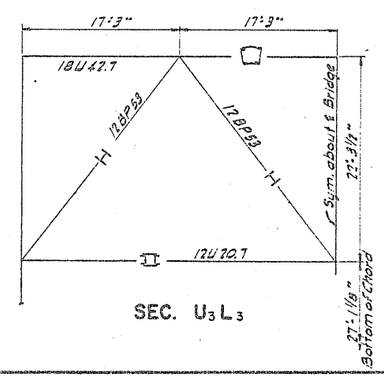
SEC U_1L_1



SEC U_2L_2



J-J K-K L-L



SEC. U_3L_3

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

SWAY BRACING

708(1)-27

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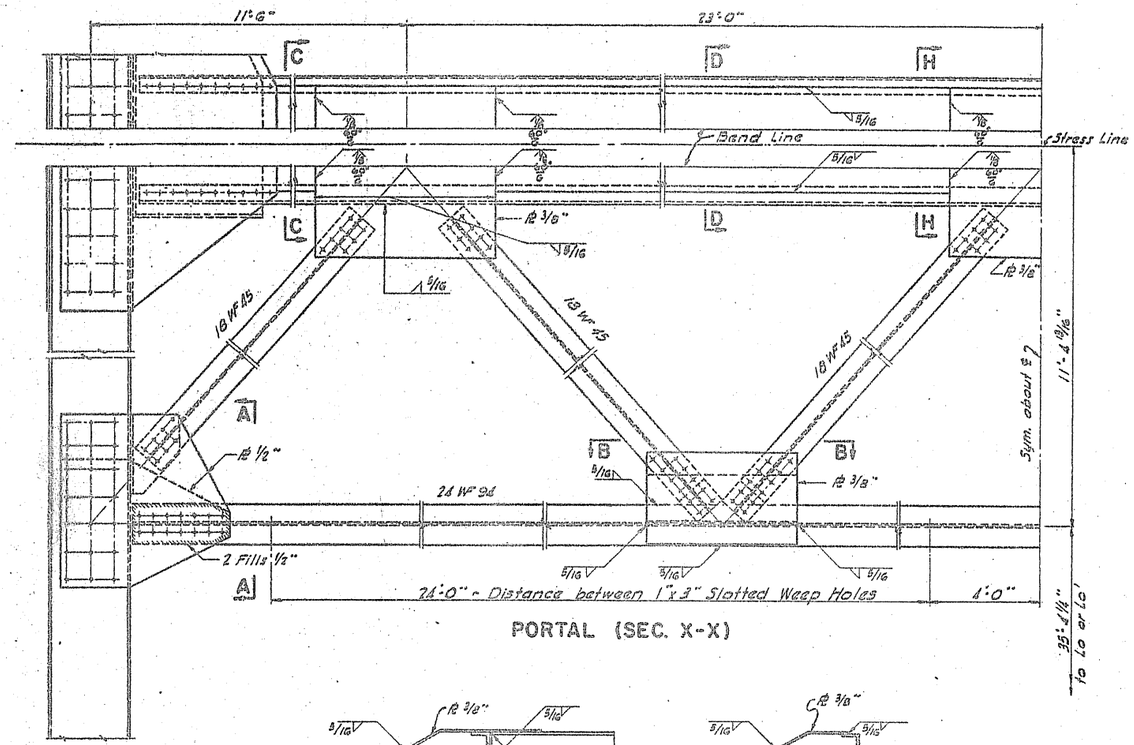
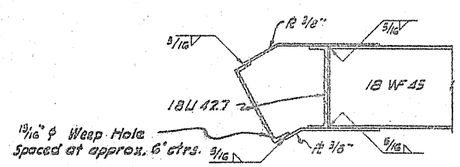
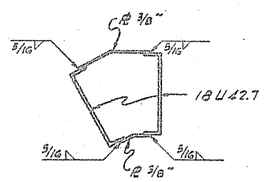
FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	F-708(1)		30	43



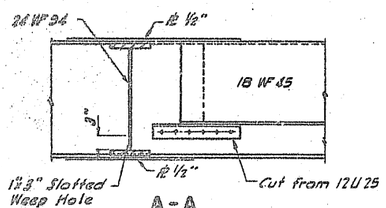
PORTAL (SEC. X-X)



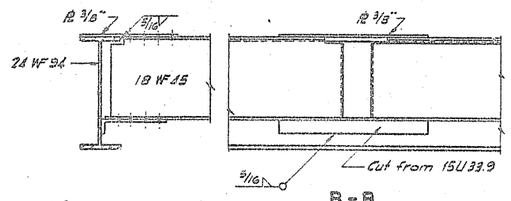
C-C



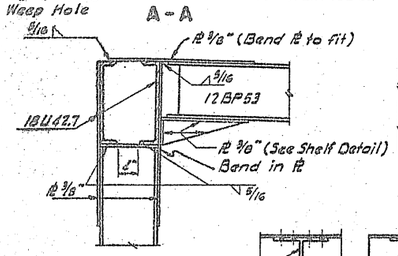
D-D
(Typical top Struct. Section)



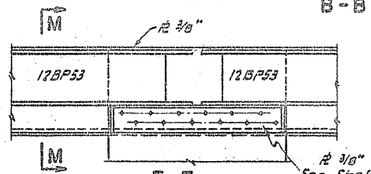
A-A



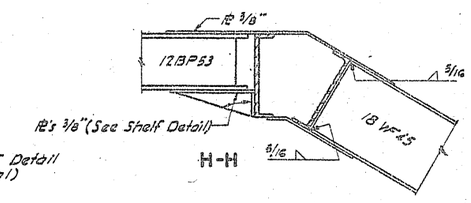
B-B



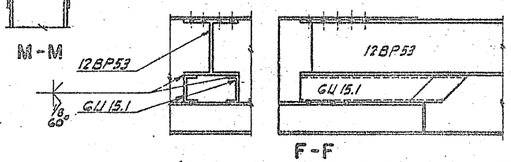
M-M



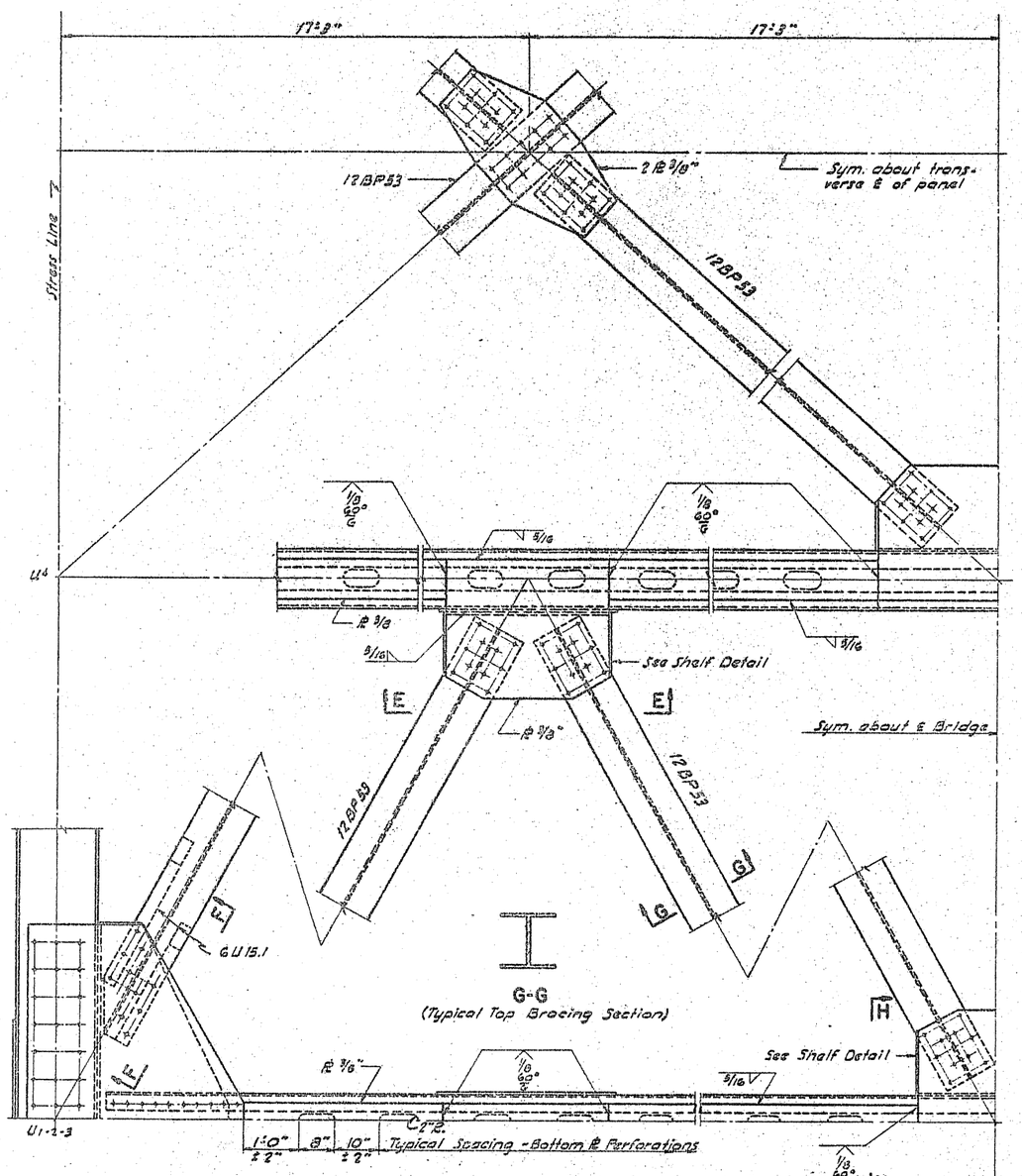
E-E



H-H



F-F
(Typical Top Bracing to Chord Connection)



G-G
(Typical Top Bracing Section)

PLAN
(Showing top bracing & connections)

RED RIVER BRIDGE
SKIDMORE AVE
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

PORTAL & TOP LATERAL TRUSS

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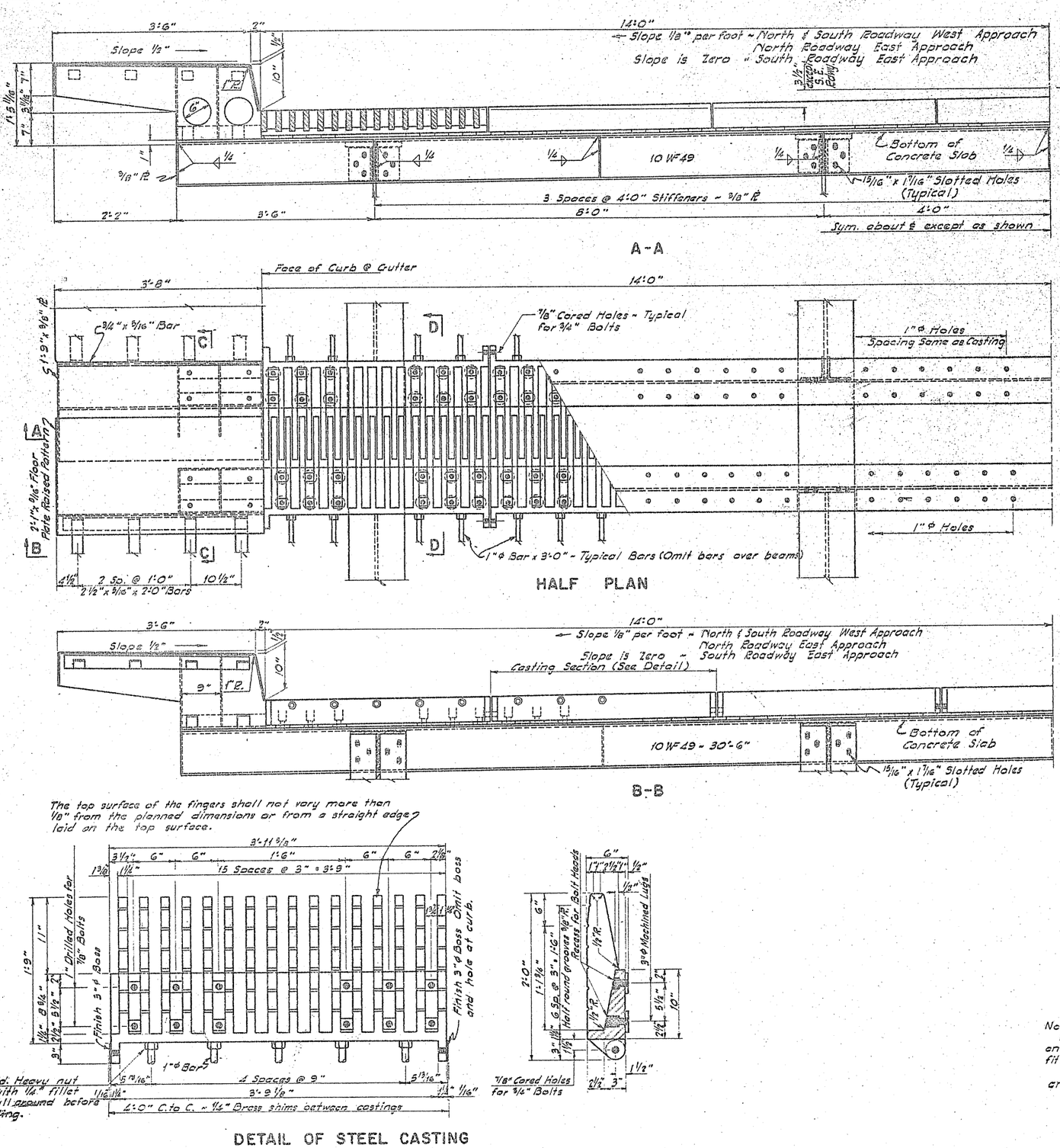
FED ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(1)		31	43



Note:
Each joint shall be completely shop assembled on its supporting beams and the parts properly fitted together.
Hot dip galvanize or metalize ends of beams and entire expansion device.

QUANTITIES

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

EXPANSION DEVICE

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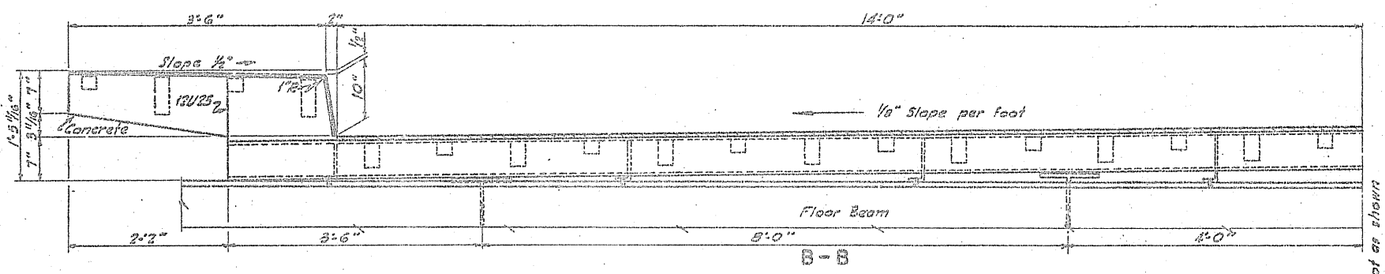
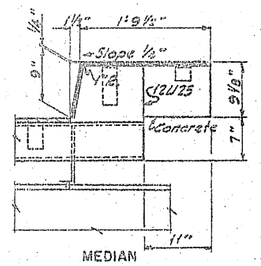
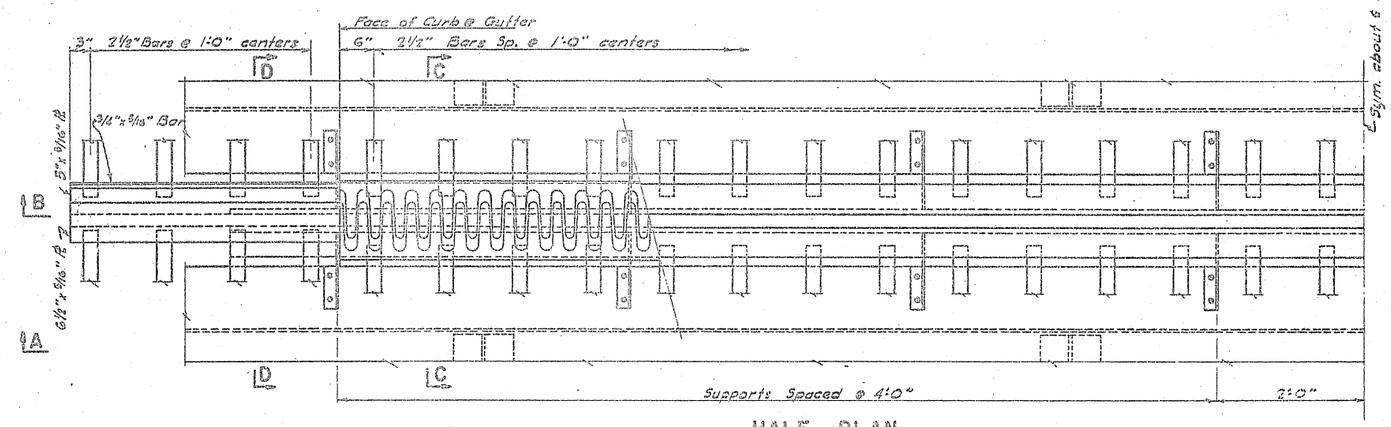
FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(1)		32	43



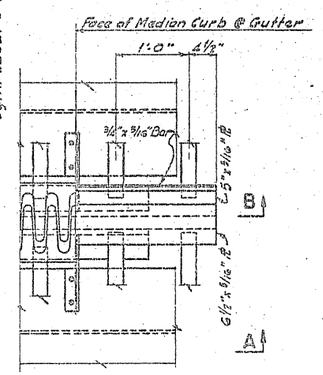
B - B



MEDIAN



HALF PLAN



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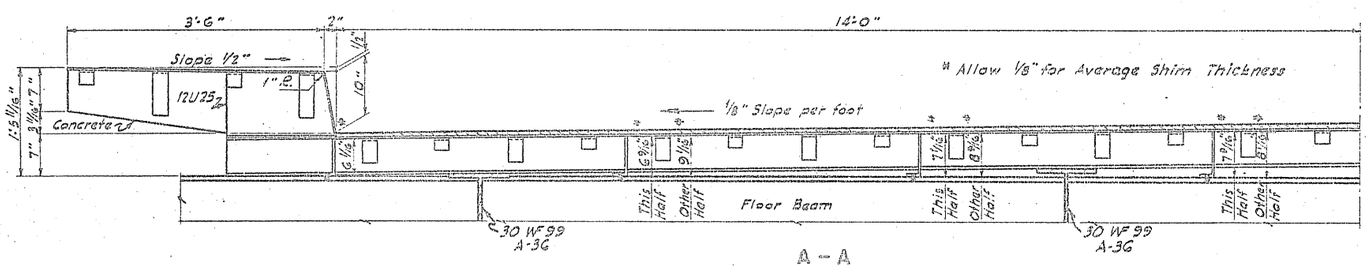
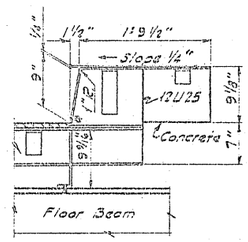
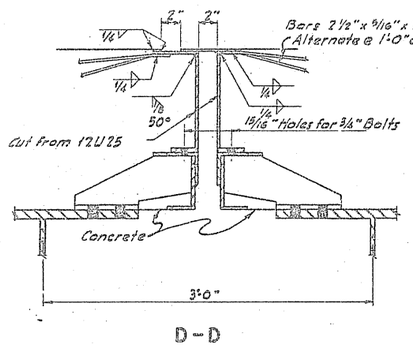


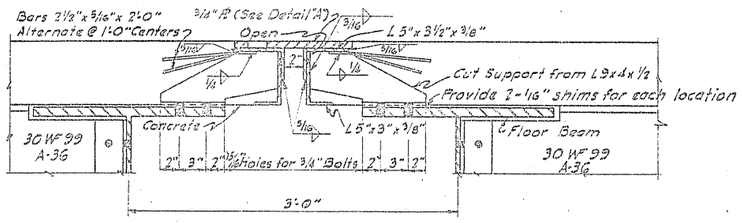
A - A



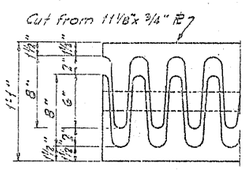
MEDIAN

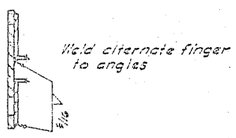


D - D



C - C





DETAIL "A"

QUANTITIES

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

SLAB JOINT AT PIER #7

708(1)-30

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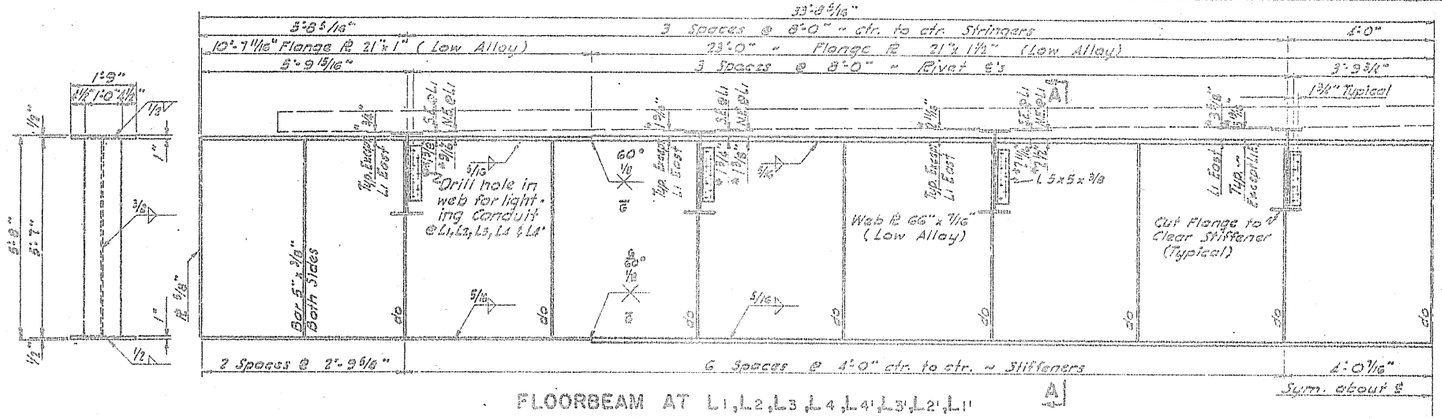
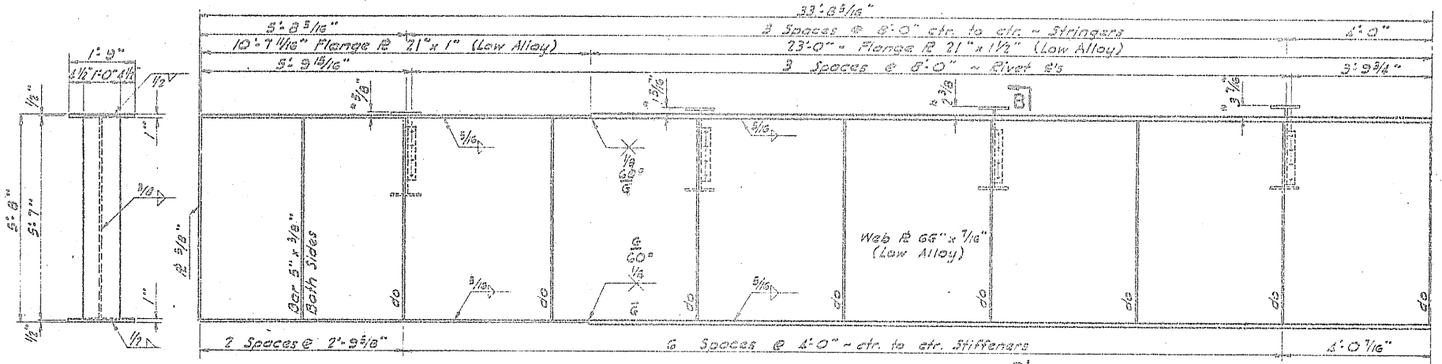
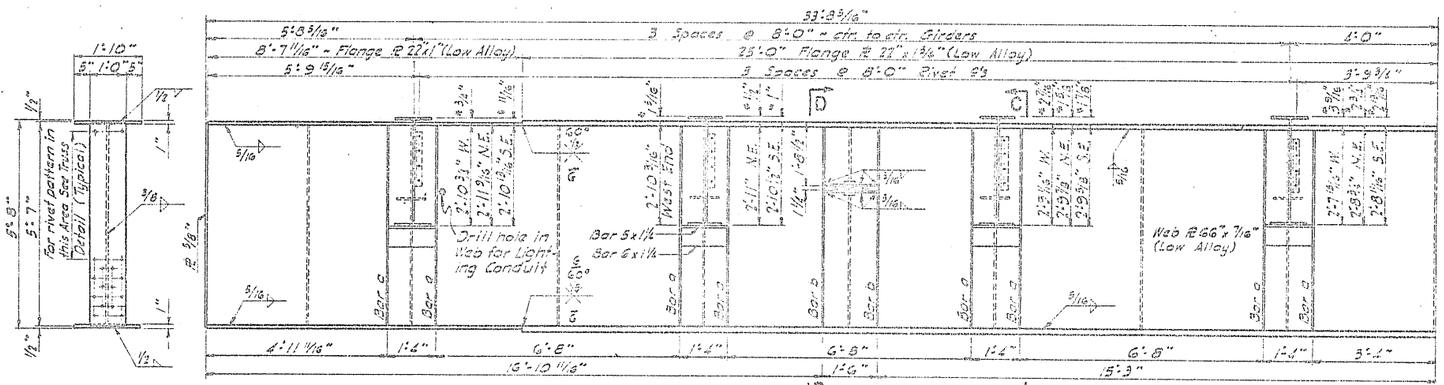
FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	P-708(1)		35	43



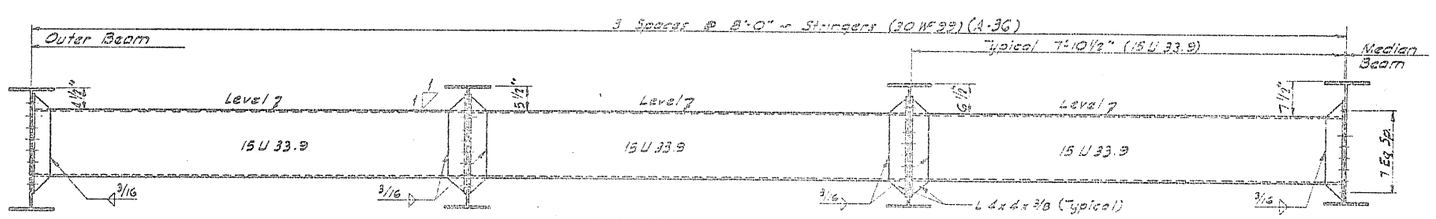
FLOORBEAM AT L1, L2, L3, L4, L4', L3', L2', L1'



FLOORBEAM AT L0'

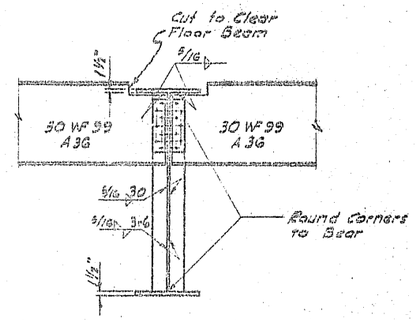


FLOORBEAM AT L0

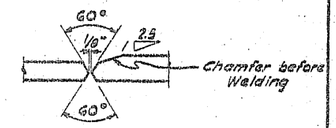


TRUSS FLOOR SYSTEM

Section Showing Diaphragms - Spaced @ Mid Panel - See Plan View Sheet 708(1)-27

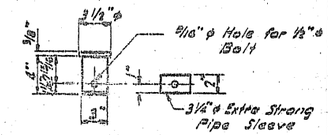


A — A

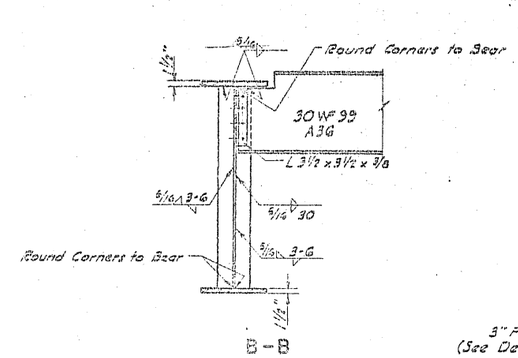


WELDING DETAIL

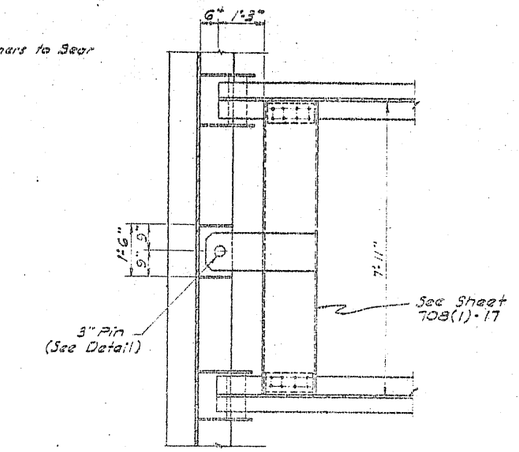
Shop Flange Splice



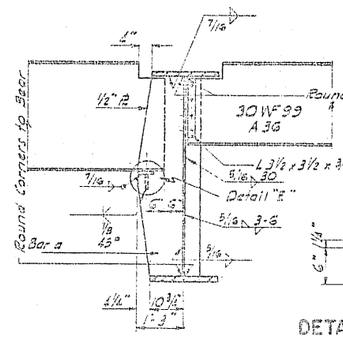
PIN DETAIL



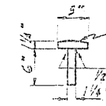
B - B



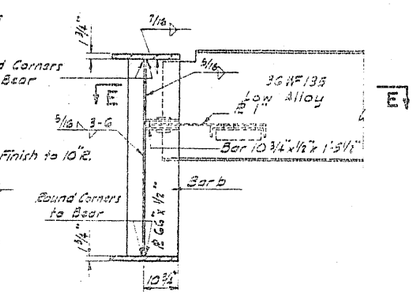
E - E



C - C



DETAIL "E"



D - D

* Includes Dead Load Deflections

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

FLOORBEAMS AND FLOOR SYSTEM DETAILS

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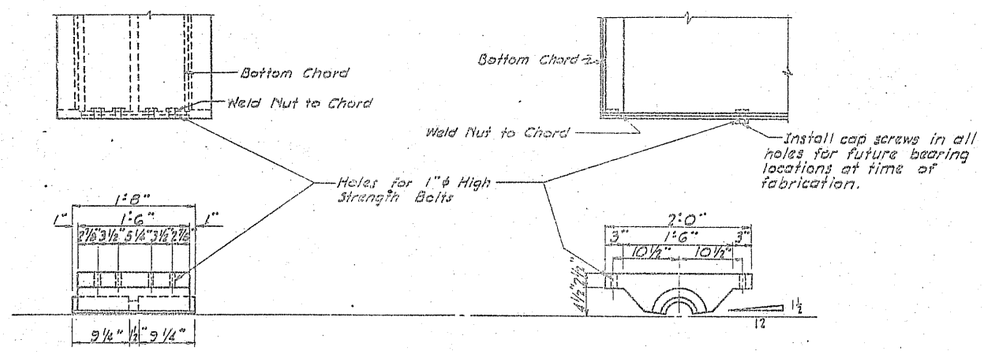
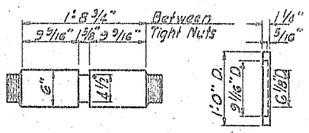
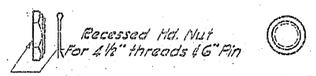
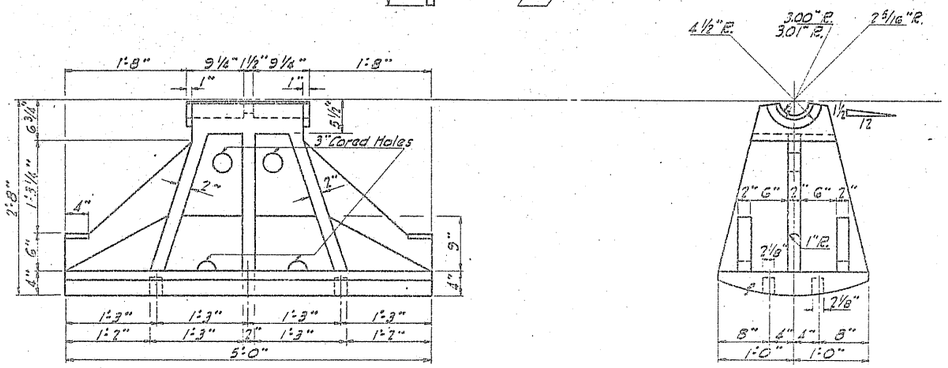
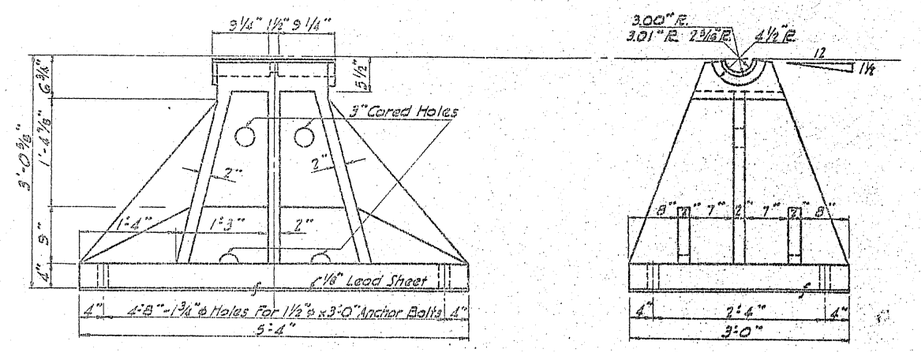
FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(1)		34	43











BEARING DETAILS RIVER PIER #7
(North Side Only)

MATERIALS:

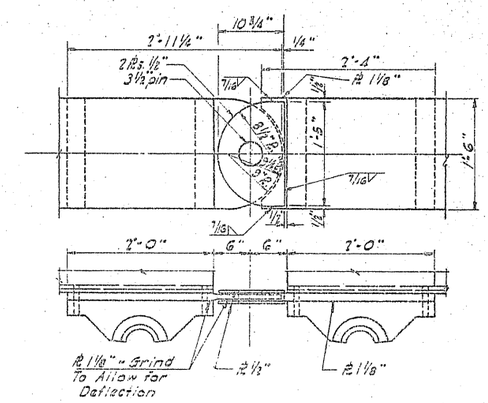
Masonry plates for expansion bearing shall be rolled steel plates.

Expansion rockers and fixed shoes shall be cast steel.

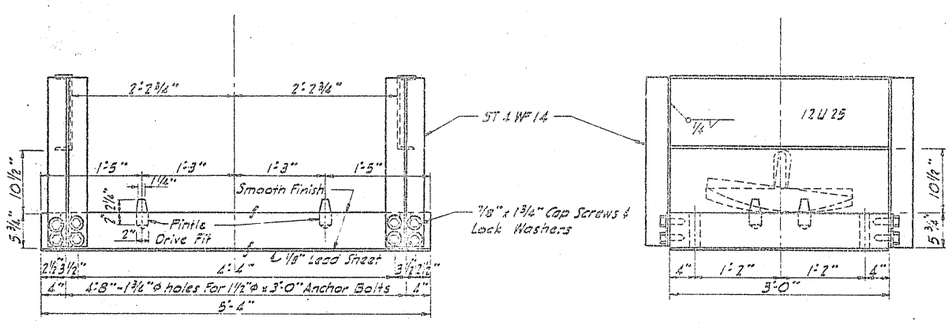
Top bolsters may be cast steel or cut from rolled steel plates. In either case the pay weight will be computed as castings.

Lead sheets shall be paid for by the pound as structural steel.

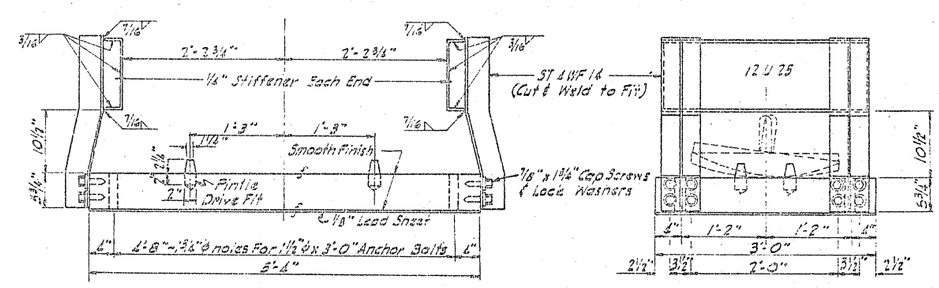
Cast steel shall conform to A.A.S.H.O. M-192, Class 70-36.



SWIVEL DETAILS RIVER PIER #7
(North Side Only)



BEARING DETAILS PIER #6 & #8



BEARING DETAILS PIER #7
(South Side Only)

QUANTITIES

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

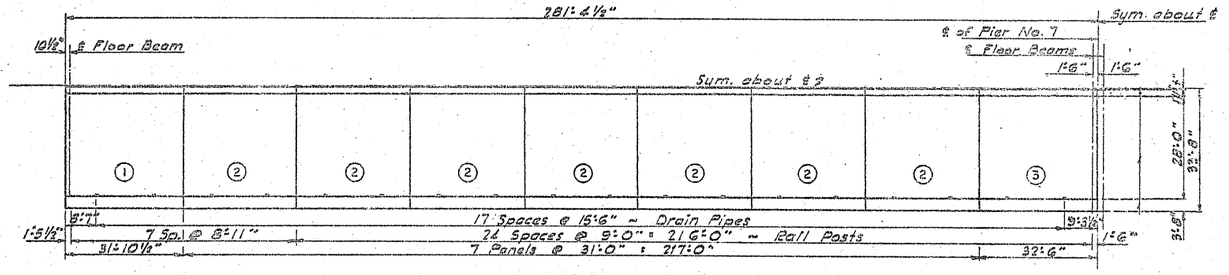
TRUSS BEARINGS

CHECKED BY
QUANTITIES MADE BY
CHECKED BY

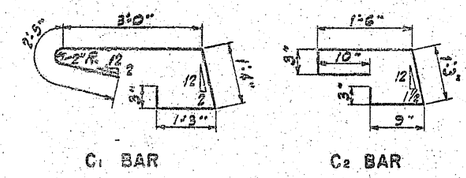
708(1)-32

708(1)-32

FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-7651(1)		35	43



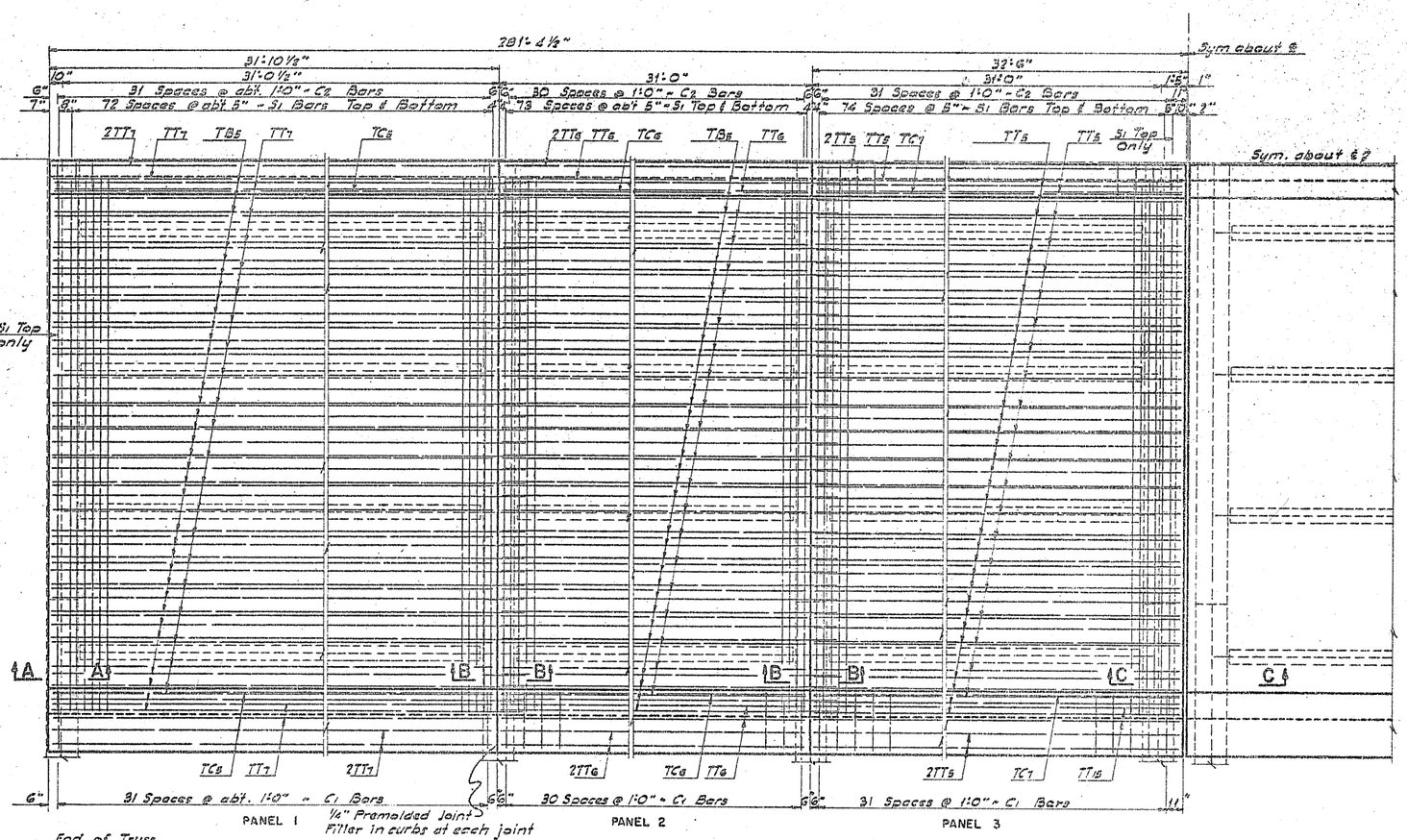
QUARTER DECK PLAN
SCALE 1" = 20'-0"



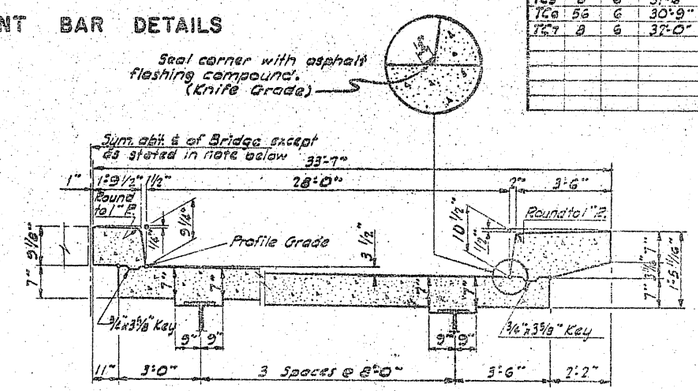
C1 BAR C2 BAR

BENT BAR DETAILS

BAR LIST				
MARK	NO.	SIZE	LENGTH	SHAPE
C1	1124	5	8'-3"	Bent
C2	1124	4	4'-10"	"
S1	5336	5	30'-2"	Str.
TBG	1824	4	32'-6"	Str.
TTG	116	4	32'-0"	"
TTG	812	4	30'-9"	"
TT7	116	4	31'-6"	"
TCG	8	6	31'-6"	"
TCG	56	6	30'-9"	"
TCT	8	6	32'-0"	"



QUARTER PARTIAL PLAN
SCALE ¼" = 1'-0"



SLAB SECTION
Showing Dimensions

Slab section showing reinforcing steel in truss is the same as the approach spans shown on sheet 708(1)-22.

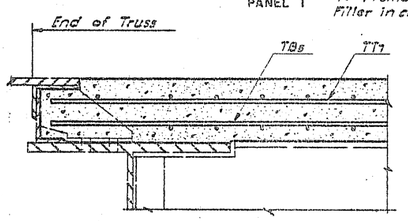
The south roadway from Sta. 1156+93.5 to Sta. 1157+93.5 shall be warped according to the profile grade and south gutter grade as shown on sheet 708(1)-22.

A camber of a ¼" shall be added to the profile grades at the mid-point of all stringers to account for dead load deflection.

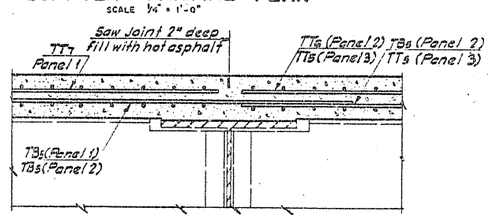
The truss floor slab, in width (30'-6") shall be poured continuously from end to end of truss.

Contraction joints, located at each floor beam, shall be sawed, preferably, within 18 hours of the time the concrete is poured, but not before the concrete has hardened to to the extent that tearing and ravelling is prevented.

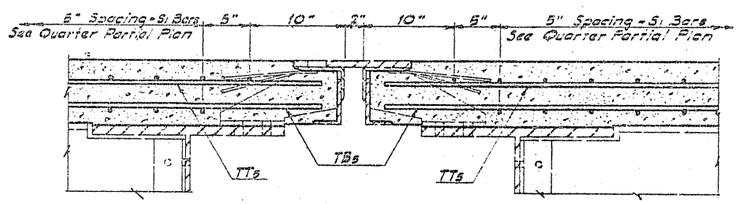
Note: For deck drains and quantities see sheet 708(1)-22



A-A



B-B



C-C

QUANTITIES	
Concrete Class AE-2	970.0 C.Y.
Reinforcing Steel	232,730 Lbs.

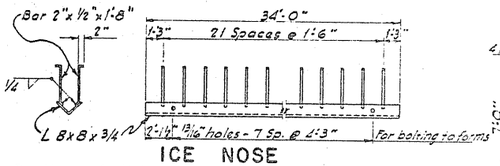
RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

TRUSS SLAB

QUANTITIES DE BY RES. HED BY AEC

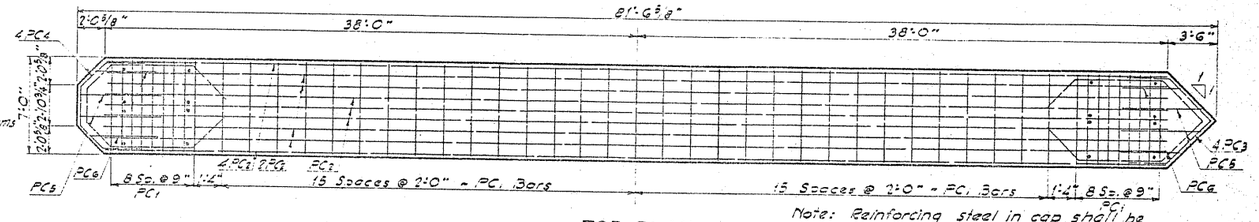
708(1)-33

FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(I)		36	43



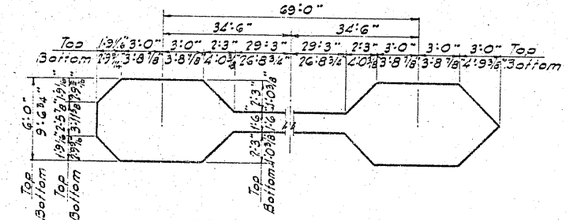
ICE NOSE

Ice Nose shall be Hot-Dip Galvanized after fabrication. It may be fabricated in two unspliced lengths of 18'-9" and 15'-9" to facilitate galvanizing.

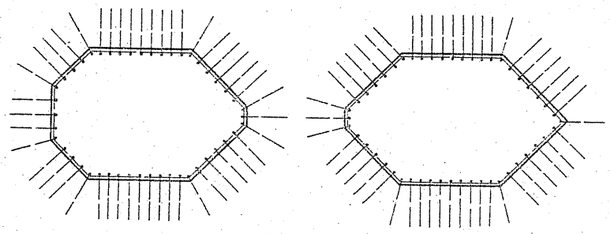


TOP PLAN

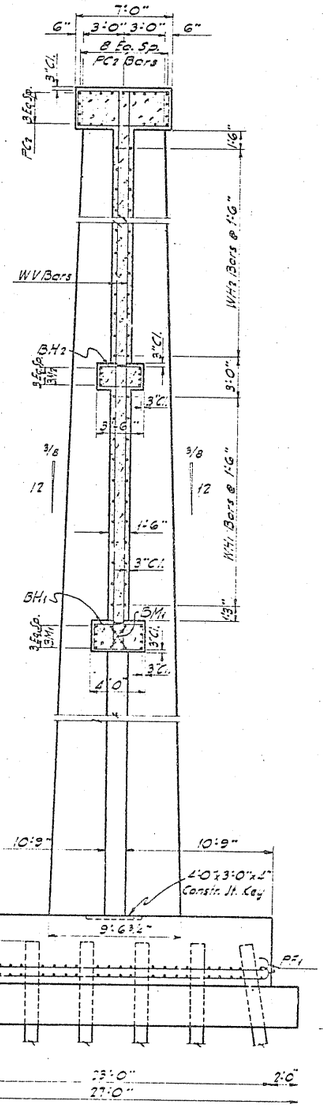
Note: Reinforcing steel in cap shall be adjusted to clear anchor bolts. (See Sheet 708(I)-32)



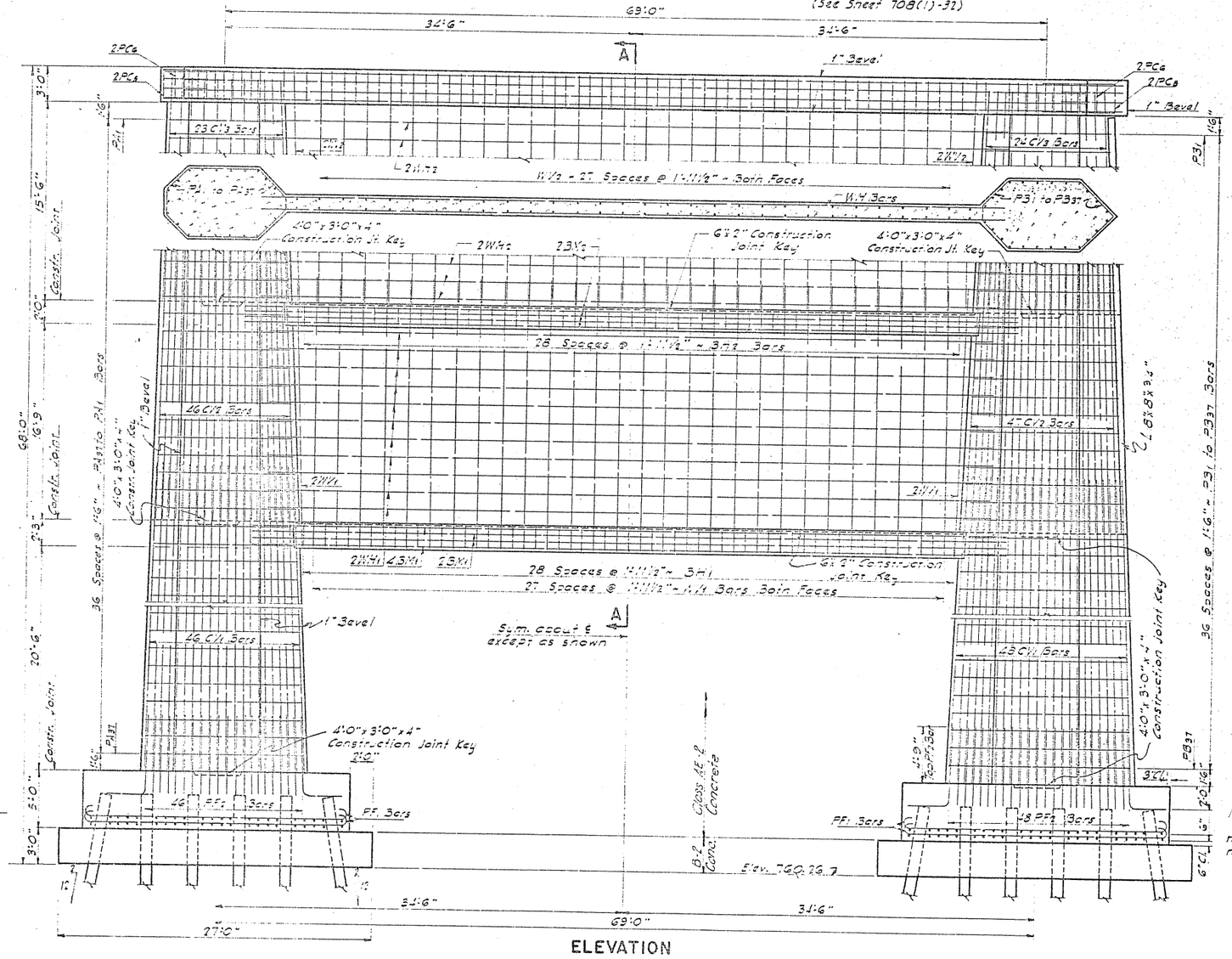
COLUMN DIMENSIONS



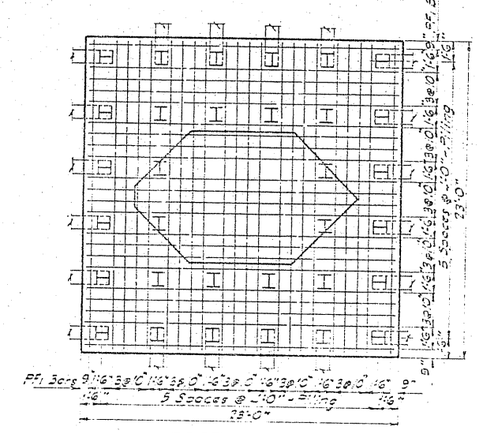
SECTION AT BASE OF COLUMNS



A-A



ELEVATION



FOOTING PLAN

Note:
Number of piles, location and maximum design loads per pile may be revised after load test.

QUANTITIES

Concrete Class 43-0	737.7	C.Y.
Concrete Class 31-2	162.0	C.Y.
Reinforcing Steel	75,100	Lbs.
Structural Steel	1,842	Lbs.

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

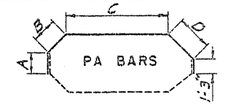
PIER NO. 7

TRACING | QUANTITIES
MADE BY CH. | JED BY | MADE BY | CHECKED BY

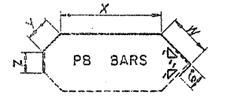
LIVE LOAD HS-20 | DEAD LOAD

708(I)-34

FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(I)		37	43



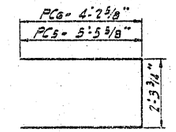
MARK	A	B	C	D	LENGTH
PA1	2'-2 1/2"	2'-3 1/2"	5'-9 3/4"	3'-0 1/4"	14'-7"
PA2	2'-2 3/4"	2'-4"	5'-10 1/4"	3'-1"	14'-9"
PA3	2'-3 3/8"	2'-2 1/8"	5'-10 5/8"	3'-1 7/8"	14'-11"
PA4	2'-3 3/8"	2'-2 1/8"	5'-11 1/8"	3'-2 5/8"	15'-1"
PA5	2'-3 5/8"	2'-3 1/8"	5'-11 5/8"	3'-3 3/8"	15'-3"
PA6	2'-3 7/8"	2'-3 7/8"	6'-0"	3'-4 1/4"	15'-5"
PA7	2'-4 1/4"	2'-4 1/4"	6'-0 1/2"	3'-5"	15'-7"
PA8	2'-4 1/2"	2'-4 3/4"	6'-1"	3'-5 3/4"	15'-9"
PA9	2'-4 5/8"	2'-7 1/4"	6'-1 1/2"	3'-6 5/8"	15'-11"
PA10	2'-5"	2'-7 3/4"	6'-1 7/8"	3'-7 3/8"	16'-1"
PA11	2'-5 3/8"	2'-8 1/8"	6'-2 3/8"	3'-8 1/8"	16'-3"
PA12	2'-5 1/2"	2'-8 5/8"	6'-2 7/8"	3'-9"	16'-5"
PA13	2'-5 3/4"	2'-9 1/8"	6'-3 3/8"	3'-9 3/4"	16'-7"
PA14	2'-6 1/8"	2'-9 1/2"	6'-3 3/4"	3'-10 5/8"	16'-9"
PA15	2'-6 3/8"	2'-10"	6'-4 1/4"	3'-11 3/8"	16'-11"
PA16	2'-6 3/8"	2'-10 1/2"	6'-4 3/4"	4'-0 1/8"	17'-1"
PA17	2'-6 3/4"	2'-11"	6'-5 1/4"	4'-1"	17'-3"
PA18	2'-7 1/4"	2'-11 3/8"	6'-5 5/8"	4'-1 3/4"	17'-5"
PA19	2'-7 1/2"	2'-11 7/8"	6'-6 1/8"	4'-2 1/2"	17'-7"
PA20	2'-6 5/8"	3'-0 3/8"	6'-6 5/8"	4'-3 3/8"	17'-8"
PA21	2'-7"	3'-0 3/4"	6'-7 1/8"	4'-4 1/8"	17'-10"
PA22	2'-7 3/8"	3'-1 1/4"	6'-7 1/2"	4'-4 7/8"	18'-0"
PA23	2'-7 1/2"	3'-1 3/4"	6'-8"	4'-5 3/4"	18'-2"
PA24	2'-7 3/4"	3'-2 1/4"	6'-8 1/2"	4'-6 1/2"	18'-4"
PA25	2'-8 1/8"	3'-2 5/8"	6'-8 7/8"	4'-7 3/8"	18'-6"
PA26	2'-8 3/8"	3'-3 1/8"	6'-9 3/8"	4'-8 1/8"	18'-8"
PA27	2'-8 5/8"	3'-3 5/8"	6'-9 7/8"	4'-8 7/8"	18'-10"
PA28	2'-8 3/4"	3'-4 1/8"	6'-10 3/8"	4'-9 3/4"	19'-0"
PA29	2'-9 1/4"	3'-4 1/2"	6'-10 3/4"	4'-10 1/2"	19'-2"
PA30	2'-9 1/2"	3'-5"	6'-11 1/4"	4'-11 1/4"	19'-4"
PA31	2'-9 3/8"	3'-5 1/2"	6'-11 3/4"	5'-0 1/8"	19'-6"
PA32	2'-10 1/8"	3'-5 7/8"	7'-0 1/8"	5'-0 7/8"	19'-8"
PA33	2'-10 3/8"	3'-6 3/8"	7'-0 5/8"	5'-1 3/8"	19'-10"
PA34	2'-10 1/2"	3'-6 7/8"	7'-1 1/8"	5'-2 1/2"	20'-0"
PA35	2'-10 3/4"	3'-7 3/8"	7'-1 5/8"	5'-3 1/4"	20'-2"
PA36	2'-11"	3'-7 3/4"	7'-2 1/8"	5'-4 1/8"	20'-4"
PA37	2'-11 3/8"	3'-8 1/4"	7'-2 1/2"	5'-4 7/8"	20'-6"
Total Length one Set 629'-1"					



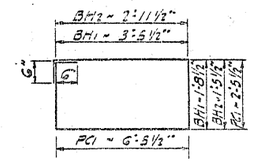
MARK	W	X	Y	Z	LENGTH
PB1	3'-11"	5'-9 3/4"	3'-0 1/4"	1'-3"	14'-6"
PB2	3'-11 3/4"	5'-10 1/4"	3'-1"	1'-3"	14'-8"
PB3	4'-0 3/8"	5'-10 5/8"	3'-1 7/8"	1'-2 7/8"	14'-10"
PB4	4'-1 5/8"	5'-11 1/8"	3'-2 5/8"	1'-2 7/8"	15'-0"
PB5	4'-2 1/4"	5'-11 3/8"	3'-3 3/8"	1'-2 3/4"	15'-2"
PB6	4'-3"	6'-0"	3'-4 1/4"	1'-2 3/4"	15'-4"
PB7	4'-3 3/4"	6'-0 1/2"	3'-5"	1'-2 3/4"	15'-6"
PB8	4'-4 5/8"	6'-1"	3'-5 3/4"	1'-2 5/8"	15'-8"
PB9	4'-5 3/8"	6'-1 1/2"	3'-6 5/8"	1'-2 1/2"	15'-10"
PB10	4'-6 1/8"	6'-1 7/8"	3'-7 3/8"	1'-2 3/8"	16'-0"
PB11	4'-7"	6'-2 3/8"	3'-8 1/8"	1'-2 1/2"	16'-2"
PB12	4'-7 3/4"	6'-2 7/8"	3'-9"	1'-2 1/8"	16'-4"
PB13	4'-8 1/2"	6'-3 3/8"	3'-9 3/4"	1'-2 3/8"	16'-6"
PB14	4'-9 3/8"	6'-3 3/4"	3'-10 5/8"	1'-2 1/4"	16'-8"
PB15	4'-10 1/8"	6'-4 1/4"	3'-11 3/8"	1'-2 1/4"	16'-11"
PB16	4'-11"	6'-4 3/4"	4'-0 1/8"	1'-2 1/8"	17'-1"
PB17	4'-11 3/4"	6'-5 1/4"	4'-1"	1'-3"	17'-3"
PB18	5'-0 1/2"	6'-5 5/8"	4'-1 3/4"	1'-3 1/8"	17'-5"
PB19	5'-1 3/8"	6'-6 1/8"	4'-2 1/2"	1'-3"	17'-7"
PB20	5'-2 1/8"	6'-6 5/8"	4'-3 3/8"	1'-2 7/8"	17'-9"
PB21	5'-2 7/8"	6'-7 1/8"	4'-4 1/8"	1'-2 7/8"	17'-11"
PB22	5'-3 3/4"	6'-7 1/2"	4'-4 7/8"	1'-2 3/4"	18'-1"
PB23	5'-4 1/2"	6'-8"	4'-5 3/4"	1'-2 3/4"	18'-3"
PB24	5'-5 1/4"	6'-8 1/2"	4'-6 1/2"	1'-2 3/4"	18'-5"
PB25	5'-6 1/8"	6'-8 7/8"	4'-7 3/8"	1'-2 3/8"	18'-7"
PB26	5'-6 7/8"	6'-9 3/8"	4'-8 1/8"	1'-2 3/8"	18'-9"
PB27	5'-7 3/4"	6'-9 7/8"	4'-8 7/8"	1'-2 1/4"	18'-11"
PB28	5'-8 1/2"	6'-10 3/8"	4'-9 3/4"	1'-2 1/8"	19'-1"
PB29	5'-9 1/4"	6'-10 3/4"	4'-10 1/2"	1'-2 1/2"	19'-3"
PB30	5'-10 1/8"	6'-11 1/4"	4'-11 3/8"	1'-3 1/4"	19'-6"
PB31	5'-10 7/8"	6'-11 3/4"	5'-0 1/8"	1'-3 1/4"	19'-8"
PB32	5'-11 5/8"	7'-0 1/8"	5'-0 7/8"	1'-3 1/8"	19'-10"
PB33	6'-0 1/4"	7'-0 5/8"	5'-1 5/8"	1'-3 1/4"	20'-0"
PB34	6'-1 1/4"	7'-1 1/8"	5'-2 1/2"	1'-3 1/8"	20'-2"
PB35	6'-2"	7'-1 5/8"	5'-3 1/4"	1'-3 1/8"	20'-4"
PB36	6'-2 7/8"	7'-2 1/8"	5'-4 1/8"	1'-2 7/8"	20'-6"
PB37	6'-3 3/4"	7'-2 1/2"	5'-4 7/8"	1'-2 7/8"	20'-8"
Total Length one Set 650'-1"					

BAR LIST

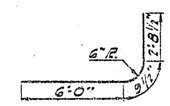
MARK	NO.	SIZE	LENGTH	SHAPE
PF1	176	11	25'-0"	Bent
PF2	94	11	9'-6"	"
CV1	96	11	27'-6"	Str.
CV2	93	11	23'-6"	"
CV3	47	11	18'-0"	"
BH1	29	5	11'-4"	Bent
BH2	29	5	9'-10"	"
BM1	24	8	33'-3"	Str.
BM2	16	8	34'-3"	"
WV1	60	5	22'-9"	Str.
WV2	60	5	18'-3"	Str.
WH1	44	5	31'-6"	Str.
WH2	42	5	32'-6"	"
PC1	49	5	18'-10"	Bent
PC2	38	8	39'-9"	Str.
PC3	4	5	13'-6"	Bent
PC4	4	5	12'-3"	"
PC5	4	8	10'-9"	"
PC6	4	8	13'-3"	"
PA	23 sets	4	629'-1"	Bent
PB	23 sets	4	650'-1"	"



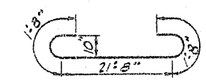
PC5-6



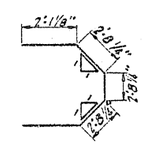
BH1-2, PC1



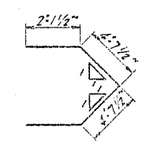
PF2



PF1



PC4



PC3

BENT BAR DETAILS

QUANTITIES

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.- E. GRAND FORKS, MINN

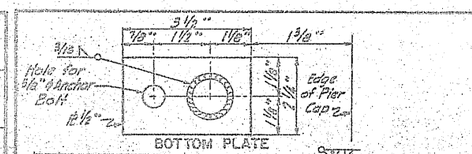
PIER NO. 7

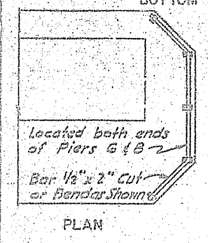
REINFORCING STEEL DETAILS

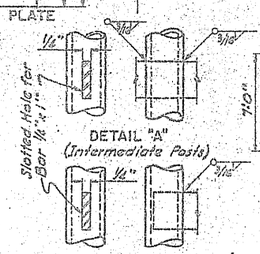
CHECKED BY	L.V.M.	
MADE BY	P.H.L. & R.R.L.	
CHECKED BY	W.E.G.	
TRACING		
QUANTITIES	MADE BY	
	CHECKED BY	

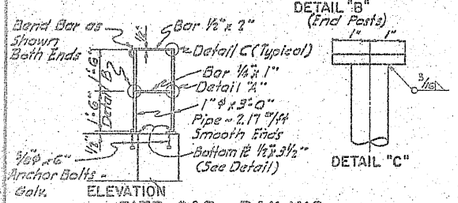
708(I)-35

FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(1)		38	43



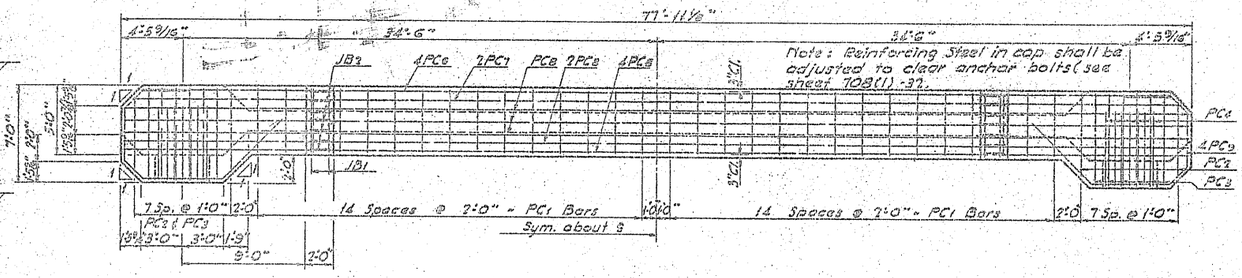




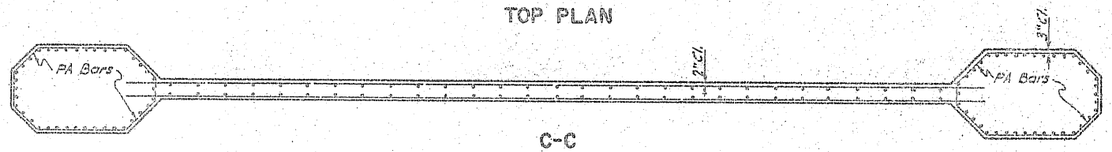


PIER CAP RAILING

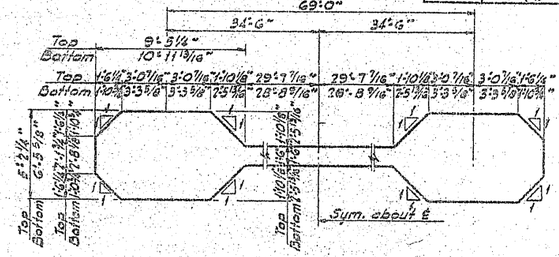
To be paid for as Structural Steel - 4 Rails



TOP PLAN

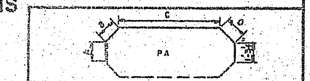


C-C

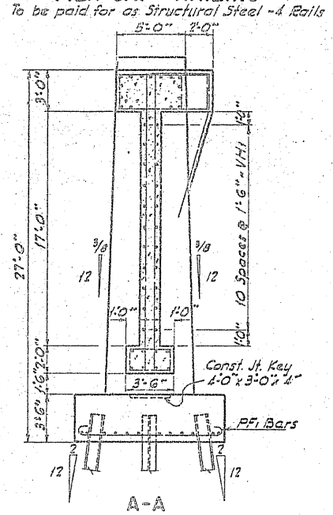


COLUMN DIMENSIONS

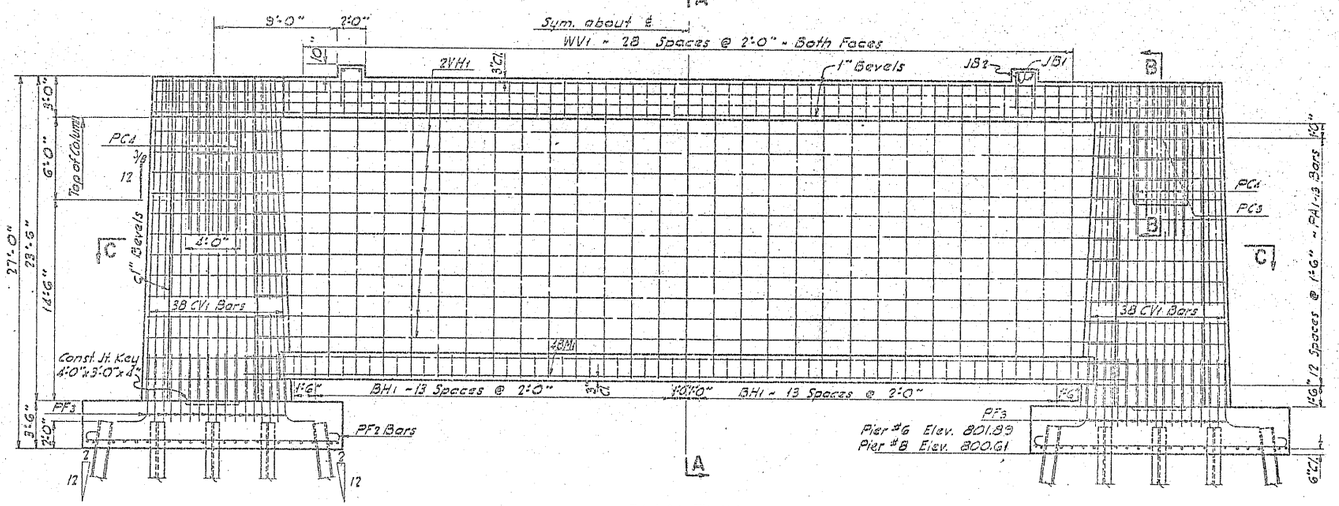
Below Cap



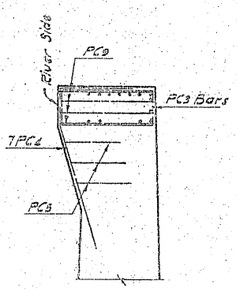
MARK	A	B	C	D	TOTAL
PA1	1'-10 3/4"	1'-11 1/8"	5'-10 1/4"	2'-5 1/8"	13'-5"
PA2	1'-11 1/8"	1'-11 1/8"	5'-10 1/8"	2'-5 1/8"	13'-7"
PA3	1'-11 1/8"	2'-0 1/8"	5'-11 3/8"	2'-6 1/8"	13'-9"
PA4	1'-11 3/8"	2'-0 3/4"	5'-11 3/4"	2'-7 1/8"	13'-11"
PA5	2'-0"	2'-1 1/4"	6'-0 1/4"	2'-8 1/8"	14'-1"
PA6	2'-1 1/8"	2'-1 3/4"	6'-0 5/8"	2'-9 1/8"	14'-3"
PA7	2'-1 5/8"	2'-2 3/8"	6'-1 1/8"	2'-9 7/8"	14'-5"
PA8	2'-1 7/8"	2'-2 5/8"	6'-1 1/2"	2'-10 3/4"	14'-8"
PA9	2'-2 1/8"	2'-3 1/8"	6'-2"	2'-11 1/2"	14'-10"
PA10	2'-2 5/8"	2'-3 1/2"	6'-2 3/8"	3'-0 1/4"	15'-0"
PA11	2'-2 3/4"	2'-4"	6'-2 7/8"	3'-1 1/8"	15'-2"
PA12	2'-3 1/8"	2'-4 1/2"	6'-3 1/4"	3'-1 7/8"	15'-4"
PA13	2'-3 1/2"	2'-5"	6'-3 3/4"	3'-2 3/4"	15'-6"



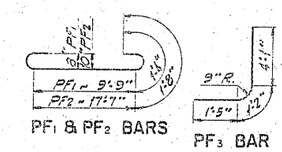
A-A

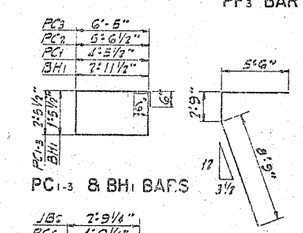


ELEVATION



B-B

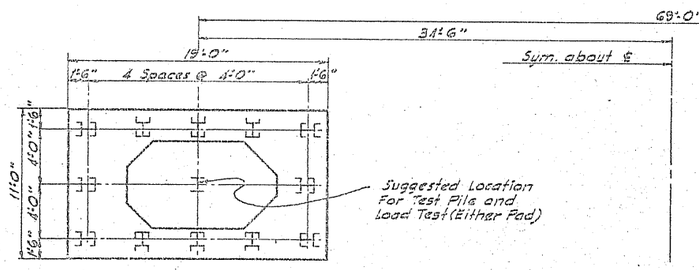




BENT BAR DETAILS

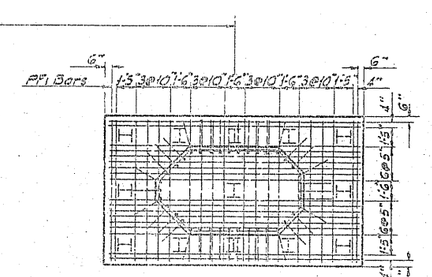
BAR LIST

MARK	NO	SIZE	LENGTH	SHAPE
PC1	40	8	13'-5"	Bent
PC2	36	11	70'-11"	"
PC3	76	8	6'-8"	"
CV1	76	8	23'-3"	Str.
BH1	30	5	9'-10"	Bent
BH1	16	8	34'-0"	Str.
WV1	58	5	21'-6"	Str.
WH1	44	5	32'-6"	Str.
PC1	30	5	14'-10"	Bent
PC2	4	8	17'-0"	"
PC3	12	8	18'-9"	"
PC4	14	6	17'-0"	"
PC5	6	6	11'-6"	"
PC6	8	8	39'-3"	Str.
PC7	4	8	39'-9"	"
PC8	18	8	40'-6"	"
PC9	8	8	22'-3"	Bent
JB1	4	4	4'-6"	Str.
JB2	14	5	7'-0"	Bent



FOOTING PLAN

Showing Dimensions



FOOTING PLAN

Showing Reinforcing

QUANTITIES

	ONE PIER	
Concrete Class AE-2	751.9	C.Y.
Reinforcing Steel	22,243	Lbs.
Structural Steel	140	Lbs.

RED RIVER BRIDGE

SKIDMORE AVE.

GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

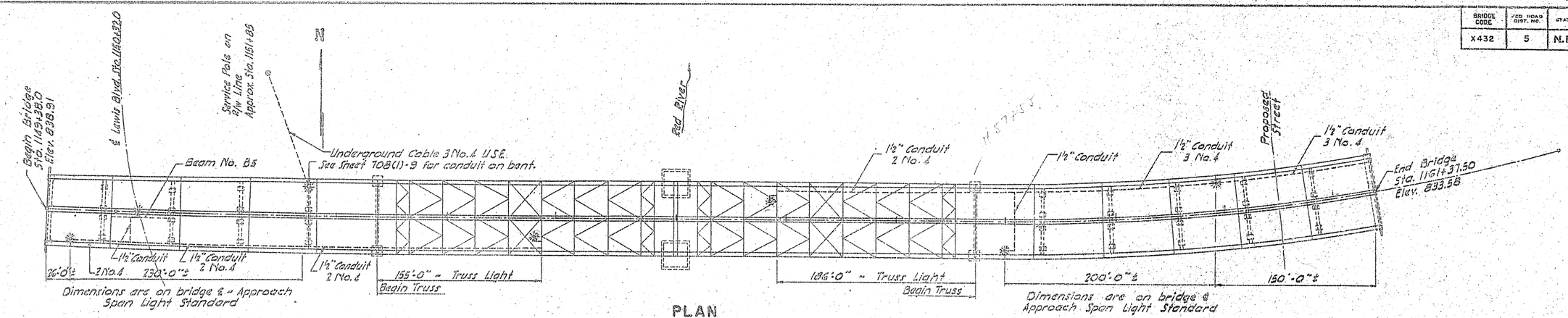
PIER NO. 6 & 8

TRACING	CHECKED BY	8-21-61
	MADE BY	8-21-61
QUANTITIES	CHECKED BY	8-21-61

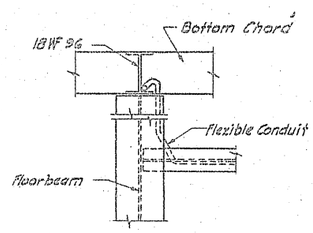
LIVE LOAD	3.68 T.
DESIGN LOAD	73.70 T.

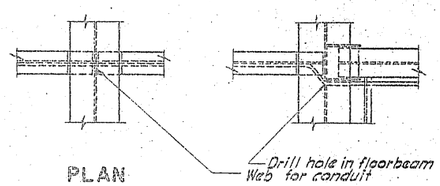
708(1)36

BRIDGE CODE	FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
X432	5	N.D.	F-708(1)		39	43

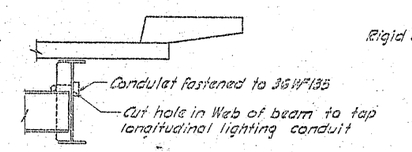


PLAN
(Showing Lighting Conduit)

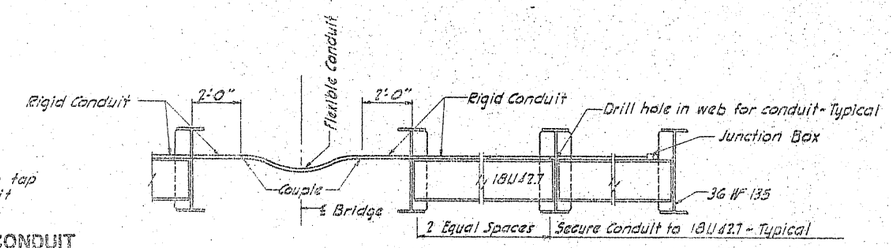




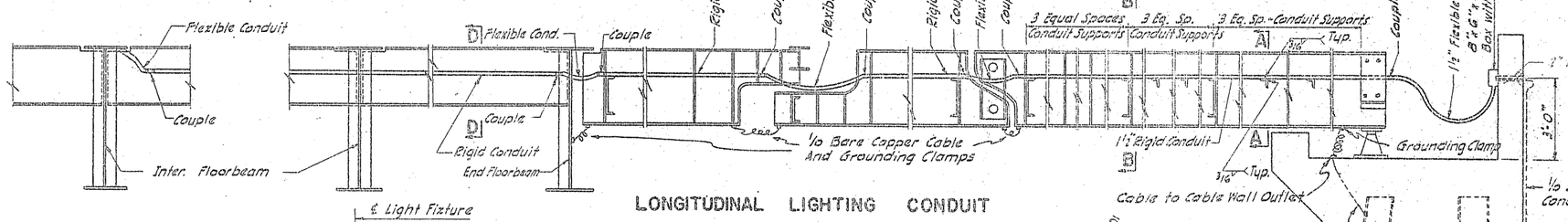
PLAN



OUTLET FROM LONG. LIGHTING CONDUIT
(For connection to post see sheet 708(1)-37A)



TRANSVERSE LIGHTING CONDUIT

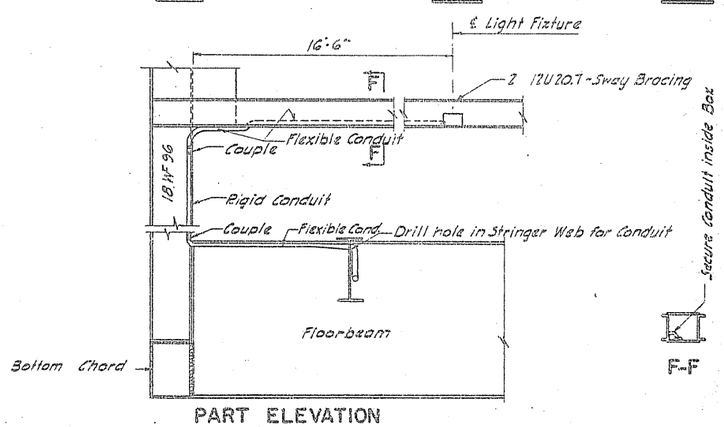


LONGITUDINAL LIGHTING CONDUIT

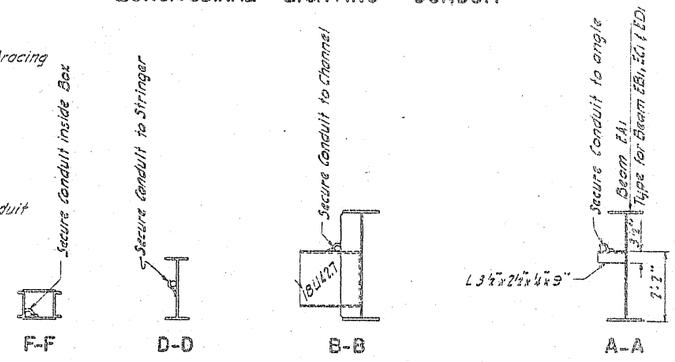
ESTIMATE OF MATERIALS

1	Service Pole
1	Photo-electric Controller
1	Meter Trough
1	Entrance Cabinet with Switch and Fuse Block
1	Ground Rod and Clamp (5/8" Diam. x 8' Long)
1	Entrance Cap
4	Lamp Std., Luminaires, Arms, Ballasts, etc. (400 watt)
2	Truss Luminaires, Arms, Ballasts, etc. (400 watt)
1	Luminaire Under Structure, Arm, Ballast, etc. (250 watt)
1	Junction Box (8 x 6 x 4)
4	Lamp Standard Supports
1200	L. F. 1½" Rigid Conduit
100	L. F. 1½" Flexible Conduit
100	L. F. Cable 3 No. 4 U.S.E.
300	L. F. Cable 1 No. 10 U.S.E.
2700	L. F. Cable 1 No. 4 U.S.E.
100	L. F. Stranded Bare Copper 1/0
	Misc. Clamps, Couplings, Junction Boxes, Bolts, Tape, etc.

The above estimate of quantities is only approximate. The cost of any additions necessary to complete the lighting system as described shall be included in the lump sum price bid for "Lighting System".



PART ELEVATION
(Showing Truss Light Outlet)



TYPICAL CONDUIT CONNECTIONS
(Maximum distance between anchorage - 6'-3")

QUANTITIES	
Lighting System	1 Lump Sum

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D. - E. GRAND FORKS, MINN.

LIGHTING LAYOUT

CHECKED BY
MADE BY
QUANTITIES CHECKED BY

708 (1)-37

Ce 19-03 708 (1)-37

FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
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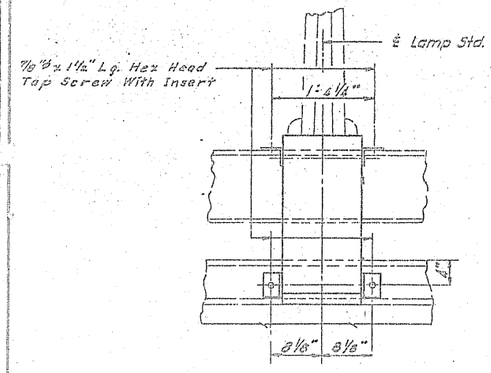
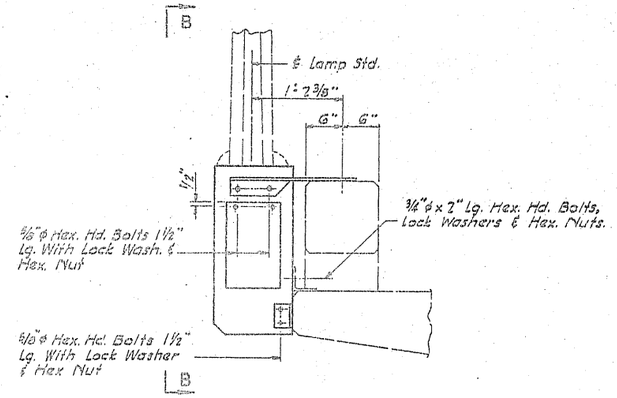
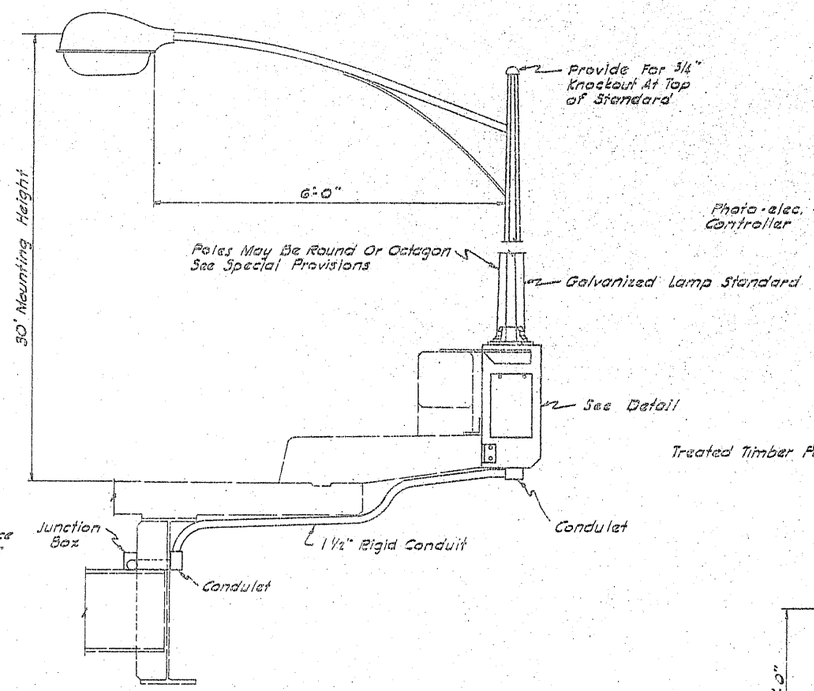


B-B

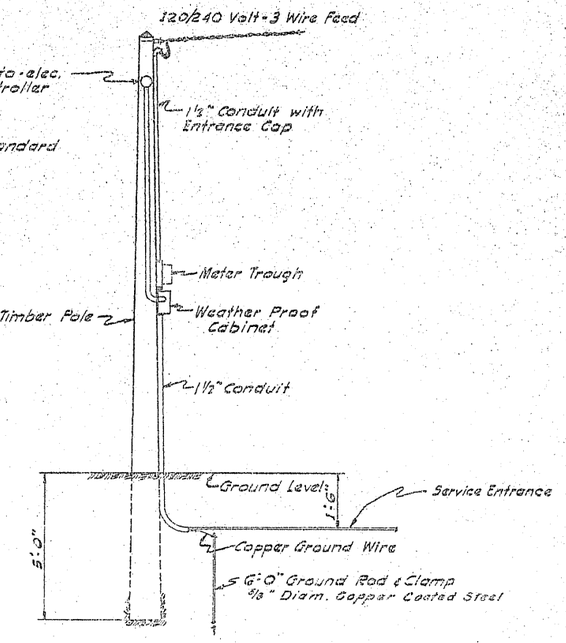


SIDE ELEVATION

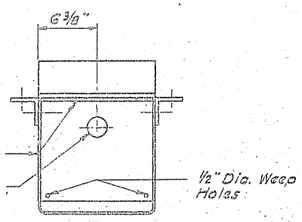


LIGHT DETAIL FOR APPROACH SPANS

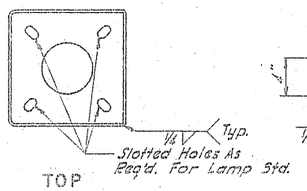
4 Required



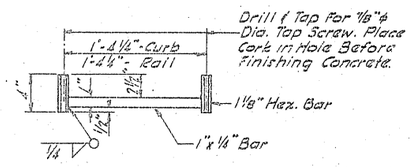
POWER FEEDER POINT



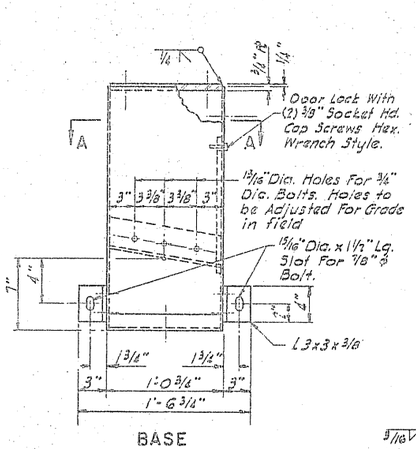
A-A



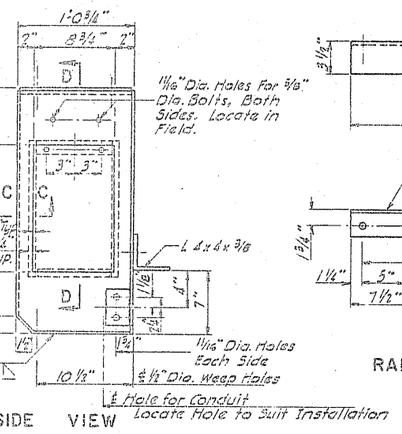
TOP



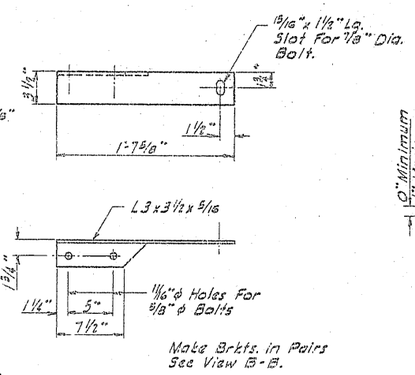
INSERT



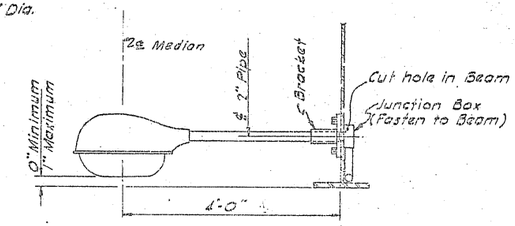
BASE



SIDE VIEW

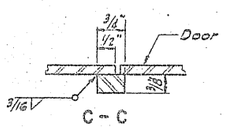


RAILING BRACKET

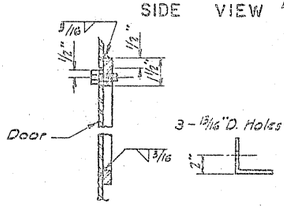


LIGHT UNDER STRUCTURE

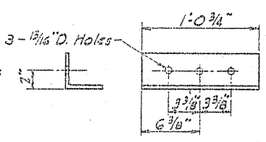
Light to be located over ℄ of Lewis Blvd.
Attach to Beam # 5.
1 Req'd



C-C



D-D



4 x 4 x 3/8 SEAT ANGLE

LAMP STANDARD SUPPORT DETAILS

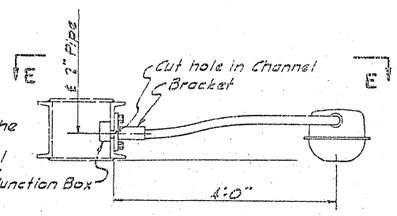
NOTES:

Bolts shall conform to Section 120.10 of the Std. Specs.

Plates, bars & angles shall be structural steel conforming to Section 119.10 of the Std. Specs.

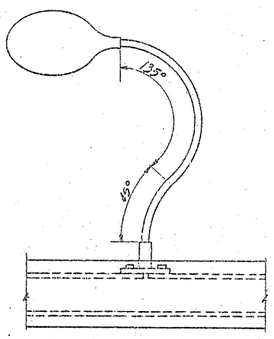
All structural steel and hardware to be galv. per A.S.T.M. D 123 & D 153.

Provide supports for ballast if ballast is to be placed in Lamp Standard Support.



LIGHT DETAIL FOR TRUSS

See Sheet 708(1)-31 for Location
2 Req'd



E-E

QUANTITIES

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

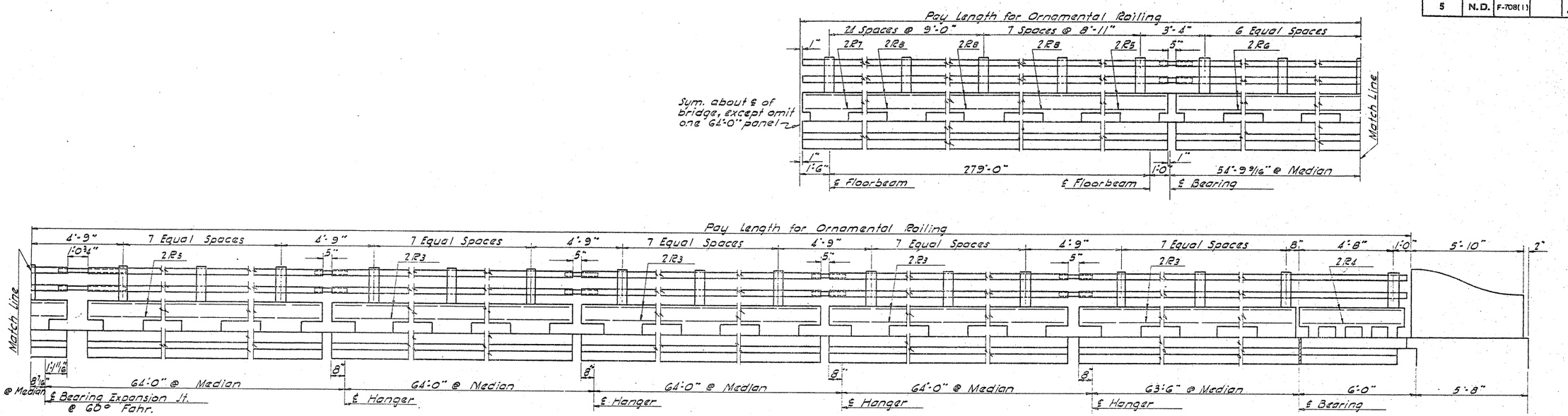
LIGHTING DETAILS

QUANTITIES MADE BY CHECKED BY

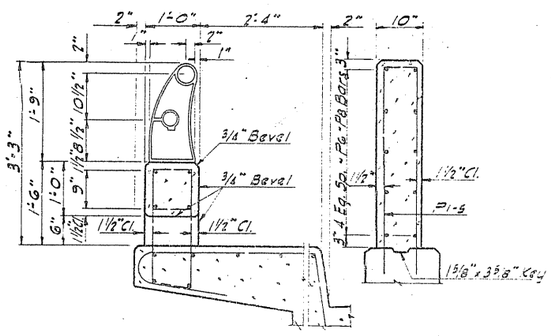
708(1)-37A

708(1)-37A

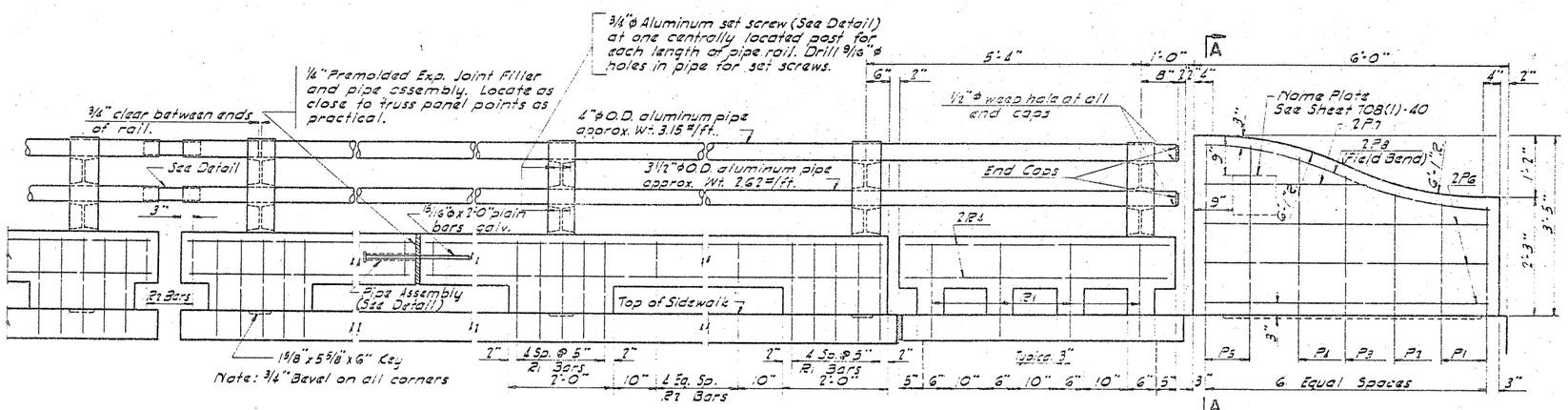
FED. ROAD DIST. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N.D.	F-708(11)		41	43



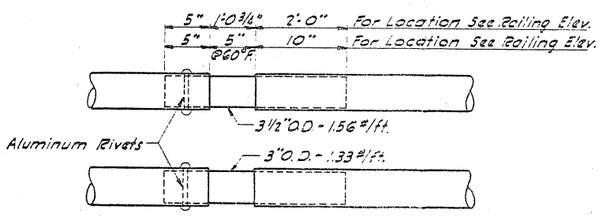
HALF ELEVATION



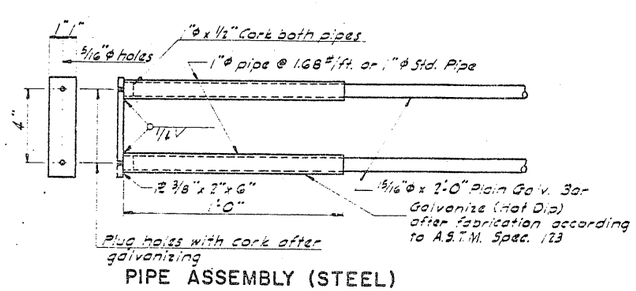
TYPICAL SECTION A-A



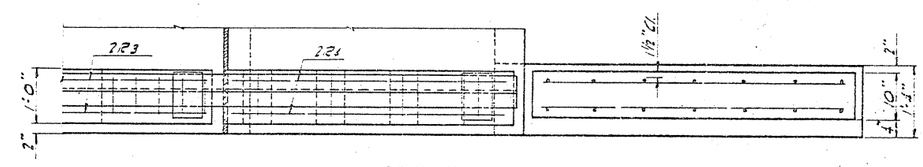
PART ELEVATION



EXPANSION JOINT IN RAIL



PIPE ASSEMBLY (STEEL)



PART PLAN

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

RAILING DETAILS

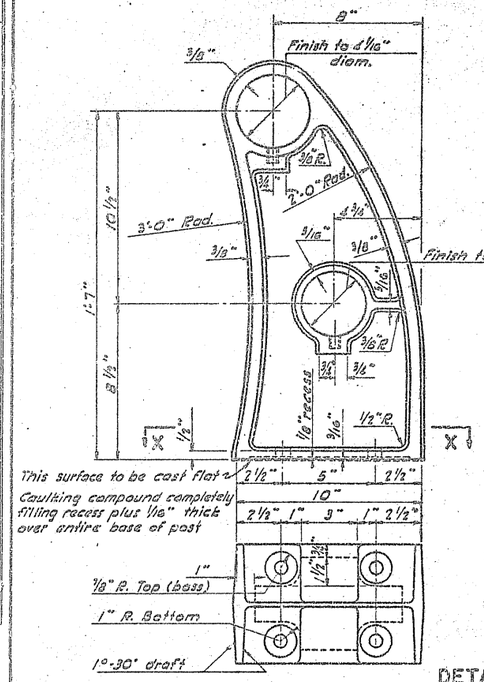
BR. NO. 9090

CHECKED BY F.D.O. & L.W.M.
MADE BY R.S.H.
CHECKED BY D.D.B.
TRACING MADE BY
CHECKED BY
QUANTITIES CHECKED BY

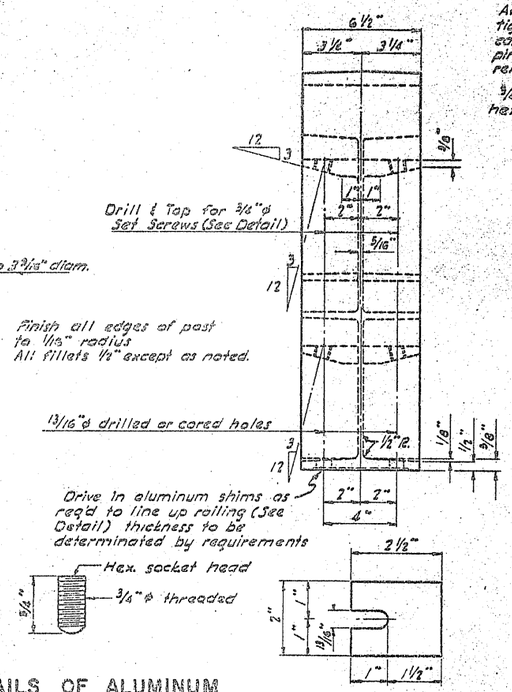
708 (1) - 38

708 (1) - 38

FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	F-708(11)		42	43



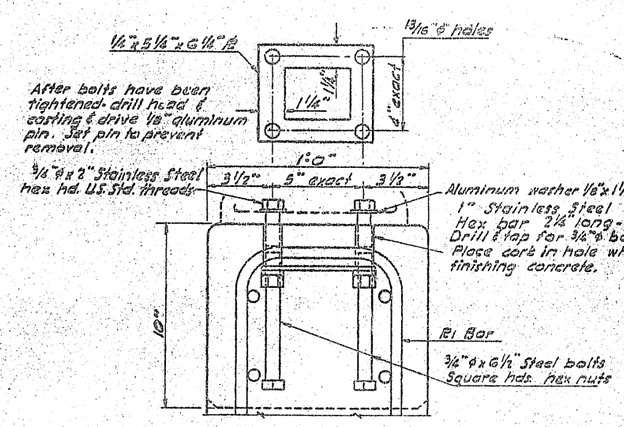
SECTION X-X



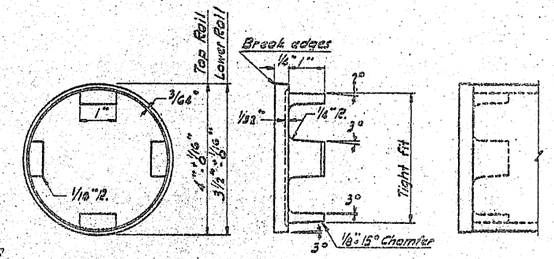
DETAILS OF ALUMINUM SET SCREWS

DETAIL OF ALUMINUM SHIMS

RAIL POST DETAILS



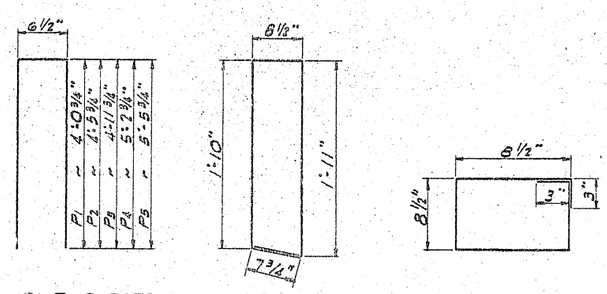
ANCHOR BOLT ASSEMBLY



CAST ALUMINUM END CAP FOR RAIL

Drive Fit All End Caps.

BAR LIST (END POSTS)				
MARK	NO	SIZE	LENGTH	SHAPE
P1	4	5	8'-8"	Bent
P2	2	5	9'-6"	"
P3	2	5	10'-6"	"
P4	2	5	11'-0"	"
P5	4	5	11'-6"	"
P6	12	4	5'-6"	Str.
P7	4	4	3'-0"	"
P8	4	5	5'-9"	Weld Band
R1	1520	4	8'-9"	Bent
R2	1570	3	3'-4"	"
R3	112	5	32'-8"	Str
R4	16	4	5'-0"	"
R5	48	5	31'-8"	"
R6	32	5	79'-0"	"
R7	16	5	32'-0"	"
R8	112	5	30'-8"	"



P1 To P5 BARS R1 BARS R2 BARS

BENT BAR DETAILS

NOTES:

Rail posts shall be permanent mold types of cast aluminum alloy and shall conform to A.S.T.M. Spec. B106, alloy SG70B. Castings shall be heat-treated T_6 temper.

Rail pipe to be aluminum drawn tube conforming to A.S.T.M. Spec. B210, alloy 6061-T_6, or extruded tube conforming to A.S.T.M. Spec. B235, alloy 6061-T_6, or aluminum alloy pipe conforming to A.S.T.M. Spec. B241, alloy 6061-T_6. Mill finish.

Set screws to be aluminum alloy 2024-T_4 conforming to A.S.T.M. Spec. B211. Set screws shall have at least 0.0002 inch anodic coating and chromate seal.

Pins to be aluminum alloy 6061-T_6 conforming to A.S.T.M. Spec. B211.

All aluminum castings shall be finished by grinding off the gates and parting line, followed by sand blasting and one coating of lacquer.

All concrete in end posts and parapet shall be class AE-2 and shall be given the "Rubbed Surface Finish".

All posts shall be set normal to the centerline grade.

Each section of pipe rail shall project through, or into, at least three and perferably four posts.

The bottom of rail posts in contact with concrete shall be coated with caulking compound conforming to the latest Federal Spec. TT-C-598. grade I-for gun application, white, natural or grey in color and furnished in cartridges. Application shall be made to clean and dry surfaces above 40°F. Joints may be freed from dust by swabbing with caulking compound waste, moistened with turpentine or universal spirits. Compound shall be placed with care to prevent smearing the area outside and around the base of the post.

All pipe, rail posts, anchor bolt assemblies, bolts, shims, set screws, end caps and compound shall be included in the price bid for Ornamental Metal Railing.

9/16" x 1 1/2" slotted holes shall be provided in top and bottom rails to match holes in the rail posts.

QUANTITIES		
Concrete Class AE-2	106.3	Cu.
Reinforcing steel	18,564	Lbs.
Ornamental Railing	2570.27	Lin. Ft.

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

RAILING DETAILS

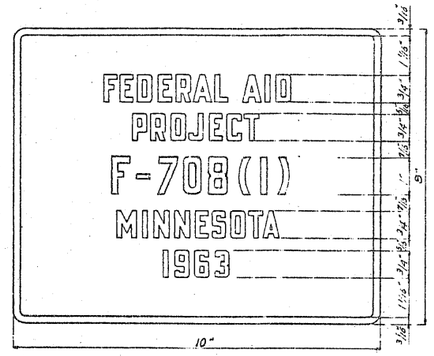
708(11)-39

708(11)-39

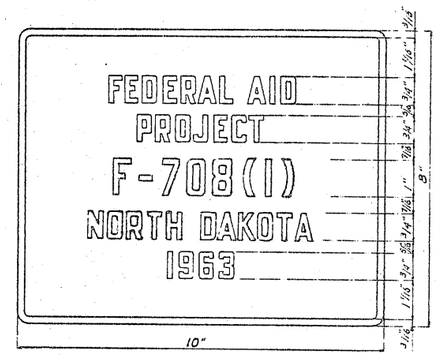
FED. ROAD DIV. NO.	STATE	PROJ. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
5	N. D.	F-708(1)		43	43

Note: All dimensions for 3/4" high letters and numbers shall be in direct proportion to those shown above for the 1" high letters and numbers.

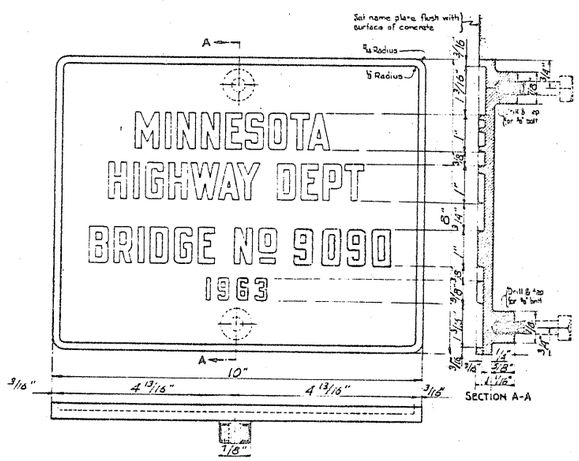
LETTERS FOR BRIDGE NAME PLATES



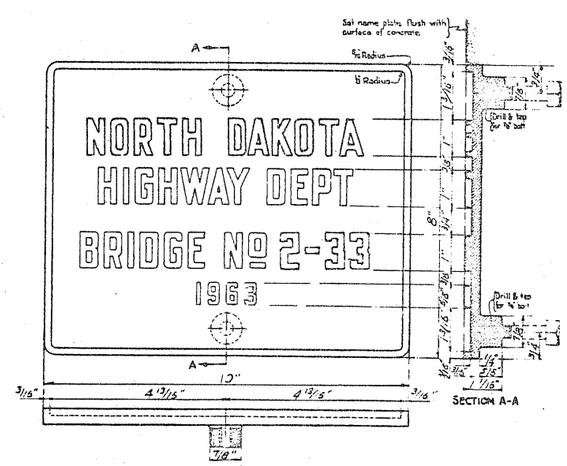
F.A.P. NAME PLATE



F.A.P. NAME PLATE



BRIDGE NAME PLATE



BRIDGE NAME PLATE

NOTES:

F.A.P. and bridge name plates shall be installed as shown on sheets 708(1)-38 and 364(10)-6 in the following locations:

N.D. Bridge No. 2-33 Name Plate, West End, South Retaining Wall Adjacent to Abutment

N.D.-F.A.P. Name Plate, West End, North Retaining Wall Adjacent to Abutment.

Minn. Br. No. 9090 Name Plate, East End, North Rail End Post.

Minn.-F.A.P. Name Plate, East End, South Rail End Post.

Name plates shall be cast of bronze composed of the following materials:

Copper - 84-86%
Tin - - 4-6%
Lead - - 4-6%
Zinc - - 4-6%

Lettering shall be approximately 3/16" above the surface of the plate and shall conform to the condensed pattern letters and spacing shown on this sheet. Vertical and horizontal spacing shall be kept in proportion to the spacing shown.

The top surface of the letters and frame shall be burnished. The background of the plate shall have a deep brown oxidized finish.

The draft on the letters shall be not more than 3° in 12".

Furnish two steel bolts 3/8" x 3" long with each plate.

Section A-A shown on the bridge name plate is typical for the F.A.P. name plate also.

The cost of furnishing and placing the Minn. name plates and furnishing only of the N.D. name plates as described on this sheet shall be included in the unit price bid for Structural Steel.

A rubbed proof of the patterns shall be submitted to the North Dakota Highway Bridge Department for approval before the name plates are cast.

RED RIVER BRIDGE
SKIDMORE AVE.
GRAND FORKS, N.D.-E. GRAND FORKS, MINN.

NAME PLATES

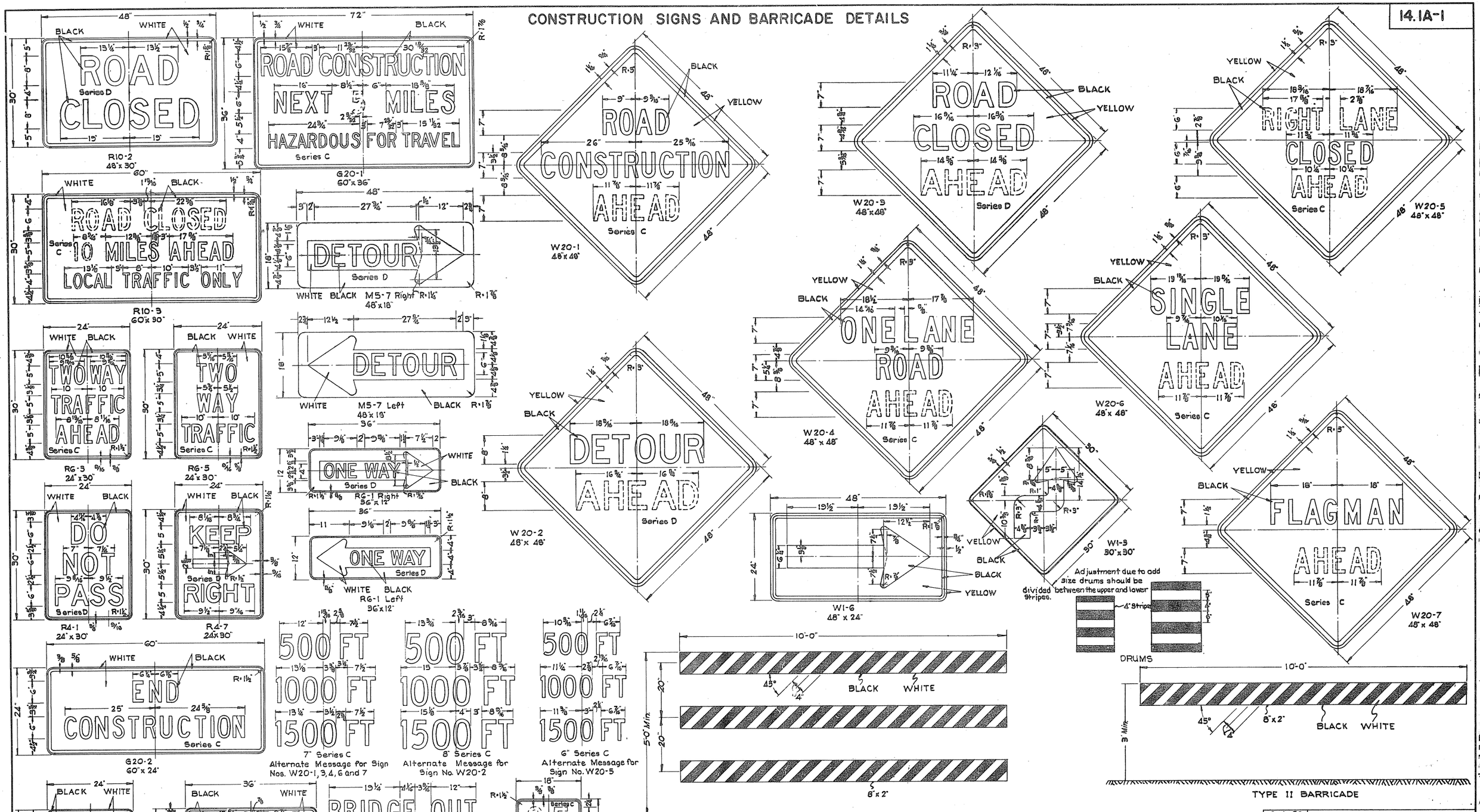
TRACING | CHECKED BY
QUANTITIES | CHECKED BY

708(1)-40

708(1)-40

CONSTRUCTION SIGNS AND BARRICADE DETAILS

14.1A-1



Notes: All signs shall have reflectorized message, border and background, unless shown otherwise on the plans. In no case is the color black reflectorized. The white stripes on the barricade shall be reflectorized. See std 14.1A-2 for assembly details and sign locations.

Revisions
8-1-62
8-6-62
8-15-62

NORTH DAKOTA
STATE HIGHWAY DEPARTMENT

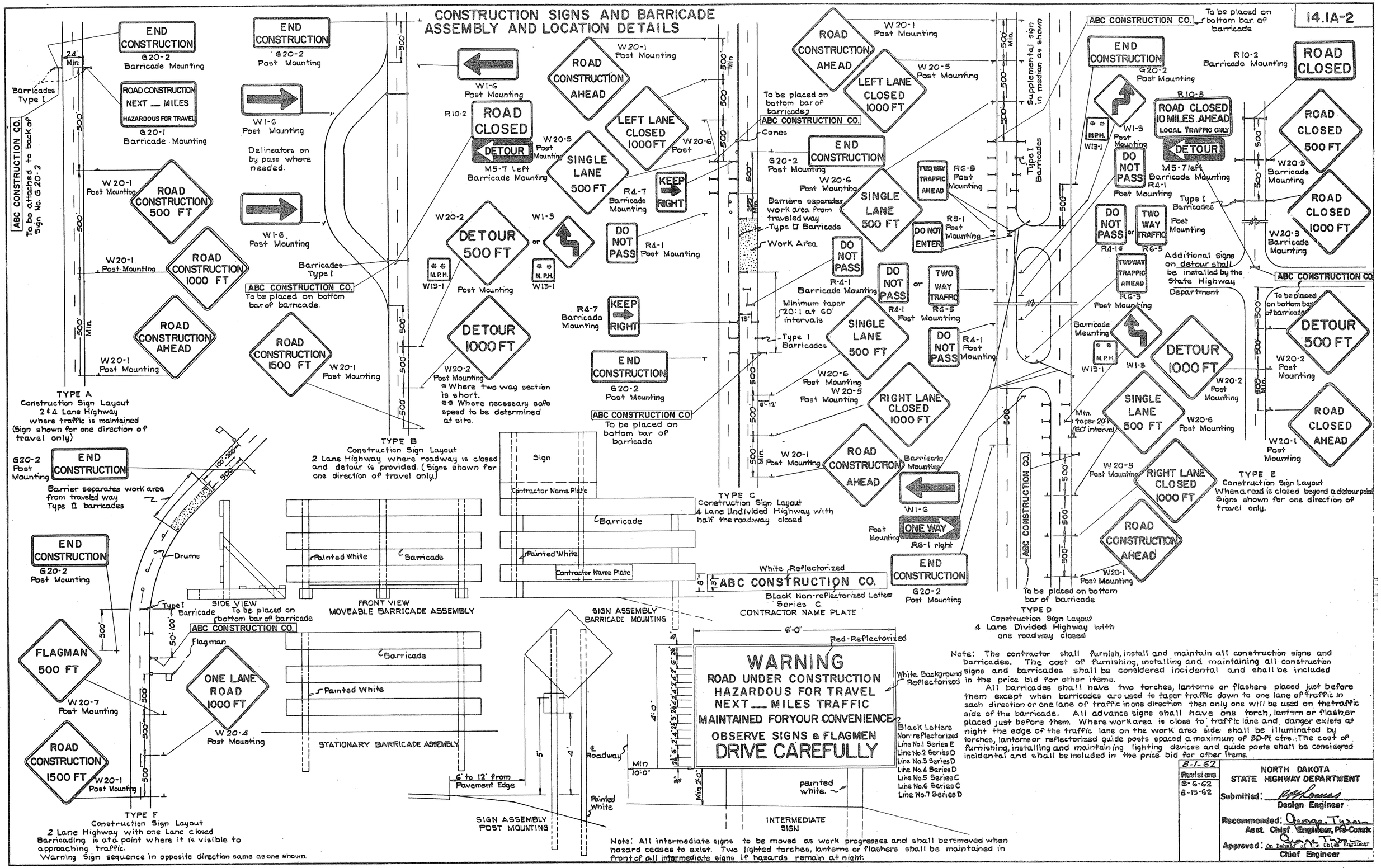
Submitted: R. P. Thomas
Design Engineer

Recommended: George Tyson
Asst. Chief Engineer, Pre-Constr.

Approved: George Tyson
Chief Engineer

CONSTRUCTION SIGNS AND BARRICADE ASSEMBLY AND LOCATION DETAILS

14.1A-2



Note: The contractor shall furnish, install and maintain all construction signs and barricades. The cost of furnishing, installing and maintaining all construction signs and barricades shall be considered incidental and shall be included in the price bid for other items.

All barricades shall have two torches, lanterns or flashers placed just before them except when barricades are used to taper traffic down to one lane of traffic in each direction or one lane of traffic in one direction then only one will be used on the traffic side of the barricade. All advance signs shall have one torch, lantern or flasher placed just before them. Where work area is close to traffic lane and danger exists at night the edge of the traffic lane on the work area side shall be illuminated by torches, lanterns or reflectorized guide posts spaced a maximum of 50-ft ctrs. The cost of furnishing, installing and maintaining lighting devices and guide posts shall be considered incidental and shall be included in the price bid for other items.

Note: All intermediate signs to be moved as work progresses and shall be removed when hazard ceases to exist. Two lighted torches, lanterns or flashers shall be maintained in front of all intermediate signs if hazards remain at night.

Revisions
8-1-62
8-6-62
8-15-62

NORTH DAKOTA STATE HIGHWAY DEPARTMENT

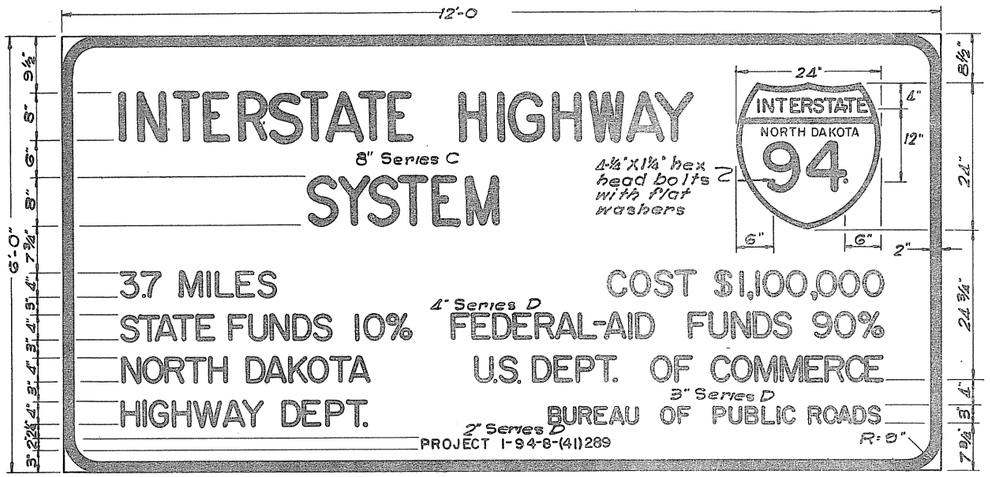
Submitted: R.P. Rowes
Design Engineer

Recommended: George Tyson
Asst. Chief Engineer, Pre-Constr.

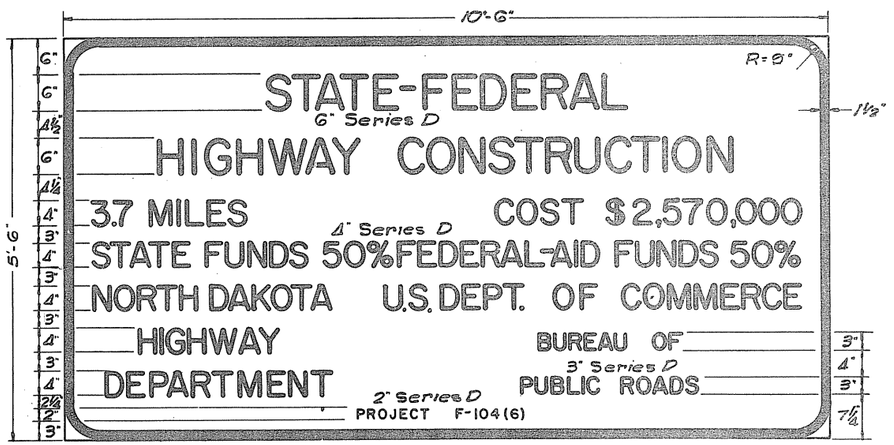
Approved: George Tyson
On Behalf of the Chief Engineer
Chief Engineer

CONSTRUCTION IDENTIFICATION SIGNS

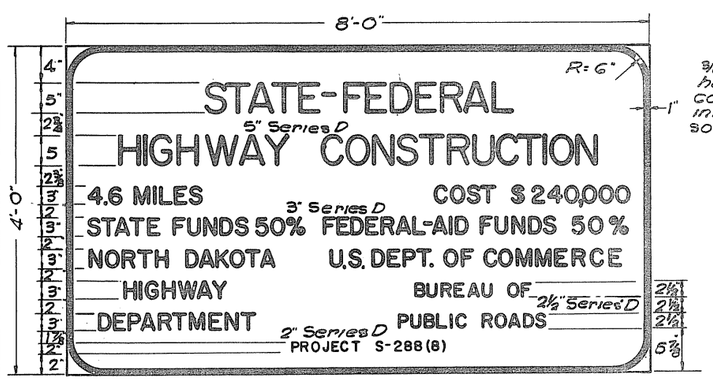
FOR INTERSTATE, PRIMARY, SECONDARY AND COUNTY SECONDARY SYSTEM PROJECTS



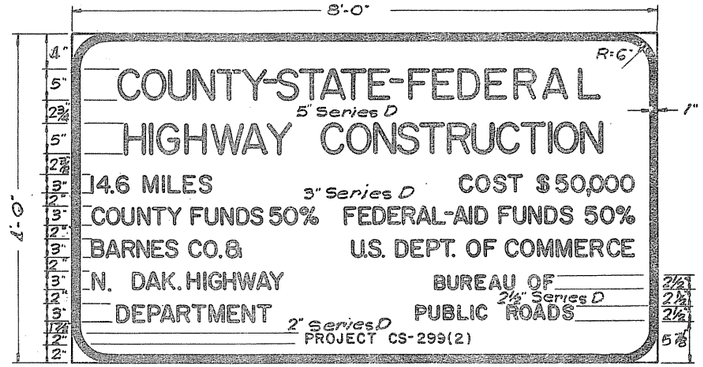
SIGN NO. ISCI-1



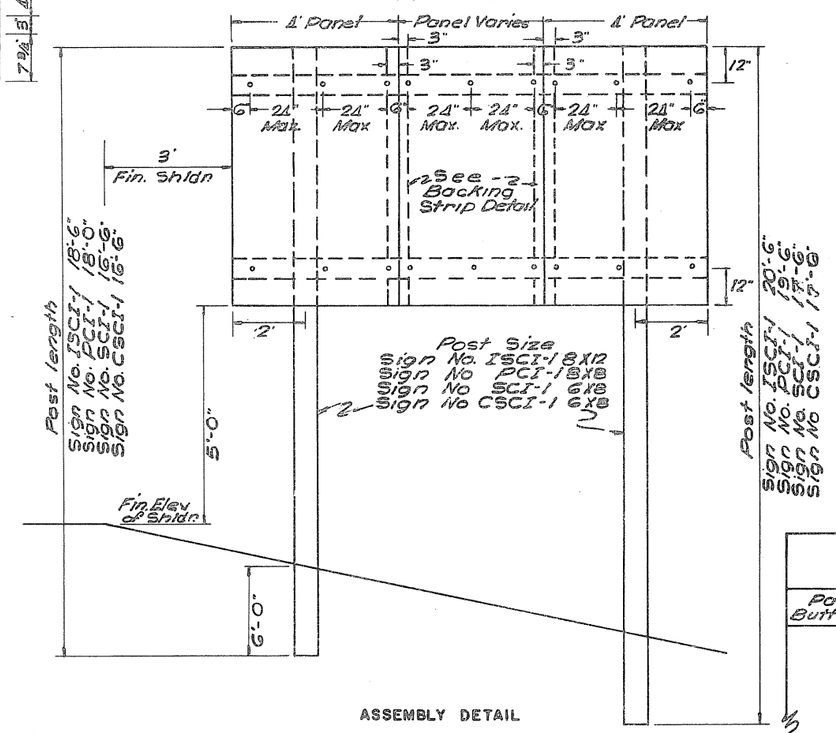
SIGN NO. PCI-1



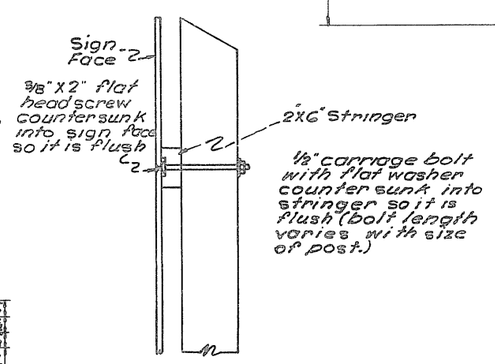
SIGN NO. SCI-1



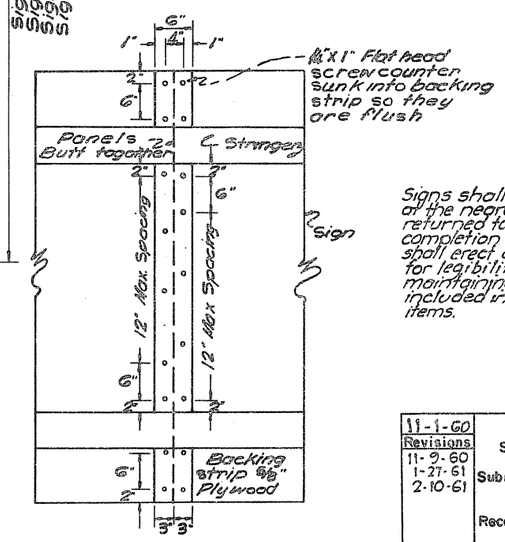
SIGN NO. CSCI-1



ASSEMBLY DETAIL



MOUNTING DETAIL



BACKING STRIP DETAIL

Sign No. SCI-1 & CSCI-1 do not require backing strips. To be in one unit.

NOTES:

MATERIALS: All bolts, nuts, washers and screws shall be Galvanized Steel or Aluminum.

SIGN BACKING: Sign backing shall be ⅝" High Density Overlay Plywood conforming to the requirements as set forth in Commercial Standards C.S. 45-55 for Douglas Fir Plywood. The plywood shall be B-B density overlay 60/60. The entire sign including backing strip shall be given (1) one coat of prime and (2) two coats of outside white paint.

PAINTING: All painting on signs shall conform to the applicable provisions of section 80 and section 132 of the Standard Specifications.

HOLES: All holes shall be Pre-Drilled before painting.

SIGN FACE: Borders and message except 24" shield shall be in black non-reflectorized letters and adhered or screened onto white painted sign. The Interstate shield backing shall be .125 aluminum sheeting and shall conform to the AASHO manual for signing of Interstate and Defense Highways and shall be reflectorized.

TIME OF PLACEMENT: Construction identification signs are to be erected at the time physical construction work starts on any Federal-aid project.

PLACEMENT: The signs are to be appropriately placed so as to inform the traveling public of work in progress. For Construction along an existing highway one sign at each end of construction section, facing traffic approaching the work. Where the highway construction section is not generally along an existing highway, the signs shall be placed on the more important cross roads at points where the work is visible to the public. Location will be shown on the plans and Placement designated by Eng. The construction identification sign shall be located so as not to obscure or detract from the effectiveness of other official signs. See plans for correct message for all signs. Cost figures shall be furnished to the contractor after the letting.

SIGN SUPPORTS: Posts and Stringers shall be of the size shown of pine, fir, spruce, cedar, larch aspen, American elm or ash and shall be No. 1 grade surfaced four sides complying with grading rules which conform to the basic provisions of the "American Lumber Standards" and shall be free of heart centers.

Signs shall be furnished by the State at the nearest District Office and returned to the District Office upon completion of the Project. The Contractor shall erect and adequately maintain for legibility. Cost of delivering, erecting, maintaining and returning shall be included in the price bid for other items.

Revisions
11-1-60
11-9-60
1-27-61
2-10-61

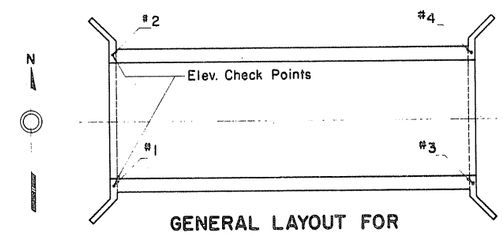
NORTH DAKOTA
STATE HIGHWAY DEPARTMENT

Submitted: Design Engineer

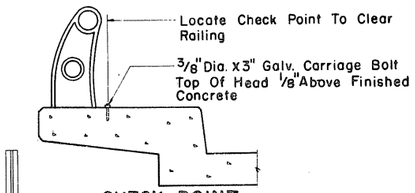
Recommended: Director, Staff Division

Approved: Chief Engineer

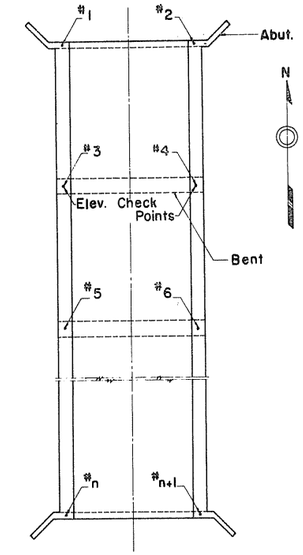
BRIDGE BENCH MARKS



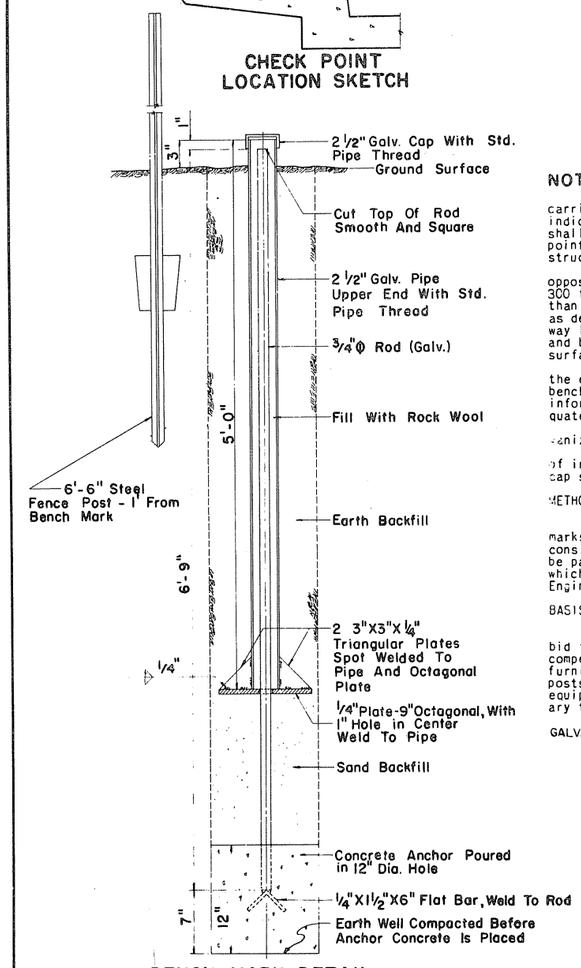
GENERAL LAYOUT FOR SINGLE SPAN



CHECK POINT LOCATION SKETCH



GENERAL LAYOUT FOR MULTIPLE SPAN



BENCH MARK DETAIL

NOTES:

Elevation check points shall consist of 3/8"x 3" galvanized carriage bolts (or equal) set in the concrete curb at the points indicated on the General Layout Sketches. The top of bolt head shall project above the finished concrete 1/8". Elevation check points shall be placed on each curb over each unit of the substructure for each bridge at a structural location.

Two bench marks as detailed hereon shall be set at diagonal opposite positions away from the structure location and at least 300 feet from the nearest point on the bridge or bridges (if more than one at a location). These bench marks shall be constructed as detailed on this sheet and located near the Highway Right-of-way lines. The steel fence post shall extend 4'-0" above ground and be painted with two coats of white paint suitable for steel surfaces.

The Project Engineer shall run a set of levels determining the elevation of each check point on the structure and the two bench marks immediately after the completion of the bridge. This information shall be submitted to the Bridge Engineer with adequate information locating each check point and bench mark.

Except for fence posts, all metal parts to be hot dip galvanized after punching, shearing, welding, and fabrication.

Threads of cap and pipe are not to be galvanized. At time of installation these threads are to be coated with grease and cap screwed to snug fit.

METHOD OF MEASUEMENT:

Each set of Bridge Bench Marks consisting of two bench marks and the required number of elevation check points shall be considered as one unit for bidding purposes and the quantity to be paid for shall be the number of sets of bridge bench marks which have been installed complete in place and accepted by the Engineer.

BASIS OF PAYMENT:

Bridge Bench Marks shall be paid for at the contract price bid for each set of Bridge Bench Marks, which price shall be full compensation for all excavation, backfill and clean-up, and for furnishing, hauling and placing all elevation check points, fence posts, galvanized pipe, caps, rods, sand backfill, concrete, rock equipment, tools and incidentals, including galvanizing, necessary to complete this item.

GALVANIZING:

After fabrication the complete assembly shall be Hot Dip Galvanized.

Revised 10-20-59
Revised 6-9-59

NORTH DAKOTA
STATE HIGHWAY DEPARTMENT

Submitted: Joseph R. Kirby
Bridge Engineer

Recommended: M. Sherebom
Director Staff Division

Approved: R. L. Bradley
Chief Engineer

Date: 3-3-58