**BRIDGE DESIGN GROUP CHAIRMAN FACTUAL REPORT**

(89 pages)

**NATIONAL TRANSPORTATION SAFETY BOARD**
**OFFICE OF HIGHWAY SAFETY**
**WASHINGTON, DC 20594**

# BRIDGE DESIGN GROUP CHAIRMAN FACTUAL REPORT

## A. ACCIDENT

| | |
|---|---|
| Type of Accident: | Bridge Collapse |
| Date and Time: | August 1, 2007 at 6:05 pm CDT |
| Location: | Interstate Highway 35W Bridge over the Mississippi River, Minneapolis, Hennepin County, MN |
| Fatalities: | 13 |
| Injuries: | 145 |
| Case Number: | HWY07MH024 |

## B. BRIDGE DESIGN GROUP

| | | |
|---|---|---|
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## C. ACCIDENT SYNOPSIS

About 6:05 p.m. (CDT), on Wednesday, August 1, 2007, the 35W Interstate Highway Bridge over the Mississippi River, in Minneapolis, Minnesota experienced a catastrophic failure in the main span of the deck truss portion of the 1907-foot-long bridge. As a result, approximately 1,000 feet of the deck truss collapsed with about 456 feet of the main span falling into the river. An assessment of the gusset plates within the deck truss revealed that the connections at U10, U10 prime, L11 and L11 prime were under-designed. The bridge was comprised of eight traffic lanes, with four lanes in each direction. At the time of the collapse, a roadway construction project was underway that resulted in the closure of two northbound and two southbound traffic lanes causing traffic queues on the bridge. A total of 111 vehicles were documented as being on the portion of the bridge that collapsed. Of these, 17 vehicles were recovered from the water. As a result of the bridge collapse, 13 people died and 145 people were injured.

## D. DETAILS OF THE REPORT

The Bridge Design Group Chairman Factual Report is organized by the following topic areas and page numbers:

| Topic Area | | | Page Number |
|---|---|---|---|

# 1. PREFATORY DATA

## 1.1 ACCIDENT LOCATION

The collapsed portion of the bridge was located on I-35W approximately 1 mile northeast of junction I-94. The bridge spanned the Mississippi River, Minnesota Commercial Railroad Tracks, West River Parkway, and 2nd Street. Figure 1 illustrates the accident site was located in the City of Minneapolis Hennepin County, Minnesota.

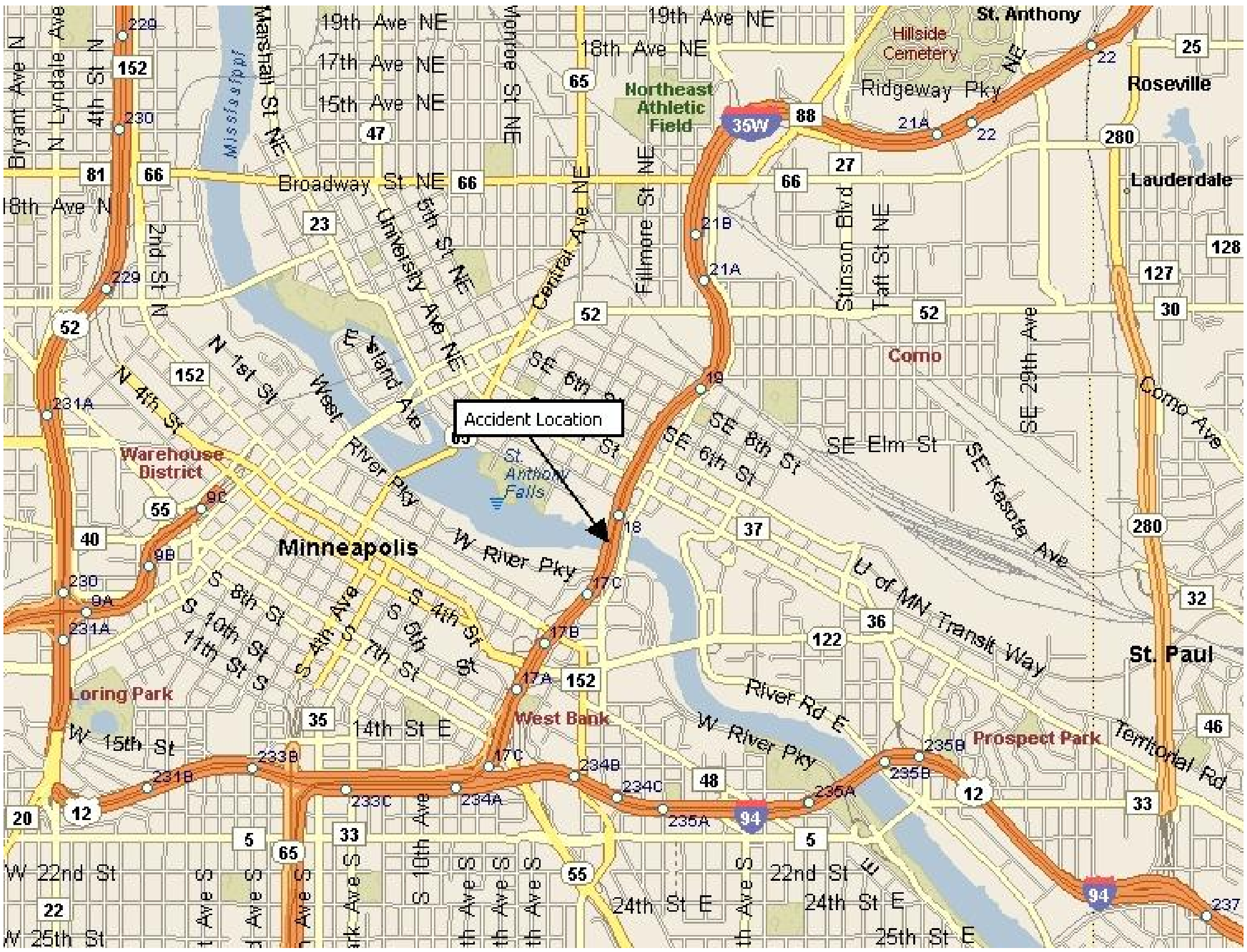


**Figure 1 – Location Map**

# 2. BRIDGE DESCRIPTION

## 2.1 GENERAL DESCRIPTION

Opened to traffic in 1967, the I-35W Bridge had 13 piers, 14 spans, and a total length of 1,907 feet. The split deck had four travel lanes in the northbound direction and four travel lanes in the southbound direction. The bridge deck widened at the north end to accommodate on and off ramps, and curved slightly at the south end to accommodate the approach roadway alignment.

Spans 6 through 8 were "fracture critical" steel deck trusses approximately 988 feet long and comprised of welded "built-up" members. Each deck truss was approximately 60 feet deep at piers 6 and 7, and connected to each other laterally by welded floor beam trusses, which cantilevered beyond the deck trusses on both sides and supported the 27 inch deep rolled beam roadway stringers. Each end of the deck truss cantilevered over a pier to support the adjacent approach span with a unique crossbeam configuration. The approach spans framed into a crossbeam, which was supported by rocker bearings on the cantilevered truss ends. Spans 1 through 5 comprised the south approach. Spans 1 and 2 consisted of 33 inch deep rolled beams while spans 3 through 5 consisted of 48 inch deep welded plate girders. The north approach was comprised of spans 9 through 14. Span 9 consisted of 48 inch deep welded plate girders while spans 9 through 11 consisted of welded plate girders 36 inches deep. Spans 12 through 14 were cast-in-place 24 inch deep voided concrete slabs.

## 2.2 STRUCTURE INVENTORY

The Minnesota Department of Transportation (Mn/DOT) provided the structure inventory report for the I-35W Bridge over the Mississippi River.

**Table 1 – Mn/DOT Structure Inventory Report for I-35W Bridge (Bridge #9340)**

| | |
|---|---|
| **General** | |
| Maintenance Crew | 7627 |
| District | Metro |
| Maintenance Area | Metro |
| County | 27 – Hennepin |
| City | Minneapolis |
| Description Location | 1 MI NE of JCT 94 |
| Latitude | 44d 58m 50.89s |
| Longitude | 93d 14m 40.09s |
| Custodian | Mn/DOT |
| Owner | Mn/DOT |
| Inspection By | Mn/DOT – Metro District |
| Year Built | 1967 |
| **Structure** | |
| Main Span Type | Continuous Steel Deck Truss |
| Main Span Detail (Type of Truss) | Warren with Vertical |
| Approach Span Type | Continuous Steel Beam Span |
| Number of Spans | Main: 3, Approach: 11, Total: 14 |
| Main Span Length | 456 feet |
| Structure Length (Total Length of Bridge) | 1,907 feet |
| Deck Width | 113.3 feet and varies |
| Deck Material | Cast-In-Place Concrete |
| Wear Surface Type | Low Slump Concrete |
| Wear Surface Install Year | 1978 |
| Wear Course/Fill Depth | 2 inches |

| | |
|---|---|
| Structure Area | 219,086 square feet |
| Roadway Area | 201,511 square feet |
| **Roadway** | |
| Lanes | 8 lanes on bridge |
| Average Daily Traffic (ADT) | 141,000 (2004) |
| Heavy Commercial ADT | 5,640 |
| Functional Classification | Urban Principal Arterial Interstate Highway |
| **Roadway Dimensions** | |
| Roadway Width | 52 feet northbound, 52 feet southbound |
| Median Width | 4 feet |
| **Miscellaneous Bridge Data** | |
| Field Connections | Riveted |
| Cantilever | Friction |
| Abutment Foundation | Concrete Footing Pile |
| Pier Foundation | Concrete Spreading Rock |
| **Paint** | |
| Year Painted | 1968 |
| Percent Unsound | 15% |
| Painted Area | 490,200 square feet |
| Primer Type | Lead |
| **Bridge Signs** | |
| Posted Load | Not Required |
| Traffic | Not Required |
| Horizontal | Not Required |
| Vertical | Not Applicable |
| **Inspection** | |
| Deficient Status | Structurally Deficient |
| Sufficiency Rating | 50 |
| Last Inspection Date | June 15, 2006 |
| Inspection Frequency | 12 months |
| Inspector Name | Metro |
| **Condition Codes** | |
| Deck (6% Unsound) | 5 |
| Superstructure | 4 |
| Substructure | 6 |
| Channel | 7 |
| **Appraisal Ratings** | |
| Structure Evaluation | 4 |
| Deck Geometry | 4 |
| Underclearances | 7 |
| Waterway Adequacy | 9 |
| Approach Alignment | 8 |
| **In Depth Inspection** | |
| Fracture Critical | June 2006 |
| Underwater | February 2005 |

| Waterway | |
|---|---|
| Waterway Opening | 50,000 square feet |
| Navigational Control | Permit Required |
| Pier Protection | Not Required |
| Navigational Vertical / Horizontal Clearance | 64 feet / 400 feet |
| MN Scour Code | Low Risk |
| Scour Evaluation Year | 1993 |
| **Capacity Ratings** | |
| Design Load | HS20MOD |
| Operating Rating | HS 33.0 |
| Inventory Rating | HS 20.0 |
| Rating Date | December 1, 1995 |
| Mn/DOT Permit Codes | A: 1, B: 1, C: 1 |

## 3. TRUSS DIAGRAMS

Figure 2 shows the tension and compression members of the continuous steel deck truss. Tension members are shown in red, compression members are shown in blue, and reversal members are shown in green. Four piers supported the truss (5, 6, 7, and 8).

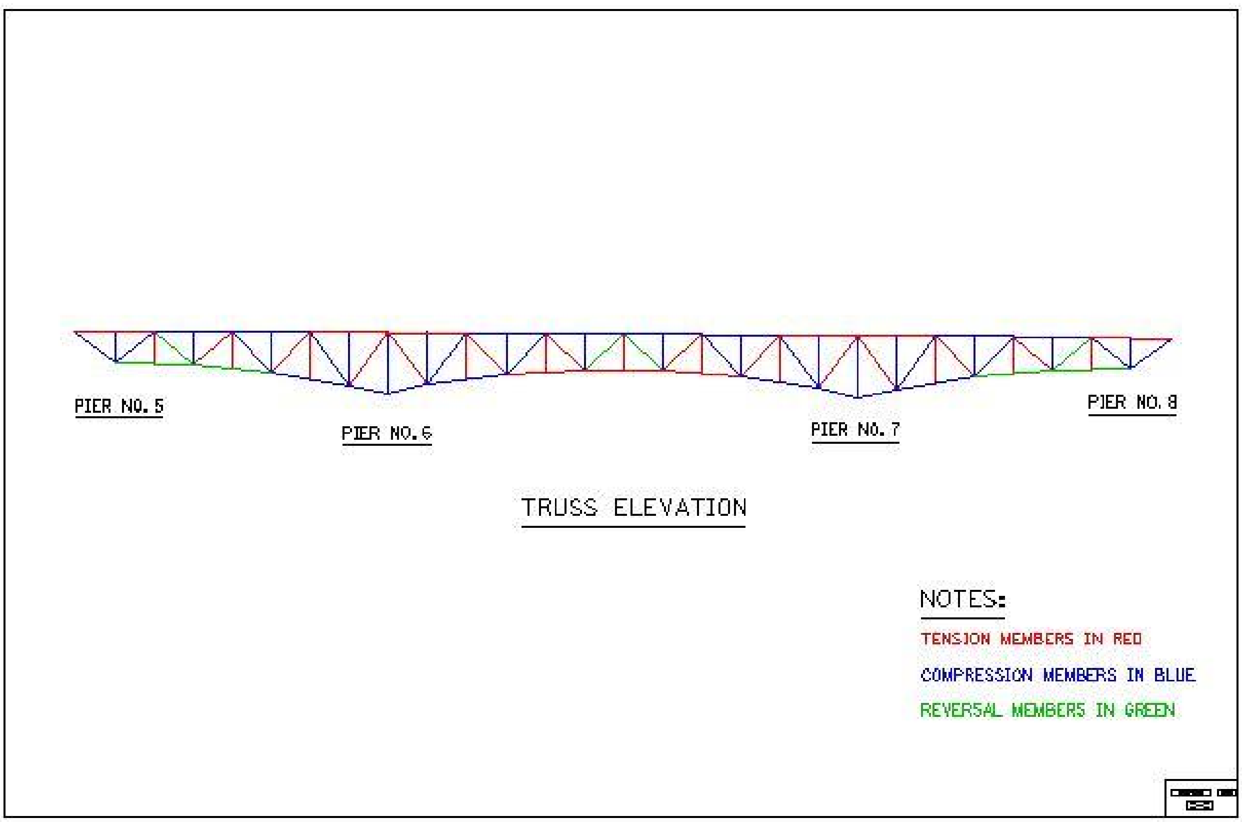


**Figure 2 – Tension and compression members of the continuous steel deck truss**

Figure 3 shows the entire elevation of the bridge from the south abutment to the north abutment. Also shown in the figure are the approach spans on either side of the continuous steel deck truss. Four piers supported the south approach spans (1, 2, 3, and 4) and five piers supported the north approach spans (9, 10, 11, 12, and 13).

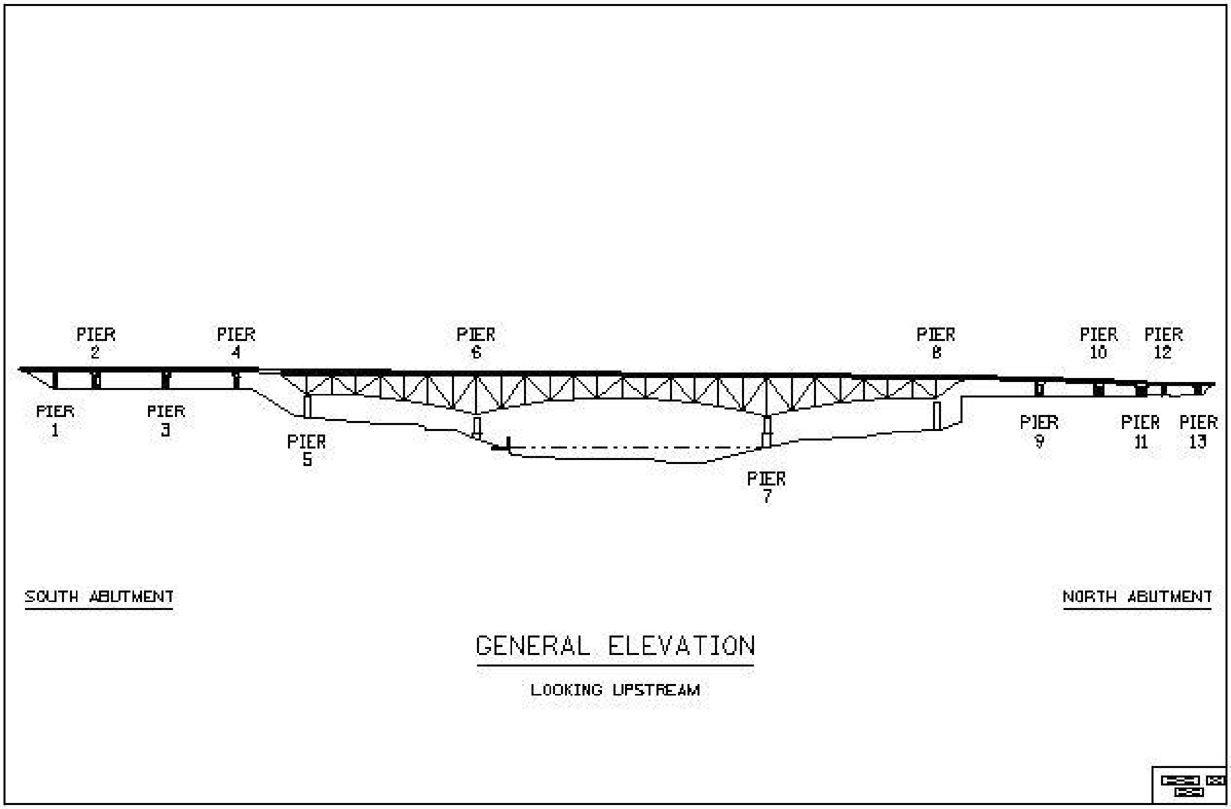


**Figure 3 – Elevation of entire length of bridge from south abutment to north abutment**

Table 2 shows the distance between the piers and the type of superstructure used to support the deck.

**Table 2 – Distance between each pier and type of girder / truss**

| Location | Distance (feet) | Type of Girder / Truss |
|---|---|---|
| South abutment to Pier 1 | 53 feet 1 inch | 2 span continuous beams |
| Pier 1 to Pier 2 | 72 feet | 2 span continuous beams |
| Pier 2 to Pier 3 | 110 feet | 3 span continuous girders |
| Pier 3 to Pier 4 | 110 feet | 3 span continuous girders |
| Pier 4 to Pier 5 | 109 feet 4 inches | 3 span continuous girders |
| Pier 5 to Pier 6 | 265 feet 8 inches | 3 span continuous trusses |
| Pier 6 to Pier 7 | 456 feet | 3 span continuous trusses |
| Pier 7 to Pier 8 | 265 feet 11 inches | 3 span continuous trusses |
| Pier 8 to Pier 9 | 168 feet 1 inch | 3 span continuous girders |
| Pier 9 to Pier 10 | 94 feet | 3 span continuous girders |

| | | |
|---|---|---|
| Pier 10 to Pier 11 | 68 feet | 3 span continuous girders |
| Pier 11 to Pier 12 | 47 feet | 3 span continuous voided slab |
| Pier 12 to Pier 13 | 57 feet 11 inches | 3 span continuous voided slab |
| Pier 13 to North abutment | 30 feet 1 inch | 3 span continuous voided slab |

The original bridge design accounted for thermal expansion using a combination of fixed and expansion bearings. The fixed bearings were used at piers 1, 3, 7, 9, 12, and 13. The expansion bearings were used at the south abutment, north abutment, and at piers 2, 4, 5, 6, 8, 10, and 11.

## 4. DECK WIDTH, THICKNESS, AND HORIZONTAL ALIGNMENT

The total width of the deck was approximately 113 feet and 4 inches. The width of the four northbound travel lanes was approximately 52 feet. The width of the four southbound travel lanes was also 52 feet. The total number of travel lanes on the bridge was eight travel lanes. The northbound lanes were separated from the southbound lanes by a 4-foot wide median barrier. The railing and curb located on the outside edges of the bridge was approximately 2 feet and 8 inches wide.

The bridge deck, in 1967, was constructed to an approximate depth of 6 ½ inches (minimum) using cast-in-place concrete. As discussed later in this report, the average thickness of the deck was increased due to major renovations on the bridge. After the bridge collapse, concrete core testing was done to verify the average thickness of the bridge deck.

A horizontal curve existed on the south end of the bridge that consisted of a 3 degree and 15 minute curve for vehicles traveling in the northbound and southbound direction of travel.

### 4.1 LOCATION OF EXPANSION JOINTS AND HINGE

The total number of expansion joints on the deck was eleven. The first expansion joint was located at the south abutment. The second expansion joint was located near Pier 2. The third expansion joint was located at the south end of the steel truss near Pier 5. The fourth thru eighth expansion joints were located at nodes 4, 8, 14, 8', and 4' respectively on the main truss spans (refer to the Structural Investigation Group Factual Report for the location of nodes on the main truss spans). The ninth expansion joint was located at the north end of the steel truss near Pier 8. The tenth expansion joint was located at Pier 11. The eleventh expansion joint was located at the north abutment.

The bridge contained only one hinge located between Piers 1 and Pier 2 in Span 2.

## 5. <u>NATIONAL BRIDGE INSPECTION STANDARDS (NBIS)</u>

### 5.1 GENERAL DESCRIPTION

The U.S. Department of Transportation Federal Highway Administration (FHWA) had the legislative authority under the Code of Federal Regulations (23 CFR Part 650) to develop a national bridge inspection program. The CFR indicated the following:

> *"**650.301 Purpose.***
> *This subpart sets the national standards for the proper safety inspection and evaluation of all highway bridges in accordance with 23 U.S.C. 151.*
>
> ***650.307 Bridge inspection organization***
> *(a) Each State transportation department must inspect, or cause to be inspected, all highway bridges located on public roads that are fully or partially located within the State's boundaries, except for bridges that are owned by Federal agencies."*

The national bridge inspection program was formed as a direct result from a bridge collapse that occurred in Point Pleasant, West Virginia on December 15, 1967 that killed 46 people. The tragic collapse aroused national interest in the safety inspection and maintenance of bridges when a 2,235-foot section of the Silver Bridge collapsed into the Ohio River.

The national bridge inspection program consists of national bridge inspection standards (NBIS) and a national bridge inventory (NBI). The national bridge inspection standards (NBIS) were first established in 1971 to set national requirements regarding bridge inspection frequency, inspector qualifications, report formats, and inspection and rating procedures. The national bridge inventory (NBI) is the aggregation of structure inventory and appraisal data collected by each state to fulfill the requirements of the program. The structure inventory data consists of fields that include identification of the bridge, structure type and material, age and service, geometric data, navigation data, and classification. The structure appraisal data consists of fields that include condition, load rating and posting, appraisal, proposed improvements, and inspections.

The national bridge inspection standards require bridges be inspected at regular intervals not to exceed 24 months.

> *"**650.311 Inspection frequency***
> *(a) Routine inspections. (1) Inspect each bridge at regular intervals not to exceed twenty-four months."*

Bridge inspectors are required to be trained regarding proper bridge inspection techniques and complete a Federal Highway Administration (FHWA) approved comprehensive bridge inspection training course.

## 5.2 BRIDGE INSPECTION DEFINITIONS

Bridges or culverts that carry vehicular traffic and are longer than 20 feet are part of the National Bridge Inventory system. Listed below are standard terms and definitions used in the bridge inspection industry.

**General Condition Ratings –** general condition ratings describe the current condition of a bridge or culvert. The general condition ratings are an overall assessment of the physical condition of the deck (riding surface), the superstructure (load carrying members such as beams or trusses that support the driving surface), substructure (abutments and piers) or culvert. General condition ratings range from 0 (failed condition) to 9 (excellent).

**Structurally Deficient Bridge –** the classification structurally deficient is used to determine eligibility for federal bridge replacement or rehabilitation funding. Bridges are classified as structurally deficient if they have a general condition rating for the deck, superstructure, substructure or culvert as 4 or less or if the road approaches regularly overtop due to flooding. A general condition rating of 4 means that the component rating is described as poor. Examples of poor condition include corrosion that has caused significant section loss of steel support members, movement of substructures, or advanced cracking and deterioration in concrete bridge decks. For bridge owners, the classification structurally deficient is a reminder that the bridge may need further analysis that may result in load posting, maintenance, rehabilitation, replacement or closure.

The fact that a bridge is structurally deficient does not imply that it is unsafe. A structurally deficient bridge typically needs maintenance and repair and eventual rehabilitation or replacement to address deficiencies. To remain open to traffic, structurally deficient bridges can be posted, if required, with reduced weight limits that restrict the gross weight of vehicles using the bridges. If unsafe conditions are identified during a physical inspection, the structure is closed.

**Functionally Obsolete Bridge –** a functionally obsolete bridge is one that was built to standards that do not meet the minimum requirements for a new bridge. These bridges are not necessarily rated as structurally deficient, nor are they inherently unsafe. Functionally obsolete bridges include those that have inadequate vehicular capacity or sub-standard geometric features such as narrow lanes, narrow shoulders, poor approach alignment or inadequate vertical or horizontal under clearance. The classification functionally obsolete is also a term used as a priority status for federal bridge replacement and rehabilitation funding eligibility.

**Fracture Critical Bridge –** a fracture critical bridge typically has a steel superstructure with load (tension) carrying members arranged in a manner in which if one fails, the bridge could collapse. Examples of fracture critical bridges are two girder bridges or truss bridges. The classification of fracture critical does not mean the bridge is inherently unsafe. The NBIS defines a fracture critical member as a non-redundant member that is in tension.

**Sufficiency Rating –** sufficiency rating is a computed numerical value that is used to determine eligibility of a bridge for federal funding. The sufficiency rating formula result varies from 0 to 100. The formula includes factors for structural condition, bridge geometry, and traffic considerations. The sufficiency rating formula is contained in the December 1995 Edition of the "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges". A bridge with a sufficiency rating of 80 or less is eligible for federal bridge rehabilitation funding. A bridge with a sufficiency rating of less than 50 is eligible for federal bridge replacement funding.

## 5.3 INSPECTION TYPES AND INTERVALS

U.S. federal regulations define eight types of bridge inspections. Three of these, fracture critical member inspection, routine inspection, and underwater inspection occur at intervals set by regulation. The standard interval for a fracture critical member inspection and routine inspection are 24 months. The standard interval for an underwater inspection is 60 months. The eight types of bridge inspections are described below:

**Damage Inspection** – An unscheduled inspection to assess structural damage resulting from environmental factors or human actions.

**Fracture Critical Member Inspection** – A hands-on inspection of a fracture critical member or member components that may include visual and other nondestructive evaluation.

**Hands-On Inspection** – Inspection within arms length of the component. Inspection uses visual techniques that may be supplemented by non-destructive testing.

**In-Depth Inspection** – A close-up inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using routine inspection procedures; hands-on inspection may be necessary at some locations.

**Initial Inspection** – First inspection of a bridge as it becomes a part of the bridge inventory to provide all Structure Inventory and Appraisal data and other relevant data and to determine baseline structural conditions.

**Routine Inspection** – Regularly scheduled inspection consisting of observations and/or measurements needed to determine the physical and functional condition of the bridge, to identify any changes from initial or previously recorded conditions, and to ensure that the structure continues to satisfy present service requirements.

**Special Inspection** – An inspection scheduled at the discretion of the bridge owner, used to monitor a particular known or suspected deficiency.

**Underwater Inspection** – Inspection of the underwater portion of a bridge substructure and the surrounding channel that cannot be inspected visually at low water by wading or probing, generally requiring diving or other appropriate techniques.

## 5.4 CONDITION RATINGS FOR I-35W BRIDGE (BRIDGE #9340)

The Federal Highway Administration (FHWA) provided the condition ratings, sufficiency rating, and status of the I-35W Bridge (Bridge #9340) from 1983 through 2007. Table 3 shows the condition ratings, sufficiency ratings, and status for Bridge #9340 from 1983 through 2007 as recorded on the National Bridge Inventory (NBI) forms (See Attachment 1 – National Bridge Inventory (NBI) forms for the I-35W Bridge (Bridge #9340) from 1983 through 2007).

The I-35W Bridge (Bridge #9340) had been structurally deficient for the past 16 years dating back to 1991 when the superstructure received its first condition rating of 4. Since 1991, the superstructure was recorded as receiving a condition rating of 4 on each of the NBI forms, except for 1999, when the condition rating was not available.

**Table 3 – Condition ratings, sufficiency ratings, and status for Bridge #9340 from 1983 through 2007 as recorded on the National Bridge Inventory (NBI) forms**

| Year | Deck Condition Rating | Superstructure Condition Rating | Substructure Condition Rating | Sufficiency Rating | Status |
|---|---|---|---|---|---|
| 1983 | 6 | 7 | 6 | 80.1 | Not Deficient |
| 1984 | 6 | 7 | 6 | 80.1 | Not Deficient |
| 1985 | 6 | 7 | 6 | 80.1 | Not Deficient |
| 1986 | 6 | 7 | 6 | 79.6 | Not Deficient |
| 1987 | 6 | 7 | 6 | 79.8 | Not Deficient |
| 1988 | 6 | 7 | 6 | 79.8 | Not Deficient |
| 1989 | 6 | 8 | 6 | 75.5 | Not Deficient |
| 1990 | 6 | 7 | 6 | 75.5 | Not Deficient |
| 1991 | 6 | 4 | 6 | 46.5 | Structurally Deficient |
| 1992 | 6 | 4 | 6 | 46.5 | Structurally Deficient |
| 1993 | 6 | 4 | 6 | 46.5 | Structurally Deficient |
| 1994 | 6 | 4 | 6 | 46.5 | Structurally Deficient |
| 1995 | 6 | 4 | 6 | 46.5 | Structurally Deficient |
| 1996 | 6 | 4 | 6 | 49 | Structurally Deficient |
| 1997 | 6 | 4 | 6 | 49 | Structurally Deficient |
| 1998 | 6 | 4 | 6 | 49 | Structurally Deficient |
| 1999 | N/A[1] | N/A | N/A | 76 | Not Deficient |
| 2000 | 5 | 4 | 6 | 48 | Structurally Deficient |
| 2001 | 5 | 4 | 6 | 48 | Structurally Deficient |
| 2002 | 5 | 4 | 6 | 50 | Structurally Deficient |
| 2003 | 5 | 4 | 6 | 50 | Structurally Deficient |
| 2004 | 5 | 4 | 6 | 50 | Structurally Deficient |
| 2005 | 5 | 4 | 6 | 50 | Structurally Deficient |

[1]N/A means no value submitted by Mn/DOT.

| | | | | | |
|---|---|---|---|---|---|
| 2006 | 5 | 4 | 6 | 50 | Structurally Deficient |
| 2007 | 5 | 4 | 6 | 50 | Structurally Deficient |

Mn/DOT conducted annual bridge inspections on the I-35W Bridge (Bridge #9340) from 1971 through 2007. From 1971 to 1994, the annual bridge inspections were often recorded (in four year increments) on the same inspection form. The same form was used each year for up to four years, than a new form was used. Four years of annual inspections could be recorded on one page. Starting in 1995, the annual bridge inspections were recorded on new forms.

Minnesota Rules[2] require bridges be inspected annually, unless a longer interval not to exceed two years is authorized by the commissioner. Mn/DOT acknowledged to NTSB investigators there are no exemptions for 2 year inspections of fracture critical or structurally deficient bridges.

> *"Each bridge must be inspected annually, unless a longer interval not to exceed two years is authorized by the commissioner."*

Based on Mn/DOT's annual inspections of the I-35W Bridge (Bridge #9340), FHWA required Mn/DOT to submit annually the condition rating of the deck, superstructure, and substructure of the bridge. FHWA used the condition ratings, furnished by Mn/DOT, to compute the sufficiency rating and deficient status of the bridge. FHWA required the data be submitted in a text file format. The format was contained in Appendix E of FHWA's "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges"[3].

Mn/DOT acknowledged in a April 22, 2008 letter to NTSB investigators the reason why the superstructure condition rating increased from 7 to 8 in 1989, why no values were submitted for the condition ratings in 1999, and why the sufficiency rating increased from 49 to 76 in 1999 (See Attachment 2 – Letter to the National Transportation Safety Board from the Minnesota Department of Transportation dated April 22, 2008):

> *"**Question:** Please explain the following discrepancies on the NBIS condition ratings. Why did the superstructure condition rating increase from 7 to 8 in 1989?*
> ***Response: Most likely an error on Mn/DOT's part in creating the FHWA data file produced the 8. Our inspection records incorrectly show "N" for superstructure in the 1988 inspection.***
>
> ***Question:** Why were no values submitted for the condition ratings in 1999?*
> ***Response: We were using Brinfo bridge inspection software at that time. The Sept 11, 1998 inspection data stored in Brinfo had deck=5, super=4, sub=6. Brinfo did not transfer that information correctly into the 1999 FHWA data submittal file, it transferred blanks instead. Brinfo was discontinued in year***

[2]Minnesota Rules 8810.9400, Frequency of Inspections and Inventory, Subpart 1 Inspection.
[3]Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges, U.S. Department of Transportation, Federal Highway Administration, Office of Engineering Bridge Division, December 1995, Appendix E.

***2000 due to frequent data errors and unreliability. Pontis has been used to store inspection data and create the FHWA submittal file since 2000.***

***Question:*** *Why did the sufficiency rating increase from 49 to 76 in 1999?*
***Response: Our 1999 FHWA submittal did not include the condition ratings for this bridge. FHWA submittal requirement does not include sufficiency ratings. They compute the sufficiency rating using their own software. We assumed that in the absence of deck, superstructure, and substructure condition data, the sufficiency rating is computed using 6 for deck, superstructure, and substructure, thereby producing a higher SR than actual conditions would produce. We confirmed this by entering a 6 for deck, super, and sub; and the SR went from the most recent value of 50, to 73."***

## 5.5 DESCRIPTION OF CONDITION RATING GUIDELINES

The description of the condition rating guidelines was contained in the Minnesota Department of Transportation Bridge Inspection Manual dated May 2007 (See Attachment 3 – Minnesota Department of Transportation Bridge Inspection Manual dated May 2007). Table 4 shows the description of the deck condition.

**Table 4 – NBI Deck Condition Description**

| Code | Description |
|---|---|
| N | **Not Applicable:** Use for culverts or bridges without decks (such as filled spandrel arches). |
| 9 | **Excellent Condition:** Deck is new condition (recently constructed). |
| 8 | **Very Good Condition:** Deck has superficial deterioration.<br>• Concrete: superficial cracking, leaching, scale or wear (no delamination, spalling, or temporary patches).<br>• Timber: superficial weathering – isolated splitting.<br>• Steel: no corrosion (paint/protection system remains sound). |
| 7 | **Good Condition:** Deck has minor (isolated) deterioration.<br>• Concrete: minor cracking, leaching, scale, or wear (isolated delamination or spalling).<br>• Timber: minor weathering or splitting (no decay or crushing) – all planks are secure.<br>• Steel: minor paint failure or corrosion (no section loss) – all connections are secure. |
| 6 | **Satisfactory Condition:** Deck has minor to moderate deterioration (no repairs are necessary).<br>• Concrete: moderate cracking, leaching, scale, or wear (minor delamination or spalling).<br>• Timber: moderate weathering or splitting (isolated decay or crushing) – some planks may be slightly loose.<br>• Steel: moderate paint failure and/or surface corrosion (minor section loss) – some connections may have worked loose. |

| | |
|---|---|
| 5 | **Fair Condition:** Deck has moderate deterioration (repairs may be necessary).<br>• Concrete: extensive cracking, leaching, scale, or wear (moderate delamination or spalling).<br>• Timber: extensive weathering or splitting (moderate decay or crushing) – some planks may be loose, broken, or require replacement.<br>• Steel: extensive paint failure and/or surface corrosion (moderate section loss) – several connections may be loose or missing, but all deck components remain secure. |
| 4 | **Poor Condition:** Deck has advanced deterioration (replacement or overlay should be planned).<br>• Concrete: advanced cracking, leaching, scale, or wear (extensive delamination or spalling) – isolated full-depth failures may be imminent.<br>• Timber: advanced weathering, splitting, or decay – numerous planks may be loose, broken, or require replacement.<br>• Steel: advanced corrosion (significant section loss) – deck components may be loose or slightly out of alignment. |
| 3 | **Serious Condition:** Deck has severe deterioration – immediate repairs may be necessary.<br>• Concrete: severe cracking, leaching, delamination, or spalling – full-depth failures may be present.<br>• Timber: severe splitting, crushing or decay – majority of planks may need replacement.<br>• Steel: severe and section loss – deck components may be severely out of alignment. |
| 2 | **Critical Condition:** Deck has failed – it may be necessary to close the bridge until repairs are completed. |
| 1 | **"Imminent" Failure Condition:** Bridge is closed – corrective action is required to open to restricted service. |
| 0 | **Failed Condition:** Bridge is closed – deck replacement is necessary. |

Table 5 shows the description of the superstructure condition.

**Table 5 – NBI Superstructure Condition Description**

| Code | Description |
|---|---|
| N | **Not Applicable:** Use for culverts. |
| 9 | **Excellent Condition:** Superstructure is in new condition (recently constructed). |
| 8 | **Very Good Condition:** Superstructure has superficial deterioration. |
| 7 | **Good Condition:** Superstructure has minor (isolated) deterioration.<br>• Steel: minor corrosion, little or no section loss.<br>• Concrete: minor scaling or non-structural cracking (isolated delamination or spalling).<br>• Timber: minor weathering or splitting (no decay or crushing).<br>• Masonry: minor weathering or cracking (joints have little or no deterioration). |

| | |
|---|---|
| 6 | **Satisfactory Condition:** Superstructure has minor to moderate deterioration. Members may be slightly bent or misaligned – connections may have minor distress.<br>• Steel: moderate corrosion (section loss or fatigue cracks in non-critical areas).<br>• Concrete: moderate scaling or non-structural cracking (minor delamination or spalling).<br>• Timber: moderate weathering or splitting (minor decay or crushing).<br>• Masonry: moderate weathering or cracking (joints may have minor deterioration). |
| 5 | **Fair Condition:** Superstructure has moderate deterioration. Members may be bent, bowed, or misaligned. Bolts, rivets, or connections may be loose or missing, but connections remain intact.<br>• Steel: extensive corrosion (initial section loss in critical stress areas). Fatigue cracks (if present) have been arrested or are not likely to propagate into critical stress areas.<br>• Concrete: extensive scaling or cracking (structural cracks may be present), moderate spalling or delamination (reinforcement may have some section loss).<br>• Timber: extensive weathering or splitting (moderate decay or crushing).<br>• Masonry: extensive weathering or cracking (joints may have slight separation or offset). |
| 4 | **Poor Condition:** Superstructure has advanced deterioration. Members may be significantly bent or misaligned. Connection failure may be imminent. Bearings may be severely restricted.<br>• Steel: significant section loss in critical stress areas. Un-arrested fatigue cracks exist that may likely propagate into critical stress areas.<br>• Concrete: advanced scaling, cracking, or spalling (significant structural cracks may be present – exposed reinforcement may have significant section loss).<br>• Timber: advanced splitting (extensive decay or significant crushing).<br>• Masonry: advanced weathering or cracking (joints may have separation or offset). |
| 3 | **Serious Condition:** Superstructure has severe deterioration – immediate repairs or structural evaluation may be required. Members may be severely bent or misaligned – connections or bearings may have failed.<br>• Steel: severe section loss or fatigue cracks in critical stress areas.<br>• Concrete: severe structural cracking or spalling.<br>• Timber: severe splitting, decay, or crushing.<br>• Masonry: severe cracking, offset or misalignment. |
| 2 | **Critical Condition:** Superstructure has critical deterioration – primary structural elements may have failed (severed, detached or critically misaligned). Immediate repairs may be required to prevent collapse or closure. The load-carrying capacity may be severely reduced. |
| 1 | **"Imminent" Failure Condition:** Bridge is closed – superstructure is no longer |

| | |
|---|---|
| | stable (corrective action might return the structure to restricted service). |
| 0 | **Failed Condition:** Bridge is closed – superstructure is beyond the point of corrective action. |

Table 6 shows the description of the substructure condition.

**Table 6 – NBI Substructure Condition Description**

| Code | Description |
|---|---|
| N | **Not Applicable:** Use for culverts or tunnels. |
| 9 | **Excellent Condition:** Substructure is in new condition (recently constructed). |
| 8 | **Very Good Condition:** Substructure has superficial deterioration. |
| 7 | **Good Condition:** Substructure has minor (isolated) deterioration.<br>• Concrete: minor cracking, leaching, or scale (isolated delaminations or spalls).<br>• Steel: minor paint failure and/or surface corrosion (little or no section loss).<br>• Timber: minor weathering or splitting (no decay or crushing).<br>• Masonry: minor weathering or cracking (joints have little or no deterioration). |
| 6 | **Satisfactory Condition:** Substructure has minor to moderate deterioration. Scour or erosion (if present) is minor and isolated. There may be slight movement or misalignment.<br>• Concrete: moderate scaling, cracking, or leaching (minor delamination or spalling).<br>• Steel: moderate paint failure and/or surface corrosion (minor section loss).<br>• Timber: moderate weathering or splitting (minor decay or crushing).<br>• Masonry: moderate weathering or cracking (joints may have minor deterioration). |
| 5 | **Fair Condition:** Substructure has moderate deterioration – repairs may be necessary. There may be moderate scour, erosion, or undermining. There may be minor settlement, movement, misalignment, or loss of bearing area.<br>• Concrete: extensive scaling, cracking or leaching (isolated structural cracks may be present) – there may be moderate delamination or spalling.<br>• Steel: extensive paint failure and/or surface corrosion (moderate section loss).<br>• Timber: extensive weathering or splitting (moderate decay or crushing).<br>• Masonry: extensive weathering or cracking (joints may have slight separation or offset). |
| 4 | **Poor Condition:** Substructure has advanced deterioration – repairs may be necessary to maintain stability. There may be extensive scour, erosion, or undermining. There may be significant settlement, movement, misalignment, or loss of bearing area.<br>• Concrete: advanced scaling, cracking, or leaching (significant structural cracks may be present) – there may be extensive delamination or spalling. |

| | |
|---|---|
| | • Steel: advanced corrosion (significant section loss).<br>• Timber: advanced splitting (significant decay or crushing).<br>• Masonry: advanced weathering or cracking (joints may have separation of offset). |
| 3 | **Serious Condition:** Substructure has severe deterioration. Immediate corrective action may be required. Scour, erosion, or undermining may have resulted in severe settlement, movement, misalignment, or loss of bearing area.<br>• Concrete: severe spalling or structural cracking.<br>• Steel: severe section loss.<br>• Timber: severe decay or crushing.<br>• Masonry: severe cracking, offset or misalignment. |
| 2 | **Critical Condition:** Substructure has critical damage or deterioration (near the point of collapse) – it may be necessary to close the bridge until corrective action is completed. Scour may have removed substructure support. |
| 1 | **"Imminent" Failure Condition:** Bridge is closed to traffic due to substructure failure – corrective action may restore the bridge to light service. |
| 0 | **Failed Condition:** Bridge is closed due to substructure failure – beyond corrective action (replacement required). |

## 5.6 SUPERSTRUCTURE CONDITION RATING DROPPED FROM 7 TO 4 IN 1991

NTSB investigators obtained a copy of the Mn/DOT Bridge Inspection Reports dated August 5, 1990 and October 18, 1993 (See Attachment 4 – Mn/DOT Bridge Inspection Reports dated August 5, 1990 and October 18, 1993). The Mn/DOT Bridge Inspection Report dated August 5, 1990 showed the superstructure received an overall condition rating of 7 because all of the elements (trusses, girders, floor beams, stringers or beams, bearing devices, arches, fascia beams, diaphragms, and spandrel columns) listed under the superstructure received a rating of 7 or above. The Mn/DOT Bridge Inspection Report dated October 18, 1993 showed the superstructure received an overall condition rating of 4 because one element (bearing devices) received a rating of 4 while the rest of the elements (trusses, girders, floor beams, stringers or beams, arches, fascia beams, diaphragms, and spandrel columns) received a rating of 7 or above. A few of the comments pertaining to the bearing devices in the October 18, 1993 Mn/DOT Bridge Inspection Report consisted of the following:

*"62) Last four bearing plates south abutment west side are quite rusty.*

*96) Bearings on Span #1 cantilever section are closed tight at 60 degrees F.*

*96) Bearing pins on truss bearing assemblies at ends of truss should be replaced with slightly longer bolts to allow for thermal thrust (on even expansion – due to temperature differences between girders and truss components."*

Listed below is a more comprehensive description of why the superstructure received a condition rating of 4 as mentioned in the Fracture Critical Bridge Inspection In-Depth Report for

Bridge #9340 dated June 2006[4] prepared by the Mn/DOT Metro District (Maintenance Operations - Bridge Inspection) (See Attachment 5 – Mn/DOT Fracture Critical Bridge Inspection In-Depth Report dated June 2006).

*"**Bridge Superstructure: NBI Condition Code 4***

***Paint System:**…Currently, the overall paint system is approximately 15% unsound. The truss members have surface rust corrosion and pack rust at the floorbeam & sway frame connections, and there is paint failure & surface rust corrosion in scattered locations. The floorbeam trusses & stringer ends have surface rust corrosion at the stringer expansion joints. Some of the areas re-painted in 1999 have severe section loss. This includes the sections of the floorbeam trusses & sway bracing located below the median, and the truss end floor beams & "crossbeams", located below the open finger joints.*

***Main Truss Members:**…The truss members have numerous poor weld details…The truss members have surface rust corrosion at the floor beam and sway frame connections. Pack rust is forming between the connection plates. There is paint failure, surface rust, and section loss, flaking rust in scattered locations.*

***Floor Beam Trusses:**…The floorbeam truss members have numerous poor welding details, including plug welded web reinforcement plates, and tack welds & welded connection plates located in tension zones. Some of the top chord splices are offset vertically, up to ½" – from original construction. The splice plates are bent. The floorbeam trusses below stringer joints have section loss, severe flaking rust. There is pack rust and surface pitting at the main truss connections.*

***Stringers:**…The stringer ends have surface rust corrosion at the expansion joints…The bolted connections to the floorbeam trusses are "working" and some bolts are loose or missing. [2006] Fascia stringers have minor section loss, with moderate flaking rust along the bottom flange.*

***Truss Bearing Assemblies:**…The truss bearings have section loss, flaking & surface rust; moderate corrosion, the bearings at piers #5 & 8 are functioning properly. They are checked during each annual inspection. The bearings at pier #6 show no obvious signs of movement, difficult to reach with snooper.*

***End Floor Beams:**…The sides facing the open finger joints have extensive section loss with surface pitting at the base of the web, and holes in the base of the vertical stiffeners. In 1998, fatigue cracks were found in two stiffener welds directly above the NE rocker bearing.*

[4]Fracture Critical Bridge Inspection In-Depth Report, Bridge #9340 (Squirt Bridge), I-35W over the Mississippi River at Minneapolis, MN, June 2006, Minnesota Department of Transportation Metro District, Maintenance Operations, Bridge Inspection, pages 11-14.

***Crossbeams & Rocker Bearings:****…The faces exposed to the finger joints have extensive surface pitting with some areas of severe section loss with holes at the base of stiffeners. The rocker bearings are measured & checked for movement during each annual inspection. All four bearings appear to be functioning. They show obvious signs of movement.*

*In 1986, the southeast rocker bearing "froze", resulting in damage to the crossbeam with two cracked vertical web stiffeners. The rocker-bearing pin was replaced. This required closing I-35W and jacking up the span. The crossbeam was repaired and the cracks in the web stiffeners were welded, crack ends drilled out, and stiffeners reinforced with angle plates. Installing braces between the crossbeam and beams #2 & 3 also reinforced the connection.*

*In 1992, a crack was found in a crossbeam stiffener weld above the northeast rocker bearing, which was drilled out. In 1997, at the same location, a weld between a vertical & horizontal stiffener was found cracked through entirely. Cracks were also discovered at the end of horizontal stiffeners near the northeast & southwest rocker bearings. Strain gauges were installed to analyze stresses, crack ends were drilled out, and installing bracing between the crossbeam and 2 stringers reinforced the northeast connection.*

***Steel Multi-Beam Approach Spans (spans #1–5 & #9-11):****…In 1998, fatigue cracks were found in several beam webs. These cracks were located in negative moment regions at the top of the diaphragm connections. At one location the web had cracked through entirely and was caused by out of plane bending in locations where the web stiffener was not rigidly connected to the top flange. After strain gauge analysis by the University of Minnesota, the diaphragm connections were modified. They were lowered, using only four bolts at each connection. Most existing cracks were drilled out. Some were too small to reach, and the fractured beam was reinforced with bolted plates.*

*In span #2, multi-beam approach span, there is a cantilever expansion hinge with sliding plate bearings. The joint is closed beyond tolerable limits, possibly due to substructure movement & pavement thrust and is no longer functioning. Some beam-ends are contacting, and some bearing plates have tipped, preventing the joint from reopening. The hinge area, with open finger joint above, was re-painted in 1999. The beam-ends have section loss, moderate surface pitting."*

## 5.7 CHANGES TO INSPECTION REPORT FORMAT

The bridge inspection report format had changed over the period of time the I-35W Bridge was constructed in 1967 until the day of the collapse. The first inspection report done by Mn/DOT was in 1971. The inspection report format used from 1971 through 1973 consisted of approximately 22 elements that were part of the substructure, superstructure, deck, channel protection, culverts, retaining wall, approaches, and signs. The inspection report format used from 1974 through 1987 consisted of approximately 24 elements that were part of the substructure, superstructure, deck, area under bridge, culverts, and other. The inspection report format used from 1988 through 1993 consisted of approximately 35 elements that were part of the substructure, superstructure, deck, area under bridge, culvert and wall, approach roadway, and other. The last inspection report format used by Mn/DOT from 1994 to date was element based to facilitate use of the Pontis bridge management software. A description of Pontis and the approximately 150 elements contained in the system is provided later in this report.

## 5.8 Mn/DOT BRIDGE SAFETY INSPECTION CERTIFICATION POLICY

Mn/DOT provided the process by which a person can become an assistant bridge inspector and a bridge inspection team leader in a December 19, 2007 letter to NTSB investigators (See Attachment 6 – Letter to the National Transportation Safety Board from the Minnesota Department of Transportation dated December 19, 2007). The following excerpts are taken from the letter:

> *"The requirements listed below have been developed by the Mn/DOT Bridge Office to comply with the National Bridge Inspection Standards (NBIS), as outlined in the Federal Code of Regulations Part 650.309, Minnesota Statute 165, and State of Minnesota Rule 8810.9300.*
>
> ***Mn/DOT Bridge Inspection Certification Levels***
>
> ***Assistant Bridge Inspector:*** *This inspection level is automatically assigned to anyone who has successfully completed the 1-week training course ("Engineering Concepts for Bridge Inspectors"). A Mn/DOT BSI certification number is assigned along with this inspection level. Note: an Assistant Bridge Inspector can only assist in bridge inspections – a certified Bridge Inspection Team Leader must be present at the bridge site at all times during a bridge inspection.*
>
> ***Bridge Inspection Team Leader:*** *A Bridge Inspection Team Leader can conduct inspections of in-service bridges & culverts on the state, county, and local highway system throughout the state of Minnesota. A certified Bridge Inspection Team Leader must be present at the bridge site at all times during a bridge inspection. There are five ways to qualify as a Bridge Inspection Team Leader:*
>
> *1. Be a registered professional engineer in the state of Minnesota, successfully complete a FHWA approved comprehensive bridge inspection training course, and pass a field proficiency test (administered by the Mn/DOT Bridge Office).*
>
> *2. Have five years of bridge inspection experience, successfully complete a FHWA approved comprehensive bridge inspection training course, and pass a field proficiency test (administered by the Mn/DOT Bridge Office).*

*3. Be certified by NICET (National Institute for Certification in Engineering Technologies) as a Level III or IV Bridge Safety Inspector, successfully complete a FHWA approved comprehensive bridge inspection training course, and pass a field proficiency test (administered by the Mn/DOT Bridge Office).*

*4. Have a bachelor's degree in engineering from an accredited college or university, successfully pass the National Council of Examiners for Engineering and Surveying Fundamentals of Engineering examination, have two years of bridge inspection experience, successfully complete a FHWA approved comprehensive bridge inspection training course, and pass a field proficiency test (administered by the Mn/DOT Bridge Office).*

*5. Have an associate's degree in engineering or engineering technology from an accredited college or university, have four years of bridge inspection experience, successfully complete a FHWA approved comprehensive bridge inspection training course, and pass a field proficiency test (administered by the Mn/DOT Bridge Office)."*

Mn/DOT offers two bridge inspection training courses each year. These courses were developed by the National Highway Institute (NHI), and are based upon the Bridge Inspectors Reference Manual (BIRM). Together, these two courses meet the definition of a "comprehensive training program in bridge inspection" as defined in the NBIS.

In addition to meeting NBIS qualifications, Mn/DOT also requires a bridge inspection team leader to pass a field proficiency test administered by the Bridge Office. The purpose of the test is to ensure compliance with NBIS standards, conform to Mn/DOT recording and coding practices, and to improve statewide consistency. The test consists of a routine inspection of an in-service bridge. The inspector is given 2 hours to examine a bridge, take notes, and determine the NBI & PONTIS condition ratings. Scoring is based on a scale of 0-100, with a passing score being 70 or more.

Certification of a bridge inspection team leader must be renewed every 4 years. To maintain certification, bridge inspection team leaders must meet the following two criteria:

1. The inspector must have attended a minimum of two refresher seminars during the four preceding years. These one-day seminars are conducted annually by the Mn/DOT Bridge Office, and

2. The inspector must have been actively engaged in bridge inspection during at least two of the four preceding years (the supervising engineer must verify this activity).

Mn/DOT employed approximately 75 bridge inspection team leaders. The 75 bridge inspection team leaders were responsible for inspecting approximately 3,500 bridges on the state highway system every 2 years. All bridge inspection team leaders had successfully completed an FHWA approved comprehensive bridge inspection training course, had a minimum of 5 years experience performing bridge safety inspections, or were professional engineers, and had passed a field proficiency test demonstrating their ability to perform bridge inspections. Only 8 of the 75 bridge inspection team leaders were assigned and regularly performed 200 fracture critical and special bridge inspections statewide. Three were engineers, three were engineering specialists, and two were certified welding inspectors with nondestructive testing (NDT) certifications.

### 5.8.1 NATIONAL BRIDGE TRAINING MANUALS

Requirements for bridge inspection are established in national standards and regulations including:

- National Bridge Inspection Standards (NBIS), Code of Federal Regulations Section 23, Highways, Part 650, subpart C-National Bridge Inspection Standards 2007.
- American Association of State Highway and Transportation Officials (AASHTO) "Manual for Condition Evaluation for Bridges", 2nd Edition dated 2000 and Interims.
- Bridge Inspector's Reference Manual (BIRM) dated December 2006 FHWA NHI 03-003.
- Federal Highway Administration (FHWA) Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges (Coding Guide) dated 1995.

The Transportation Research Board conducts and publishes research via the National Cooperative Highway Research Program (NCHRP). That research is typically proposed by AASHTO states or FHWA to further a transportation issue. Often those research results are adopted in whole or part by AASHTO or FHWA and incorporated in manuals such as those noted above. It is only after the NCHRP results are adopted by AASHTO or FHWA that they have standing as a national standard.

## 5.9 TOTAL NUMBER OF BRIDGES INSPECTED IN NBIS PROGRAM

The total number of bridges inspected in the NBIS Program is approximately 600,000 bridges. There are approximately 23 different types of bridges defined in the NBI. Some of the more common types of bridges include slab bridges, stringer / multi-beam or girder, girder and floorbeam system, tee beam, box beam or girders, frame, orthotropic, truss-deck, truss-thru, arch-deck, arch-thru, suspension, movable-bascule, moveable-swing, culvert, mixed types, and channel beam bridges. Table 7 shows the number of bridges broken down by sufficiency rating for all highway bridges.

**Table 7[5] – Number of bridges broken down by sufficiency rating for all highway bridges**

| Description | Sufficiency Rating (SR) 0 – 49.9 | Sufficiency Rating (SR) 50 - 80 | Sufficiency Rating (SR) > 80 | Total |
|---|---|---|---|---|
| All Highway Bridges | 70,158 | 175,188 | 354,651 | 599,997 |

Table 8 shows the number of fracture critical bridges for all highway bridges.

**Table 8[6] – Number of fracture critical bridges for all highway bridges**

| Description | Fracture Critical Bridges |
|---|---|
| All Highway Bridges | 19,273 |

Table 9 shows the number of structurally deficient and functionally obsolete bridges broken down by each state for all highway bridges. Table 9 shows the top 5 states with structurally deficient bridges were Pennsylvania, Oklahoma, Iowa, Missouri, and California.

**Table 9[7] – The number of structurally deficient and functionally obsolete bridges broken down by each state for all highway bridges**

| State | All Highway Bridges | Structurally Deficient | Functionally Obsolete |
|---|---|---|---|
| Alabama | 15,881 | 1,899 | 2,158 |
| Alaska | 1,229 | 155 | 179 |
| Arizona | 7,348 | 181 | 600 |
| Arkansas | 12,531 | 997 | 1,908 |
| California | 24,184 | 3,140 | 3,837 |
| Colorado | 8,366 | 580 | 824 |
| Connecticut | 4,175 | 358 | 1,042 |
| Delaware | 857 | 20 | 112 |
| District of Columbia | 245 | 24 | 128 |
| Florida | 11,663 | 302 | 1,692 |
| Georgia | 14,563 | 1,028 | 1,888 |
| Hawaii | 1,115 | 142 | 358 |
| Idaho | 4,104 | 349 | 452 |
| Illinois | 25,998 | 2,501 | 1,840 |
| Indiana | 18,494 | 2,030 | 2,004 |
| Iowa | 24,776 | 5,153 | 1,455 |
| Kansas | 25,461 | 2,991 | 2,372 |
| Kentucky | 13,637 | 1,362 | 2,928 |

[5]Table 7 represents December 2007 data from the Federal Highway Administration.
[6]ibid.
[7]ibid.

| | | | |
|---|---|---|---|
| Louisiana | 13,342 | 1,780 | 2,180 |
| Maine | 2,387 | 349 | 468 |
| Maryland | 5,127 | 388 | 980 |
| Massachusetts | 5,018 | 585 | 1,987 |
| Michigan | 10,923 | 1,584 | 1,304 |
| Minnesota | 13,067 | 1,156 | 423 |
| Mississippi | 17,007 | 3,002 | 1,315 |
| Missouri | 24,071 | 4,433 | 3,108 |
| Montana | 4,980 | 473 | 541 |
| Nebraska | 15,475 | 2,382 | 1,241 |
| Nevada | 1,705 | 47 | 156 |
| New Hampshire | 2,364 | 383 | 358 |
| New Jersey | 6,448 | 750 | 1,501 |
| New Mexico | 3,850 | 404 | 294 |
| New York | 17,361 | 2,128 | 4,518 |
| North Carolina | 17,783 | 2,272 | 2,787 |
| North Dakota | 4,458 | 743 | 249 |
| Ohio | 27,998 | 2,862 | 4,001 |
| Oklahoma | 23,524 | 5,793 | 1,614 |
| Oregon | 7,318 | 514 | 1,155 |
| Pennsylvania | 22,325 | 5,802 | 3,934 |
| Rhode Island | 748 | 164 | 232 |
| South Carolina | 9,221 | 1,260 | 808 |
| South Dakota | 5,924 | 1,216 | 261 |
| Tennessee | 19,838 | 1,325 | 2,776 |
| Texas | 50,271 | 2,186 | 7,851 |
| Utah | 2,851 | 233 | 254 |
| Vermont | 2,712 | 500 | 467 |
| Virginia | 13,417 | 1,208 | 2,234 |
| Washington | 7,651 | 400 | 1,661 |
| West Virginia | 7,001 | 1,058 | 1,515 |
| Wisconsin | 13,798 | 1,302 | 789 |
| Wyoming | 3,030 | 389 | 231 |
| Puerto Rico | 2,146 | 241 | 822 |
| | | | |
| **Totals** | **599,766** | **72,524** | **79,792** |

Table 10 shows the number of steel deck truss bridges in each state that are structurally deficient. Table 10 shows the top 4 states with steel deck truss bridges that are structurally deficient were California, Pennsylvania, Oregon, and Iowa. Minnesota had two steel deck truss bridges that were structurally deficient, the I-35W Bridge over the Mississippi River (Bridge #9340) and 1st Street over the Mississippi River (Bridge #5947). Only the 1st Street over the Mississippi River (Bridge #5947) is identified as structurally deficient in Table 10, since the I-35W Bridge over the Mississippi River (Bridge #9340) no longer exists.

**Table 10[8] – Number of steel deck truss bridges in each state that are structurally deficient**

| State | Number of Steel Deck Truss Bridges | Number of Steel Deck Truss Bridges that are Structurally Deficient |
|---|---|---|
| Alabama | 2 | 1 |
| Alaska | 7 | 3 |
| Arizona | 7 | 3 |
| Arkansas | 11 | 3 |
| California | 50 | 22 |
| Colorado | 6 | 1 |
| Connecticut | 5 | 2 |
| Delaware | 1 | 0 |
| District of Columbia | 0 | 0 |
| Florida | 0 | 0 |
| Georgia | 2 | 0 |
| Hawaii | 2 | 0 |
| Idaho | 7 | 2 |
| Illinois | 17 | 3 |
| Indiana | 9 | 5 |
| Iowa | 9 | 8 |
| Kansas | 13 | 6 |
| Kentucky | 9 | 2 |
| Louisiana | 2 | 0 |
| Maine | 6 | 1 |
| Maryland | 9 | 0 |
| Massachusetts | 18 | 4 |
| Michigan | 4 | 2 |
| Minnesota | 4 | 1 |
| Mississippi | 0 | 0 |
| Missouri | 4 | 1 |
| Montana | 9 | 2 |
| Nebraska | 2 | 1 |
| Nevada | 1 | 0 |
| New Hampshire | 3 | 1 |
| New Jersey | 8 | 5 |
| New Mexico | 5 | 0 |
| New York | 32 | 6 |
| North Carolina | 1 | 0 |
| North Dakota | 0 | 0 |
| Ohio | 16 | 3 |
| Oklahoma | 10 | 3 |
| Oregon | 37 | 12 |

[8]Table 10 represents December 2007 data from the Federal Highway Administration.

| | | |
|---|---|---|
| Pennsylvania | 48 | 16 |
| Rhode Island | 0 | 0 |
| South Carolina | 0 | 0 |
| South Dakota | 2 | 1 |
| Tennessee | 6 | 1 |
| Texas | 8 | 2 |
| Utah | 1 | 1 |
| Vermont | 8 | 6 |
| Virginia | 9 | 6 |
| Washington | 22 | 3 |
| West Virginia | 15 | 2 |
| Wisconsin | 14 | 3 |
| Wyoming | 3 | 1 |
| Puerto Rico | 2 | 0 |
| | | |
| **Totals** | **466** | **145** |

### 5.10 FHWA's APPORTIONMENT PROCESS FOR HIGHWAY BRIDGE PROGRAM (HBP) FUNDS

The apportionment process for Highway Bridge Program (HBP) funds is defined in 23 USC Section 144. FHWA is authorized to carry out this section in determining the apportionment of funds to each State. The apportionment process for HBP funds involves the following steps: (1) gather National Bridge Inventory (NBI) and bridge construction unit cost (BCUC) information from States and federal agencies; (2) identify eligible bridges; (3) compute State apportionment factors; and (4) compute the amount of HBP funds to be apportioned to each State. These steps and the intermediate steps in the process are discussed in Attachment 7 – FHWA's Apportionment Process for Highway Bridge Program (HBP) Funds.

## 6. Mn/DOT INSPECTION PROGRAM

### 6.1 Mn/DOT PONTIS ELEMENT CONDITION RATINGS

The 1991 Inter-Modal Surface Transportation Efficiency Act (ISTEA) mandated that all states develop and implement a Bridge Management System (BMS) by October of 1998. Mn/DOT adopted an element based bridge inspection format in 1994 that is compatible with a bridge management software called Pontis. Pontis is also licensed by approximately 43 states. The Pontis element condition ratings provide a detailed condition of the bridge by dividing the bridge into separate elements, which are then rated individually based upon the severity and extent of any deterioration. This rating system was developed by the American Association of State Highway and Transportation Officials (AASHTO), and is outlined in the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements.

An "element" refers to structural members (beams, pier columns, decks, etc.), or any other components (railings, expansion joints, approach panels, etc.) commonly found on a bridge. The Mn/DOT bridge inspection manual includes approximately 150 elements, including the AASHTO CoRe (commonly recognized) elements, as well as elements added by Mn/DOT to better represent the bridge types and components found in Minnesota.

The Mn/DOT Pontis element list is arranged in groups based upon the element type or the material. Each Pontis element is assigned a number – AASHTO CoRe deck elements are numbered between 1 and 99, AASHTO CoRe superstructure elements are numbered between 100 and 199, and AASHTO CoRe substructure elements are numbered between 200 and 299. Smart flag elements and elements added by Mn/DOT are numbered between 300 and 999 (elements higher than 370 were added by Mn/DOT).

The Pontis inspection data in combination with detailed spreadsheets are used by Mn/DOT to develop a 20 year plan to help identify bridges that need rehabilitation or replacement due to condition, age and traffic volume.

Mn/DOT does not typically inspect bridges in the winter months for practical and safety reasons. Frigid temperatures, storms, snow, and ice make it difficult to operate inspection equipment on the bridge. If a crack does occur in a steel member during the winter, it may not be detected until the next inspection. All cracks found are typically analyzed and repaired as soon as possible including during the winter months.

### 6.2 GUSSET PLATE NOT INCLUDED AS AN ELEMENT DESCRIPTION IN PONTIS

The Mn/DOT Pontis element list does not include an element name for gusset plates. The AASHTO Guide for Commonly Recognized (CoRe) Structural Elements would consider gusset plates a non-CoRe structural element because of the inability to accurately model a deterioration rate (for example missing bolts, etc.). The AASHTO Guide indicated that any significant problems with gusset plates would be noted in smart flag elements. The AASHTO Guide for Commonly Recognized (CoRe) Structural Elements[9] defined smart flags as the following:

> *"Smart flags are used to identify local problems that are not reflected in the CoRe element condition state language. The concept of Smart Flags is an option for tracking the severity of the flagged condition.*
>
> *A Smart Flag is similar to an element in that it will have multiple stages of deterioration. However, a Smart Flag does not have feasible actions. A Smart Flag is treated like an element in order to record the quantity or percentage of the distress feature and to track deterioration rates. The Smart Flags will allow States to track distress conditions in elements that do not follow the same deterioration or do not have the same units of measure as the distress described in the CoRe element."*

[9] AASHTO Guide for Commonly Recognized (CoRe) Structural Elements, American Association of State Highway and Transportation Officials, December 1997, page 4.

In reviewing the Mn/DOT Bridge Inspection Reports from 1971 through 2006, NTSB investigators found mention of loss of section to gusset plates in an October 18, 1993 Bridge Inspection Report (See Attachment 4 – Mn/DOT Bridge Inspection Report dated October 18, 1993). The report indicated the following:

> *"Additional Comments from October 13-18, 1993 Snooper Inspection.*
>
> *Downstream truss* (east truss) *at L11 inside gusset plate has loss of section 18" long and up to 3/16" deep (original thickness = ½").*
>
> *Downstream truss at L13 the lower horiz. brace between the trusses has 3/16" section loss at riveted angle."*

The gusset plate comments indicated in the October 18, 1993 Bridge Inspection Report are not repeated in subsequent Bridge Inspection Reports (from 1994 through 2006), however, the gusset plate comments are repeated in the Mn/DOT Fracture Critical Bridge Inspection Reports from 1994 through 2006. A colored photograph showing the loss of section to gusset plates at L11 (east truss) and L13 (east truss) first appeared in the 2004 Mn/DOT Fracture Critical Bridge Inspection Report[10] (See Attachment 8 – Mn/DOT Fracture Critical Bridge Inspection Annual Report dated June 2004).

Mn/DOT acknowledged in a September 21, 2007 letter to NTSB investigators that the Pontis system does not include an element name for gusset plates (See Attachment 9 – Letter to the National Transportation Safety Board from the Minnesota Department of Transportation dated September 21, 2007). Mn/DOT further indicated that any significant problems associated with gusset plates would be identified in the Pontis "smart flag" elements and fracture critical bridge inspection reports:

> *"AASHTO Developed the Pontis system, a CoRe element for gusset plates was not included. While Mn/DOT has added some custom elements, we have not added one for gusset plates. Likewise, there is no AASHTO CoRe element for pinned truss connections. Instead, the condition of these truss connections is inspected and considered when rating the AASHTO CoRe truss elements. There are Pontis "smart flags" elements for section loss, fatigue cracking, and impact damage – significant problems with any primary members or connections would be noted in these smart flag elements. Critical findings would be immediately reported. Mn/DOT also prepares supplementary "in-depth" inspection reports for fracture critical bridges (such as trusses). These reports generally include a section with field notes, which identify all of the truss members and connection panel points – any deterioration or damage to a gusset plate would be described in these notes. See the attached example from the Br 9340 2006 report."*

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[10]Fracture Critical Bridge Inspection Annual Report, Bridge #9340 (Squirt Bridge), I-35W over the Mississippi River at Minneapolis, MN, June 2004, Minnesota Department of Transportation Metro District, Maintenance Operations, Bridge Inspection, pages 22 and 23.

Mn/DOT acknowledged that recording of critical findings is not based upon any economic criteria:

> *"Neither inspectors nor engineers make decisions about recording critical findings based upon any economic criteria. Our inspection staff is empowered to take action when they find a condition they believe may impact bridge safety. They are conscientious regarding the responsibility for public safety. If conditions warrant, they will issue a Critical Finding. Mn/DOT expects to incur costs to maintain highways and bridges, is funded for such events, and routinely responds to these incidents. Construction or maintenance budgets are available if the Critical Finding requires a contract repair. Alternatively, each District has one or more Bridge Crews available for bridge repair work, and that is one of their primary functions. If a Critical Finding is identified, it is often the Bridge Crew that will implement the repair."*

6.3 Mn/DOT BRIDGE INSPECTION REPORT (JUNE 15, 2006)

The bridge inspector and bridge engineer have two distinct responsibilities. The bridge inspector is responsible for documenting corrosion, cracking in concrete or steel, condition of concrete such as spalling, fatigue cracks, substructure movements, paint condition, unusual deflections, bearing alignments, impact damage from vehicle or stream as well as many non-structural element conditions like joints, railings and approach slabs. The bridge inspector is also responsible for examining tension and compression members, especially fracture-critical members, for any evidence of a developing failure. The bridge engineer reviewing the inspection is responsible to interpret the degree of change in condition and recommend repairs if needed.

Mn/DOT provided the latest bridge inspection report for the I-35W Bridge (Bridge #9340) dated June 15, 2006 (See Attachment 10 – Mn/DOT Bridge Inspection Report dated June 15, 2006). The bridge inspection report included condition ratings for the following element names. The element names are categorized as to whether they are part of the deck, superstructure, substructure, or a miscellaneous element type.

**Deck**

- Low Slump Overlay (Concrete Deck with Uncoated Rebar) – Element #22
- Low Slump Overlay (Concrete Slab with Uncoated Rebar) – Element #48
- Strip Seal Deck Joint – Element #300
- Poured Deck Joint – Element #301
- Assembly Deck Joint (with or without seal) – Element #303
- Approach Relief Joint – Element #412
- Concrete Approach Slab (Concrete Wearing Surface) – Element #321
- Reinforced Concrete Bridge Railing – Element #331

**Superstructure**

- Painted Steel Girder or Beam – Element #107
- Painted Steel Stringer – Element #113

- Painted Steel Deck Truss – Element #131
- Painted Steel Floorbeam – Element #152
- Expansion Bearing – Element #311
- Fixed Bearing – Element #313
- Steel Hinge – Element #373
- Secondary Structural Elements – Element #380

**Substructure**

- Reinforced Concrete Column – Element #205
- Reinforced Concrete Pier Wall – Element #210
- Reinforced Concrete Abutment – Element #215
- Reinforced Concrete Pier Cap – Element #234

**Miscellaneous**

- Fatigue Cracking Smart Flag – Element #356
- Pack Rust Smart Flag – Element #357
- Concrete Deck Cracking Smart Flag – Element #358
- Underside of Concrete Deck Smart Flag – Element #359
- Substructure Settlement & Movement Smart Flag – Element #360
- Scour Smart Flag – Element #361
- Section Loss Smart Flag – Element #363
- Critical Finding Smart Flag – Element #964
- Fracture Critical Smart Flag – Element #966
- Signing – Element #981
- Approach Guardrail – Element #982
- Deck & Approach Drainage – Element #984
- Slopes & Slope Protection – Element #985
- Curb & Sidewalk – Element #986
- Miscellaneous Items – Element #988

The element names that received the worst condition ratings are shown below with an explanation of the condition state taken from the Mn/DOT Bridge Inspection Manual. The condition states are rated on a scale of 1-3, 1-4, or 1-5 (depending upon the element name). In all cases, condition state 1 is the best condition, with condition state 3, 4, or 5 being the worst condition (this is the reverse of the NBI condition ratings). The quantities listed under each element name are expressed as linear feet (LF) or each (EA). Also listed below are the notes taken from the Mn/DOT bridge inspection report for each element name. Each note entered in the report is dated (i.e. year) by the inspector. Past reports are in integral part of the Mn/DOT bridge inspection process. Mn/DOT bridge inspectors use past inspection reports as a check list and only pertinent changes in condition are noted.

**Strip Seal Deck Joint (Element #300)** – 94 linear feet (of a total 946 linear feet) received a condition state of 3. The Mn/DOT Bridge Inspection Manual indicated the following for condition state 3:

*"**Condition State 3:** Strip seal joint has severe deterioration – there may be significant leakage. Gland may be punctured, torn, or pulled loose. The joint anchorage may be damaged or deteriorated to the extent that the gland can no longer be properly anchored. Adjacent deck may have severe spalling. Joint may be severely misaligned – the function may be significantly impaired."*

The notes taken from the Mn/DOT bridge inspection report indicated the following:

*"Notes: [1978] Type H strip seal at abutments, pier 11, and stringer expansion joints (7 total). [1998] Strip gland replaced at pier 11, north abutment. South abutment joint (SBL) repaired with new product (hot pour with steel mesh). Steel extrusion was too corroded to install new gland. [1995] Pier 11 joint has numerous leaks (SBL & NBL), glands in the stringer joints have pulled out in scattered locations."*

**Poured Deck Joint (Element #301)** – 17 linear feet (of a total of 1,017 linear feet) received a condition state of 3. The Mn/DOT Bridge Inspection Manual indicated the following for condition state 3:

*"**Condition State 3:** Poured joint has severe deterioration – there may be significant leakage. Joint sealant may have failed. Adjacent deck may have severe cracking or spalling."*

The notes taken from the Mn/DOT bridge inspection report indicated the following:

*"Notes: Deck has 1,017 LF of transverse poured joints. [1997] All have leaching below (with some deck spalling)."*

**Assembly Deck Joint (with or without seal) (Element #303)** - 25 linear feet (of a total of 326 linear feet) received a condition state of 3. The Mn/DOT Bridge Inspection Manual indicated the following for condition state 3:

*"**Condition State 3:** Assembly joint has severe deterioration. Seals may have failed. Joint components may be missing. Steel components may have severe section loss. Adjacent deck may have severe spalling. Joint may be severely misaligned – joint function may be significantly impaired."*

The notes taken from the Mn/DOT bridge inspection report indicated the following:

*"Notes: Open finger joints at truss ends and span 2 hinge. [1998] Rubber "skirts" installed below truss end finger joints. The face exposed to the open finger joints have extensive section loss (surface pitting & holes in stiffeners)."*

**Painted Steel Girder or Beam (Element #107)** – 196 linear feet (of a total of 10,596 linear feet) received a condition state of 4. The Mn/DOT Bridge Inspection Manual indicated the following for condition state 4:

*"**Condition State 4:** Painted steel element has extensive deterioration – repairs may be required, but the load-carrying capacity of the element has not been significantly reduced. There may be severe corrosion, with extensive flaking rust. While there may be significant section loss, structural analysis is not yet required (section loss is less than 10% of the effective section). Connections may have started to come loose – element may be out of proper position or alignment."*

The notes taken from the Mn/DOT bridge inspection report indicated the following:

*"Notes: [1968] Bridge painted with lead base system. Approach spans have welded beams (depth transitions from 48" to 33"), with riveted connections. Spans 1 & 2 have 33" deep rolled beams with welded cover plates (square ends). [1995] Beams have salt film, minor chalking throughout, fascia beams have section loss: pitting, flaking & surface rust along the bottom flange. [1999] Beams along median (and at hinge) re-painted. Spot painting contract: truss ends, hinge joints, and area below median painted with zinc system. Paint system is 15% unsound."*

**Painted Steel Stringer (Element #113)** - 196 linear feet (of a total of 14,896 linear feet) received a condition state of 4. The explanation for condition state 4 for Painted Steel Stringer (Element #113) was the same for Painted Steel Girder or Beam (Element #107).

The notes taken from the Mn/DOT bridge inspection report indicated the following:

*"Notes: 27" deep rolled stringers (truss spans). [1995] Stringers have section loss: pitting, flaking & surface rust corrosion at expansion joints. [1999] Median stringers re-painted. [91,2000] Stringer / floorbeam connections are "working". Several bolts are loose or missing."*

**Painted Steel Deck Truss (Element #131)** - 247 linear feet (of a total of 2,127 linear feet) received a condition state of 4. The explanation for condition state 4 for Painted Steel Deck Truss (Element #131) was the same for Painted Steel Girder or Beam (Element #107).

The notes taken from the Mn/DOT bridge inspection report indicated the following:

*"Notes: Main truss members have numerous poor weld details (some cracked tack welds). [1995] Interiors of truss members have section loss: pitting, flaking & surface rust, severe pigeon debris, at the floorbeam & sway frame brace connections (with pack rust & surface pitting). [1999] Pigeons screens placed on truss member openings."*

**Painted Steel Floorbeam (Element #152)** - 623 linear feet (of a total of 3,348 linear feet) received a condition state of 4. The explanation for condition state 4 for Painted Steel Floorbeam (Element #152) was the same for Painted Steel Girder or Beam (Element #107).

The notes taken from the Mn/DOT bridge inspection report indicated the following:

*"Notes: [1986] Crossbeam web stiffeners cracked at SE rocker hinge (rocker bearing had frozen). Cracks were welded / drilled out, and bracing was added (attached to approach span beams). [1992/98] Several cracks found in crossbeam & end floorbeam at the NE rocker hinge. Some cracks were drilled out, and bracing was added (attached to approach span beams). [1998/99] End floorbeams & crossbeams re-painted. Floorbeam trusses have numerous poor weld details, section loss: pitting, flaking & surface rust, some have holes, (plug welds & tack weld in tension zones). [1994] Floorbeam trusses have salt film, chalking throughout. [1999] Median portions of floorbeam trusses (and sway braces) re-painted."*

**Steel Hinge (Element #373)** - 14 each (of a total of 18 each) received a condition state of 4. The Mn/DOT Bridge Inspection Manual indicated the following for condition state 4:

*"**Condition State 4:** Steel hinge bearing assembly has extensive deterioration – bearing function may be impaired, but the load-carrying capacity has not been significantly reduced. Debris or corrosion may be restricting movement (cleaning and/or lubrication may be required). Primary bearing components (rockers, rollers, sliding plates, elastomeric pads, pins, etc.) may have extensive wear (or deterioration), or may be misaligned. Longitudinal alignment may be at the design limits (contacting or binding), or may be completely inappropriate for the current temperature. Lateral restraint/guide systems may have failed, or there may be excessive lateral misalignment. Paint system may have failed – there may be extensive corrosion, with significant section loss. Supporting steel superstructure may have extensive deterioration."*

The notes taken from the Mn/DOT bridge inspection report indicated the following:

*"Notes: [1986] SE crossbeam rocker hinge pin replaced. Section loss at hinges, (open finger joint) steel has moderate pitting, flaking & surface rust. [1999] Crossbeam rocker hinge bearings re-painted (all show evidence of recent movement). [1995] Span 2: all hinge bearings are locked in full expansion (beam ends contacting). [1999] Span 2 hinge bearings re-painted."*

**Expansion Bearing (Element #311)** – 6 each (of a total of 125 each) received a condition state of 3. The Mn/DOT Bridge Inspection Manual indicated the following for condition state 3:

*"**Condition State 3:** Expansion bearing has severe deterioration, and is no longer functioning as intended (repair or replacement may be necessary). Bearing alignment may be beyond design limits. Bearing mechanism may be frozen (seized) or severely restricted due to corrosion or debris. Primary bearing components (sliding plates, rockers, rollers, pins, etc.) may have severe section loss, wear, or misalignment – they may have jammed, come loose or otherwise*

*failed. The lateral guide/restraint system (guide tabs, keeper bars, pintles, or pin caps) may have sheared off, bound, or otherwise failed. Uplift restraint system may have failed. Anchor bolts may have failed. Bearing seat may have severe deterioration (there may be significant loss of bearing area) – supplemental supports or load restrictions may be warranted."*

The notes taken from the Mn/DOT bridge inspection report indicated the following:

*"Notes: [94/2000] Some abutment bearings are rusty (joints leaking). [1996] South abutment bearings are in full contraction. [1994] Main truss roller bearings have section loss: pitting, flaking & surface rust, moderate corrosion."*

**Pack Rust Smart Flag (Element #357)** – The quantity for this element was coded a total of 1 each and received a condition state of 3. The Mn/DOT Bridge Inspection Manual indicated the following for condition state 3:

*"**Condition State 3:** Pack rust has resulted in significant distress to a steel element or connection. There may be significant spreading, swelling, or scalloping – steel members may be significantly deformed or distorted. However, all connectors (pins, rivets, or bolts) remain intact."*

The notes taken from the Mn/DOT bridge inspection report indicated the following:

*"Notes: [1995] Truss members have flaking & surface rust corrosion at the floorbeam & sway brace connections (with pack rust & some section loss, surface pitting)."*

**Deck & Approach Drainage (Element #984)** – The quantity for this element was coded a total of 1 each and received a condition state of 3. The Mn/DOT Bridge Inspection Manual indicated the following for condition state 3:

*"**Condition State 3:** Drainage system has failed – repairs are required. Severe ponding may present a traffic hazard. Runoff may have resulted in severe slope erosion (or significant deterioration of bridge elements). Drainage components may be disconnected, missing, or severely deteriorated."*

The notes taken from the Mn/DOT bridge inspection report indicated the following:

*"Notes: Pier 6: horizontal drain trough has inadequate slope (usually clogged). [1998/99] Drain troughs below truss end finger joints removed & replaced with rubber "skirts". [2000] "Skirts" above crossbeam rockers are clogged."*

## 6.4 FRACTURE CRITICAL BRIDGE INSPECTION IN-DEPTH REPORT, JUNE 2006

The Mn/DOT Metro District (Maintenance Operations - Bridge Inspection) prepared a Fracture Critical Bridge Inspection In-Depth Report for Bridge #9340 dated June 2006[11] (See Attachment 5 – Mn/DOT Fracture Critical Bridge Inspection In-Depth Report dated June 2006). The in-depth report indicated the following:

*"**Long Term Repair Recommendations***

- *The long term plans for this river crossing need to be defined with replacement, redecking, etc. Due to the "Fracture Critical" configuration of the main river spans and the problematic "crossbeam" details, and fatigue cracking in the approach spans, eventual replacement of the entire structure would be preferable.*

- *If bridge replacement is significantly delayed, the bridge should be re-decked. The design of the main river spans do not allow for deck widening. Any re-decking contract should also include a complete re-painting of the superstructure, elimination of the hinge joint in span #2, and reconfiguration of the deck drainage system.*

- *Depending on the projected date of bridge replacement, the bridge deck will eventually require a partial overlay repair contract. The expansion joints should also be replaced.*

***Immediate Maintenance Recommendations***

- *The plastic pigeon screens were removed on all tension and reversal members to visually inspect the member's internal diaphragms any questionable welding flaws discovered during this inspection were tested with magnetic particle equipment. These areas should be inspected during the next in-depth inspection.*

- *Fatigue cracks at girder #1C (NBL), crack at the diaphragm bottom cutout, NE side measures 2" ("front face") and NW side measures 2-1/2" ("back face"). Fatigue cracks a girder #3 (NBL), crack at the diaphragm bottom cutout, measures 1-1/2" (both sides). The cracks are located in negative moment regions where the diaphragm web stiffener was not welded to the top flange and where previous fatigue cracks occurred and were repaired in 1998 and 1999. These areas should be inspected next year for any lengthening of the cracks and drilling of possible stress relief holes.*

[11] Fracture Critical Bridge Inspection In-Depth Report, Bridge #9340 (Squirt Bridge), I-35W over the Mississippi River at Minneapolis, MN, June 2006, Minnesota Department of Transportation Metro District, Maintenance Operations, Bridge Inspection, pages 8-9.

- *Four-stringer connection bolts, all in the NBL, need replacement. At panel point #8, stringer #2 has 2 loose bolts, and the bearing block has rotated. This will likely require jacking the superstructure. Stringer bolts also need replacement at panel point #8, stringer #4, south side, and at panel point #11, stringer #3.*

- *Several strip seal joints are leaking. The glands have ripped or pulled out. Attempts were made to replace these joints during the 1998 repair contract, but the steel extrusions, which anchor the gland, had severe corrosion, and new glands could not be installed. Instead, a new product was used at the, SBL, south abutment. This utilized a hot pour seal with wire mesh reinforcing. The final product looks similar to a strip seal gland. We should monitor this joint to see how well this new gland repair performs, and consider using it at other locations.*

- *The rubber "skirts" sections above the truss end rockers, installed in 1999, tend to fill with debris. These should be flushed out annually. The horizontal drain troughs at pier #6 have inadequate slope, and are clogged.*

***Areas of Concern – Future Inspections***

- *Span 3, stringer #7 NB, has a 1-1/2" crack in the web with one 2" hole drilled. It is recommended to drill a 2" hole at the other end.*

- *During the 1998 inspection, numerous fatigue cracks were found in spans #3 – 5 and #9 – 10, the approach spans. The cracks were located in negative moment regions where the diaphragm web stiffener was not welded to the top flange. At one location the web had cracked through entirely. Most existing cracks were drilled out, and the fractured beam was reinforced with bolted plates. To reduce the stress levels, the diaphragms were lowered. Due to the widespread cracking, these areas should be inspected in-depth on an annual basis.*

- *The truss end rocker bearings & main truss bearings should be measured for movement during each annual inspection. The truss end floor beams & approach end "crossbeams" should be closely inspected. They have section loss, had flaking rust & fatigue cracks (open finger joint).*

- *The hinge joint in span #2 is locked in full expansion several beam-ends are contacting, and the hinge bearings are "frozen" and no longer functioning. Consequently, pier #1 has tipped slightly to the north, and the south abutment bearings are in full contraction. This area should be thoroughly inspected."*

### 6.5 Mn/DOT GUIDELINES FOR IN-DEPTH INSPECTION OF FRACTURE CRITICAL AND OTHER NON-REDUNDANT BRIDGES AND FOR UNDERWATER INSPECTIONS (JULY 19, 2007)

Mn/DOT provided a technical memorandum regarding guidelines for in-depth inspection of fracture critical and other non-redundant bridges and for underwater inspections dated July 19, 2007[12] (See Attachment 11 – Mn/DOT Guidelines for In-Depth Inspection of Failure Critical and Other Non-Redundant Bridges and for Underwater Inspections dated July 19, 2007). The technical memorandum provided a definition of a fracture critical bridge and outlined the general guidelines that included frequency of in-depth inspections, who is responsible for the inspections, who maintains the information files, the qualifications of the inspector, and the content of the narrative reports. The technical memorandum defined a fracture critical bridge as the following:

> *"**Definition***
> *A Fracture Critical (FC) Bridge is a bridge that is not load path redundant and that has at least one fracture critical member or member component. Fracture critical members or member components (FCM's) are steel tension members or steel tension components of members whose failure would be expected to result in collapse of the bridge."*

### 6.6 Mn/DOT "CRITICAL DEFICIENCIES" FOUND DURING BRIDGE INSPECTIONS (JULY 20, 2005)

Mn/DOT provided a technical memorandum regarding "critical deficiencies" found during bridge inspections dated July 20, 2005[13] (See Attachment 12 – Mn/DOT Technical Memorandum regarding "Critical Deficiencies" found during bridge inspections dated July 20, 2005). The technical memorandum was divided into three parts that included responsibilities of the bridge inspector, responsibilities of the engineer, and responsibilities of the Mn/DOT Bridge Office. The technical memorandum indicated the following:

> *"**Critical Deficiency:** A "Critical Deficiency" is defined as any condition discovered during a scheduled bridge inspection that threatens public safety and, if not promptly corrected, could result in collapse or partial collapse of a bridge. Critical findings include structural conditions and scour or hydraulic conditions that are found to be critical during the inspection or that are likely to become critical to the stability of the bridge before the next regularly scheduled inspection."*

[12] The version of the July 19, 2007 Technical Memorandum was a draft but did reflect the operating procedures Mn/DOT had adopted at that time. The final version was recently published as Technical Memorandum 08-01-B-01 titled Guidelines for Fracture Critical Inspections of Fracture Critical Bridges, Special Inspections for other Bridges, and for Underwater Inspections.

[13] "Critical Deficiencies" found during bridge inspections, Minnesota Department of Transportation, Program Support Division, Technical Memorandum No. 05-02-B-02, July 20, 2005.

## 6.7 URS FATIGUE EVALUATION AND REDUNDANCY ANALYSIS DRAFT REPORT, EXECUTIVE SUMMARY JANUARY 2007

URS Corporation prepared a Fatigue Evaluation and Redundancy Analysis Draft Report for Bridge #9340 that contained an executive summary dated January 2007[14] (See Attachment 13 – URS Corporation Fatigue Evaluation and Redundancy Analysis Draft Report for Bridge #9340, Executive Summary, dated January 2007). The executive summary indicated the following:

*"…The following table lists the identified 13 fracture critical truss members on one half of each truss. Due to the double symmetry of the deck truss, there are a total of 52 fracture critical main truss members on the bridge structure…*

*…The fracture critical members can be divided into two general groups: (1) relatively more fatigue sensitive members (L1-L2, L2-L3, U0-U1, U1-U2, U4-U5, and U5-U6), these members are subject to higher fatigue life check for Category E, but are subjected to lower total stresses and have thinner web plates that are more forgiving for brittle fracture; and (2) relatively more fracture sensitive members (L11-L12, L12-L13, L13-L14, U6-U7, U7-U8, U8-U9, and U9-U10), these members have larger cross sections and are subject to very low fatigue load stress ranges, satisfying all AASHTO infinite fatigue life checks for Category E, but are subjected to higher total stresses and have thicker web plates that do not tolerate the existence of through-thickness cracks before the occurrence of brittle fracture…*

*…Based on the analysis results described in this report, three equally viable retrofit approaches are recommended as follows:*

*(1) Steel plating of all 52 fracture critical truss members. This approach will provide member redundancy to each of the identified fracture critical members via additional plates bolted to the existing webs. The critical issue of this approach is to ensure that no new defects are introduced to the existing web plates through the drilled holes. This approach is generally most conservative but its relatively high cost may not be justified by the actual levels of stresses the structure experiences.*

*(2) Non-destructive examination (NDE) and removal of all measurable defects at suspected weld details of all 52 fracture critical truss members. The critical issue of this approach is to ensure that no measurable defects are missed by the NDE efforts. The fracture mechanics analysis has indicated that the dimensions of preexisting surface cracks need to be at least one quarter of the web plate thickness in order to grow and subsequently cause member fracture under the traffic load. This approach is most cost efficient.*

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[14]Fatigue Evaluation and Redundancy Analysis Draft Report, Bridge #9340, I-35W over Mississippi River, URS Corporation, Executive Summary, January 2007, pages 1-4.

*(3) A combination of the above two approaches: steel plating of the 24 more fatigue sensitive members (L1-L2, L2-L3, U0-U1, U1-U2, U4-U5, and U5-U6 in each half of the truss), and NDE of the 28 more fracture sensitive members (L11-L12, L12-L13, L13-L14, U6-U7, U7-U8, U8-U9, and U9-U10 in each half of each truss)."*

## 6.8 URS FATIGUE EVALUATION AND REDUNDANCY ANALYSIS DRAFT REPORT, JULY 2006

URS Corporation prepared a Fatigue Evaluation and Redundancy Analysis Draft Report for Bridge #9340 dated July 2006[15] (See Attachment 14 – URS Corporation Fatigue Evaluation Redundancy Analysis Draft Report for Bridge #9340 dated July 2006). The draft report indicated the following:

*"…As shown in the tables, five of the eight critical members are fracture critical, i.e., their failure would result in the failure of at least one other main truss member and thus cause instability of the structural system. The five fracture critical main truss members are: Lower Chord L1-L2, Upper Chord U0-U1, Upper Chord U4-U5, Lower Chord L12-L13, and Lower Chord L13-L14. These five members actually represent twenty main truss members due to the nearly double symmetry of the trusses…*

***Recommendations***

*Based on results of our study, the following recommendations are made:*

*(1) Five main truss members in one half of each truss, representing twenty members in the bridge, have been identified as fracture critical and should be retrofitted with the steel plating scheme developed, using high performance steel and high strength bolts. The retrofit, although not changing the fracture critical nature of the truss member, adds internal redundancy to the member and eliminates the possibility of a member fracture due to the fatigue of susceptible welded details at the internal diaphragms.*

*(2) Before the retrofit takes place, the fatigue susceptible details at the internal diaphragms inside the identified fracture critical truss tension chords should be inspected with the access hole cover plates removed during the normal inspections. The toe of the longitudinal fillet weld between the tab and the truss chord web is a primary location for the development of a fatigue crack.*

*(3) A deck replacement with a new deck that is continuous throughout the main truss spans, and composite with the truss system, can significantly*

[15]Fatigue Evaluation and Redundancy Analysis Draft Report, Bridge #9340, I-35W over Mississippi River, URS Corporation, July 2006, pages 10-14.

*reduce live load stresses in most truss members and improve the redundancy of the truss system. To minimize dead load stresses, the replacement deck should be placed in two stages, with a structural deck of minimum required thickness, plus an overlay. Alternatively, the use of light-weight concrete for the new deck can also reduce dead load effects and should be evaluated in the final design of the deck replacement.*

*(4) A preliminary analysis using the LRFD design load indicated that member forces in the main trusses and the floor trusses are no higher in a transversely unbalanced half-deck condition than the full-deck condition. However, since truss bridges have generally been designed with symmetrical dead load between the two trusses, it is more desirable to keep this symmetrical loading condition during deck replacement as much as possible. If the unbalanced half-deck procedure is to be considered, a more complete detailed analysis should be performed in the final design to evaluate the impact on all transverse members and their connections between the two main trusses.*

*(5) Based on a map of the deck longitudinal axial stress contours provided, the sequence of deck concrete pouring can be determined for placing concrete in the compression areas first and tension areas last."*

## 6.9 URS FATIGUE EVALUATION SECOND INSPECTION REPORT, NOVEMBER 2003

URS Corporation prepared a Fatigue Evaluation Bridge 9340 35W Over Mississippi River Second Inspection Report dated November 17, 2003[16] (See Attachment 15 – URS Corporation Fatigue Evaluation Bridge 9340 35W over Mississippi River Second Inspection Report dated November 17, 2003). The report indicated the following:

*"**Summary***

*Movement of the bearings was somewhat inconsistent when comparing the east truss to the west truss. Future measurements of movement will be recorded at dates with hopefully a greater amount of temperature variation from the initial readings."*

[16]Second Inspection Report for Fatigue Evaluation, Bridge #9340, 35W over Mississippi River, URS Corporation, November 17th, 18th, 2003, page 4.

### 6.10 URS FATIGUE EVALUATION INITIAL INSPECTION REPORT, JUNE 2003

URS Corporation prepared a Fatigue Evaluation Bridge 9340 35W Over Mississippi River Initial Inspection Report dated June 9, 2003[17] (See Attachment 16 – URS Corporation Fatigue Evaluation Bridge 9340 35W over Mississippi River Initial Inspection Report dated June 9, 2003). The report indicated the following:

*"**Summary and Recommendations***

*The overall condition of the truss members and connections was, from a corrosion standpoint, found to be good. Corrosion was found in localized areas, generally concentrated near the deck joints. Minor corrosion was observed at some of the locations chosen to inspect in the interior of the truss members.*

*The roller bearings did not appear to be moving freely due to the corrosion, debris and paint build up. The rocker bearings were not accessible for detailed visual observation and assessment of their movement. All of the bearings were marked in their current position and temperature readings were recorded to assist in determining movement-temperature relationships.*

*The fracture critical details at the tab locations on the interior of the box chord are very difficult to observe. The access openings are covered and observations can only be made after the cover plate is removed. It is our understanding that the cover plates are not being removed as part of Mn/DOT's regular inspection cycle. Mn/DOT should consider inspection of all of these fracture critical details as part of the normal inspection cycle due to the fracture potential of these details. Inspection of these details is clearly the most important part of future inspections of this structure. It is also recommended that scope equipment be procured to enable close visual inspection of these details."*

### 6.11 UNIVERSITY OF MINNESOTA FATIGUE EVALUATION OF THE DECK TRUSS OF BRIDGE 9340, MARCH 2001

The University of Minnesota prepared a Fatigue Evaluation of the Deck Truss of Bridge 9340 Final Report dated March 2001[18] (See Attachment 17 – University of Minnesota Fatigue Evaluation of the Deck Truss of Bridge 9340 Final Report dated March 2001). The University of Minnesota staff included a renowned fatigue expert who inspected and strain gauged the bridge as part of a live load test. The report indicated the following:

*"The main conclusions were:*

[17]Initial Inspection Report for Fatigue Evaluation, Bridge #9340, 35W over Mississippi River, URS Corporation, June 9th-June 13th, 2003, page 5.

[18]Heather M. O'Connell, Robert J. Dexter, P.E., Paul Bergson, P.E., Fatigue Evaluation of the Deck Truss of Bridge 9340, Final Report, March 2001, pages 77-79.

1. *Inspection of the bridge revealed Category D details on the main truss members and Category E members on the floor truss. No fatigue cracks were found by visual inspection of those members.*

2. *The largest stress range measured in the main truss during the controlled tests was 12.5 MPa in the lower chord, from three rows of three trucks. The analyses show that member U4U6 would have the largest stress range from this loading, 46 MPa. This is less than the fatigue threshold for the most critical details on these members, which is 48 MPa for Category D.*

3. *The largest stress range in the main truss during the open-traffic monitoring was 22 MPa and this was in another member, L3U4.*

4. *The agreement of the analyses with the measured stress ranges was best when a three-dimensional model of the whole bridge was analyzed. In both the two-dimensional and three-dimensional analyses, the agreement was best if the roller bearings at the piers were assumed to be pinned so that a horizontal reaction developed and arching action occurred.*

5. *The largest stress range measured in the floor truss during the controlled tests was 28 MPa in the lower chord, from three rows of trucks in the leftmost lane (closest to the center) in each direction. This is less than the fatigue threshold of 31 MPa for a Category E detail.*

6. *The largest stress range in the floor truss during the open-traffic monitoring was 25 MPa and this was in a diagonal.*

7. *Two-dimensional analyses were adequate for the floor truss. Very poor agreement with the measured results was obtained unless some composite action with the deck was assumed. Full composite action was too much, and optimal results were obtained by averaging the results from the non-composite case and the fully composite case.*

8. *Since the measured and calculated stress ranges were less than the fatigue threshold, it is concluded that fatigue cracking is not expected in the deck truss of this bridge.*

9. *Live-load stress ranges greater than the fatigue threshold can be calculated if the AASHTO lane loads are assumed. The actual measured stress ranges are far less primarily because the loading does not frequently approach this magnitude. While the lane loads are appropriate for a strength limit state (the loading could approach this magnitude a few times during the life of the bridge), only loads that occur more frequently than 0.01% of occurrences are relevant for fatigue. For this bridge with 15,000 trucks per day in each direction, only loads that occur on a daily basis are important for fatigue.*

*The following actions are recommended:*

1. *The members of the main truss with the highest stress ranges are U2L3, L3U4 and U4U6. These members should be inspected thoroughly, especially at the ends of the "clips" on the diaphragms in the tension members and at any intermittent fillet welds. These members should be inspected every two years as is presently done.*

2. *The lower chords and diagonals of all the floor trusses also have high stress ranges. The ends of the "fin" attachments reinforcing the splice welds are the most critical locations. Since these can be inspected easily from the catwalk, they could be inspected every 6 months."*

### 6.12 Mn/DOT MEMORANDUMS REGARDING CRACKS IN NORTH APPROACH SPAN GIRDER NEAR PIER 9

Mn/DOT provided memorandums documenting cracks in the north approach span girder near Pier 9 from October 1998 through November 2000 (See Attachment 18 – Mn/DOT Memorandums documenting cracks in the north approach span girder near Pier 9 from October 1998 through November 2000). In October 1998, Metro bridge inspectors noticed 12 crack locations in the 48" deep approach span girders at the top of the stiffener/diaphragm connection near Pier #9 at the north end of the bridge. Inspectors noticed a large inverted U-shaped crack more than 50" long in a 48" deep welded approach span girder about 20 feet south of Pier 9. Eleven other cracks were located at the web toe of the web to top flange weld in the base metal. The cracks were located in a negative moment region and were in tension. Mn/DOT recommended that close in-depth inspections be performed in these areas on a 6 month cycle.

Mn/DOT consulted with the University of Minnesota to develop a repair method and devise a plan to strengthen the girder web and prevent further propagation of the crack. It was determined that the cracks were caused by out-of-plane bending of the web plates. The University of Minnesota recommended that the rigid diaphragm connections be released, but the diaphragms not be removed entirely because they were necessary for bracing of the girders. The procedure reduced the stress that was causing the cracks. The procedure included repositioning the diaphragms by lowering and reattaching them near the bottom flange of the girders. To confirm the procedure, the University of Minnesota performed load tests on two uncracked girders.

In November 2000, Metro bridge inspectors found no additional cracks had formed in the north approach span girder near Pier 9 since 1998, and recommended that the inspection frequency be decreased to a 12 month cycle beginning with the 2001 inspection.

6.13 Mn/DOT UNDERWATER BRIDGE INSPECTION REPORT, DECEMBER 2004

Ayres Associates prepared an Underwater Bridge Inspection Report Trunk Highway No. 35W over the Mississippi River for Bridge #9340, inspection date December 8, 2004[19] (See Attachment 19 – Ayres Associates Underwater Bridge Inspection Report for Bridge #9340 dated December 8, 2004). The report indicated the following:

*"**Comprehensive Report of Deficiencies***

*The concrete surfaces below the water are in good condition.*

*Minor scaling was found above the water, but not of the quantity or depth as noted in a previous report. The total area was 2.0 feet square and ¼-inch deep penetration.*

*No other significant changes in the structure or channel condition have occurred since the last inspection.*

***Recommended Corrective Action***

*None."*

6.14 INTERVIEWS OF Mn/DOT BRIDGE INSPECTORS

The NTSB conducted four (4) interviews of Mn/DOT bridge inspectors on August 17, 2007 and August 20, 2007. The contents of the interviews were transcribed and have been entered into the public docket of the accident investigation.

## 7. HISTORY OF CONSTRUCTION PLANS FOR DECK REPAIR

7.1 1977 CONSTRUCTION PLAN FOR REPAIR OF BRIDGE #9340

The 1977 construction plan[20] for the repair of Bridge #9340 involved scarifying approximately ¼" off the bridge deck surface and adding 1 ½" of latex modified concrete or 2" of low slump concrete wearing course.

Mn/DOT explained the reasons for increasing the average thickness of the concrete deck in a January 10, 2008 email to NTSB investigators (See Attachment 20 – Email to the National Transportation Safety Board from the Minnesota Department of Transportation dated January 10, 2008):

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[19]Underwater Bridge Inspection Report Trunk Highway No. 35W over the Mississippi River, Bridge No. 9340, Ayres Associates, Inspection Date December 8, 2004, page 2.

[20]Minnesota Department of Transportation, Construction Plan for Repair of Bridge 9340 Located on T.H. 35W, Federal Project No. I-IG-IR 35W-3 (182) 106, February 17, 1977.

*"Bridge 9340 original construction from 1967 included 1½" of cover over the top reinforcing bars in the deck. By the early 1970's numerous states including Minnesota with harsh environments were having corrosion problems due to the minimal concrete cover over the uncoated reinforcing. As a protective measure Minnesota adopted a policy based on research in the mid 1970's of increasing the cover of top deck rebar to 3" with the addition of a high density concrete overlay. Other states used similar systems or membranes with bituminous overlays. The concrete overlay policy reduced the permeability of harsh chemicals from reacting with the steel and has extended the life of bridge decks at least another 20 years. Overlays were included in new designs and added to many existing bridges."*

### 7.2 1998 CONSTRUCTION PLAN FOR DECK REPAIR OF BRIDGE #9340

The 1998 construction plan[21] for the deck repair of Bridge #9340 involved median replacement, rail retrofit, drainage removal, concrete slab and pier repair, cross girder retrofit, bolt replacement, and installing an anti-icing system.

Mn/DOT explained the reasons for the median replacement and rail retrofit in a January 10, 2008 email to NTSB investigators (See Attachment 20 – Email to the National Transportation Safety Board from the Minnesota Department of Transportation dated January 10, 2008):

*"The original 1967 railings and median guardrail did not meet the requirements of NCHRP 350 for a TL-4 barrier. The F rails on the median and the modifications on the exterior barrier in 1998 do meet safety requirements. The original center median curb and guardrail and the exterior rail were deteriorating from corrosion and traffic impact. They required repair by 1998. When traffic rail modifications are made to existing bridge on the National Highway System (NHS) the FHWA requires we upgrade the railing to meet NCHRP 230/350 standards. Therefore, two new concrete Type F rails with a precast cap were added in the median. The cap between the inside railings stopped the harsh chemicals from leaking onto the underside of deck overhang and the floor trusses. Also a 10" thick inside face was added to the exterior 1 line concrete rails."*

Bridge #9340 was a candidate for an anti-icing system due to the high incidence of winter traffic crashes on the bridge. The bridge was more susceptible to "black ice" and slippery conditions because of moisture from the Mississippi River's St. Anthony Falls, nearby power plants and industrial facilities, and because of the high volume of traffic on the bridge. The formation of "black ice" was due to the combination of extreme cold and heavy vehicle exhaust from congestion on the bridge. In addition to traffic safety, the anti-icing system also

[21]Minnesota Department of Transportation, Construction Plan for Deck Repair Bridge 9340, Federal Project No. IM 035W – 3 (263), January 30, 1998.

contributed to sustainability, because the chemical used was environmentally less toxic and corrosive than sodium chloride, which traditionally had been used.

The bridge anti-icing system worked with a combination of sensors, RWIS weather stations, a computerized control system, and a series of 38 valve units and 76 spray nozzles that apply potassium acetate. The system was automatically activated based on the temperature and atmospheric conditions occurring on and above the bridge deck. A small pump house, north of the river and west of I-35W adjacent to the southbound lanes, housed a 3,100 gallon tank for potassium acetate storage, a 100 gallon tank for water storage, pumps and valves, and software for selecting spray programs. The anti-icing system was typically activated automatically but could be activated manually from the pump house and via remote control.

### 7.3 2007 CONSTRUCTION PLAN FOR REPLACING OVERLAY AND EXPANSION JOINT DEVICES FOR BRIDGE 9340

The 2007 construction plan[22] for the deck repair of Bridge #9340 involved removing the concrete wearing course to 2" deep and adding a new 2" concrete wearing course. The construction plan included removing unsound concrete from the curb and patching with concrete. The construction plan also included reconstructing the expansion joints and removing and replacing the anti-icing system spray disks and sensors in the deck.

## 8. BRIDGE LOAD RATING AND POSTING

### 8.1 BRIDGE LOAD RATING

Bridge load rating in the United States was guided by the AASHTO specifications Manual for Condition Evaluation of Bridges, Second Edition (2000)[23]. The manual indicated the following:

> *"Bridge load rating calculations provide a basis for determining the safe load capacity of a bridge. Load rating requires engineering judgment in determining a rating value that is applicable to maintaining the safe use of the bridge and arriving at posting and permit decisions. Bridge load rating calculations are based on information in the bridge file including the results of a recent inspection. As part of every inspection cycle, bridge load ratings should be reviewed and updated to reflect any relevant changes in condition or dead load noted during the inspection."*

Bridges are rated at two different stress levels, the inventory level and the operating level. The inventory rating level is equivalent to the design level of stress. A bridge subjected to no

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[22] Minnesota Department of Transportation, Construction Plan for Replacing Overlay and Expansion Joint Devices for Bridge 9340, Bridge #27873, Bridge #27874, Bridge #27879, Bridge #27879A, Bridge #27880, Bridge #27880A, Bridge #27887, Bridge #27888, Bridge #27902, and Bridge #27903 Located on T.H. 35W from North of T.H. 94 to Stinson Blvd in the City of Minneapolis, March 15, 2007.

[23] Manual for Condition Evaluation of Bridges, Second Edition 1994, as revised by the 1995, 1996, 1998 and 2000 Interim Revisions and as approved by the AASHTO Subcommittee on Bridges and Structures, American Association of State Highway and Transportation Officials, pages 49 through 51.

more than the inventory stress level can be expected to safely function for an indefinite period of time. The operating rating level is the maximum permissible live load stress level to which a structure may be subjected. Allowing an excessive volume of vehicles to use the bridge at operating level may shorten the life of the bridge.

The Manual for Condition Evaluation of Bridges (MCEB) provides guidance on when the inventory and operating ratings should be recalculated[24]:

> *"When maintenance or improvement work or change in strength of members or dead load has altered the condition or capacity of the structure, the Inventory and Operating ratings should be recalculated."*

The MCEB does not specifically mention that connections (i.e. gusset plates) be evaluated, unless, an inspection reveals deterioration or distress[25]:

> *"At each inspection, any deterioration or distress which has occurred which will materially affect the load-carrying capacity of the structure should be evaluated."*

The MCEB provides a choice of load rating methods. Load ratings can be calculated for the inventory level and operating level using the allowable stress method (primarily used in the 1960's and 1970's) and the load factor method (primarily used in the 1980's and 1990's). A new method for calculating load ratings is available today called the load and resistance factor rating method (LRFR). The LRFR method is described in the AASHTO Guide Specifications for the Strength Evaluation of Existing Steel and Concrete Bridges.

The MCEB gives the following general expression in determining the load rating of a bridge:

$$\text{Load rating factor (RF)} = {(C - A_1 DL)}\big/{(A_2 LL)}$$

Where:

Load rating factor (RF) = the rating for the live-load carrying capacity. The rating factor multiplied by the rating vehicle in tons gives the rating of the structure,

C = the capacity of the member,

Dead Load (DL) and Live Load (LL) = the dead load and live load effect on the member,

$A_1$ = factor for dead loads, and

$A_2$ = factor for live load

[24]ibid, page 9.
[25]ibid, page 73.

The MCEB gives specific values for $A_1$ and $A_2$ depending on which load rating method (allowable stress method or load factor method) and which rating level (inventory level or operating level) are used. The formula mentioned above should also be applied to all of the critical sections of the bridge. The critical section with the lowest load rating factor (RF) is typically taken as the controlling member of the bridge. This is an important index in the jurisdiction's inventory for that particular bridge.

The load rating factor (RF) may be used to determine the rating of the bridge member in tons as follows:

$$RT = (RF)W$$

Where:

RT = bridge member rating in tons,

RF = load rating factor, and

W = weight (in tons) of nominal truck used in determining the live load effect (Minnesota uses the HS20, or 36 tons, nominal truck in determining live load)

Table 11 shows the history of superstructure condition ratings, inventory ratings (IR), and operating ratings (OR) for the I-35W Bridge (Bridge #9340). The data contained in Table 11 was taken from the National Bridge Inventory from 1983 through 2007.

**Table 11 – History of superstructure condition ratings, inventory ratings (IR), and operating ratings (OR) for I-35W Bridge (Bridge #9340)**

| Year | Superstructure Condition Rating | Inventory Rating (IR)[26] | Operating Rating (OR)[27] | Design Load, HS20 (live load category for which the bridge was designed) |
|---|---|---|---|---|
| 1983 | 7 | 26.8 tons (HS14.89) | 53.6 tons (HS29.78) | 36 tons (HS20) |
| 1984 | 7 | 26.8 tons (HS14.89) | 53.6 tons (HS29.78) | 36 tons (HS20) |
| 1985 | 7 | 26.8 tons (HS14.89) | 53.6 tons (HS29.78) | 36 tons (HS20) |
| 1986 | 7 | 26.8 tons (HS14.89) | 53.6 tons (HS29.78) | 36 tons (HS20) |
| 1987 | 7 | 26.8 tons (HS14.89) | 53.6 tons (HS29.78) | 36 tons (HS20) |
| 1988 | 7 | 26.8 tons (HS14.89) | 53.6 tons (HS29.78) | 36 tons (HS20) |
| 1989 | 8 | 26.8 tons (HS14.89) | 53.6 tons (HS29.78) | 36 tons (HS20) |
| 1990 | 7 | 26.8 tons (HS14.89) | 53.6 tons (HS29.78) | 36 tons (HS20) |
| 1991 | 4 | 26.8 tons (HS14.89) | 53.6 tons (HS29.78) | 36 tons (HS20) |
| 1992 | 4 | 26.8 tons (HS14.89) | 53.6 tons (HS29.78) | 36 tons (HS20) |
| 1993 | 4 | 26.8 tons (HS14.89) | 53.6 tons (HS29.78) | 36 tons (HS20) |
| 1994 | 4 | 26.8 tons (HS14.89) | 53.6 tons (HS29.78) | 36 tons (HS20) |
| 1995 | 4 | 26.8 tons (HS14.89) | 53.6 tons (HS29.78) | 36 tons (HS20) |
| 1996 | 4 | 35.7 tons (HS19.83) | 58.5 tons (HS32.5) | 36 tons (HS20) |
| 1997 | 4 | 35.7 tons (HS19.83) | 58.5 tons (HS32.5) | 36 tons (HS20) |
| 1998 | 4 | 35.7 tons (HS19.83) | 58.5 tons (HS32.5) | 36 tons (HS20) |
| 1999 | N/A[28] | 35.7 tons (HS19.83) | 59.0 tons (HS32.78) | 36 tons (HS20) |
| 2000 | 4 | 35.7 tons (HS19.83) | 59.0 tons (HS32.78) | 36 tons (HS20) |
| 2001 | 4 | 35.7 tons (HS19.83) | 59.0 tons (HS32.78) | 36 tons (HS20) |
| 2002 | 4 | 36.0 tons (HS20) | 59.4 tons (HS33) | 36 tons (HS20) |
| 2003 | 4 | 36.0 tons (HS20) | 59.4 tons (HS33) | 36 tons (HS20) |
| 2004 | 4 | 36.0 tons (HS20) | 59.4 tons (HS33) | 36 tons (HS20) |
| 2005 | 4 | 36.0 tons (HS20) | 59.4 tons (HS33) | 36 tons (HS20) |
| 2006 | 4 | 36.0 tons (HS20) | 59.4 tons (HS33) | 36 tons (HS20) |
| 2007 | 4 | 36.0 tons (HS20) | 59.4 tons (HS33) | 36 tons (HS20) |

[26]The inventory ratings shown on the National Bridge Inventory are reported in metric tons and have been converted to U.S. tons.
[27]ibid.
[28]N/A means no value submitted by Mn/DOT.

## 8.2 BRIDGE POSTING

The posting of a maximum weight limit sign is required on a bridge if the maximum vehicle weight that state regulation allows (in Minnesota, the maximum gross vehicle weight is 40 tons or 80,000 pounds[29]) on that highway exceeds the bridge's maximum weight limit determined by the operating rating (OR).

According to the National Bridge Inspection Standards (NBIS)[30]:

> *"If it is determined under this rating procedure that the maximum legal load under State law exceeds the load permitted under the Operating Rating, the bridge must be posted in conformity with the AASHTO Manual or in accordance with State law (23 CFR Part 650, Subpart C)."*

Mn/DOT's Bridge Rating Manual stated that when the load rating factor (RF) is less than 1, the bridge will be posted[31]:

> *"Legal Load Rating: (Sometimes called Posting Rating.) The live load is one or more of the "legal trucks". If the RF is less than 1.00 (or another specified amount), the bridge will be posted."*

From the data in Table 12, the operating rating (OR) for the I-35W Bridge (Bridge #9340) varied from 53.6 tons to 59.4 tons from 1983 to 2007, respectively. Taking the formula mentioned above, RT = (RF)W, and inserting the data from Table 12, revealed the load rating factor (RF) was greater than 1.

$$RF = RT / W = (53.6 \text{ tons}) / (36 \text{ tons}) = 1.49$$

The posting of a maximum weight limit sign on the I-35W Bridge (Bridge #9340) was not required because the load rating factor (RF) for all legal trucks was never below 1.

## 8.3 VEHICLE PERMITTING

In the United States, the current weight limit for the Interstate system is 80,000 pounds gross vehicle weight (GVW) and 20,000 pounds for an axle. Minnesota's requirement is the same as stated in Minnesota Statutes, Chapter 169[32]:

***"169.824 GROSS WEIGHT SCHEDULE***

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[29] 1 U.S. ton is equal to 2,000 pounds.

[30] Legal Truck Loads and AASHTO Legal Loads for Posting, National Cooperative Highway Research Program (NCHRP) Report 575, Transportation Research Board, Washington, D.C., 2007, page 15.

[31] LRFD Bridge Design Manual, Minnesota Department of Transportation Bridge Office, Chapter 15 Bridge Rating, June 2007 Draft, page 15-4.

[32] Minnesota Statutes 2006, Chapter 169, Traffic Regulations: Size, Weight, and Load Restrictions and Permits.

*Gross vehicle weight of all axles. (a) Notwithstanding the provisions of section 169.85, the gross vehicle weight of all axles of a vehicle or combination of vehicles shall not exceed:*

*(1) 80,000 pounds for any vehicle or combination of vehicles on all state trunk highways as defined in section 160.02, subdivision 29, and for all routes designated under section 169.832, subdivision 11…*

***169.823 TIRE WEIGHT LIMITS***

*(2) where the gross weight on any single axle exceeds 18,000 pounds, except that on designated local routes and state trunk highways the gross weight on any single axle shall not exceed 20,000 pounds…*

***169.826 GROSS WEIGHT SEASONAL INCREASES***

*Subd. 3.* **Excess weight permit.** *When the ten percent increase is in effect, a permit is required for a motor vehicle, trailer, or semitrailer combination that has a gross weight in excess of 80,000 pounds, an axle group weight in excess of that prescribed in section 169.824, or a single axle weight in excess of 20,000 pounds and which travels on interstate routes."*

The federal bridge formula calculates the maximum load in pounds carried on any group of 2 or more consecutive axles. The federal bridge formula was established to provide a simple means of determining whether or not a vehicle could be allowed to travel without a permit.

$$\mathrm{W} = 500\left( {}^{LN}\!\big/_{(N-1)} + 12N + 36 \right)$$

Where:

W = the allowable gross weight in pounds on any group of two or more consecutive axles,

L = the distance in feet between the extreme of any group of two or more consecutive axles, and

N = the number of axles included in the group under consideration

Another alternative way of presenting the bridge formula is a table as shown in Table 12. No vehicle or combination of vehicles can be operated in Minnesota without a permit, where the total weight on any group of two or more consecutive axles of any vehicle or combination of vehicles exceeds that given in Table 12. Table 12 is taken from the Minnesota Statutes, Chapter 169.

**Table 12 – Axle weight limits for State of Minnesota**

| Distances in feet between centers of foremost and rearmost axles | Maximum gross weight in pounds on a group of: | | | | | |
|---|---|---|---|---|---|---|
| | 2 consecutive axles of a 2 axle vehicle | 3 consecutive axles of a 3 axle vehicle | 4 consecutive axles of a 4 axle vehicle | 5 consecutive axles of a 5 axle vehicle | 6 consecutive axles of a 6 axle vehicle | 7 consecutive axles of a 7 axle vehicle |
| 4 | 34,000 | - | - | - | - | - |
| 5 | 34,000 | - | - | - | - | - |
| 6 | 34,000 | - | - | - | - | - |
| 7 | 34,000 | 37,000 | - | - | - | - |
| 8 | 34,000 | 38,500 | - | - | - | - |
| 8 plus | 34,000 | 42,000 | - | - | - | - |
| | (38,000)[33] | - | - | - | - | - |
| 9 | 35,000 | 43,000 | - | - | - | - |
| | (39,000) | - | - | - | - | - |
| 10 | 36,000 | 43,500 | 49,000 | - | - | - |
| | (40,000) | - | - | - | - | - |
| 11 | 36,000 | 44,500 | 49,500 | - | - | - |
| 12 | - | 45,000 | 50,000 | - | - | - |
| 13 | - | 46,000 | 51,000 | - | - | - |
| 14 | - | 46,500 | 51,500 | 57,000 | - | - |
| 15 | - | 47,500 | 52,000 | 57,500 | - | - |
| 16 | - | 48,000 | 53,000 | 58,000 | - | - |
| 17 | - | 49,000 | 53,500 | 59,000 | - | - |
| 18 | - | 49,500 | 54,000 | 59,500 | - | - |
| 19 | - | 50,500 | 55,000 | 60,000 | - | - |
| 20 | - | 51,000 | 55,500 | 60,500 | 66,000 | 72,000 |
| 21 | - | 52,000 | 56,000 | 61,500 | 67,000 | 72,500 |
| 22 | - | 52,500 | 57,000 | 62,000 | 67,500 | 73,000 |
| 23 | - | 53,500 | 57,500 | 62,500 | 68,000 | 73,500 |
| 24 | - | 54,000 | 58,000 | 63,000 | 68,500 | 74,000 |
| 25 | - | (55,000) | 59,000 | 64,000 | 69,000 | 75,000 |
| 26 | - | (55,500) | 59,500 | 64,500 | 70,000 | 75,500 |
| 27 | - | (56,500) | 60,000 | 65,000 | 70,500 | 76,000 |
| 28 | - | (57,000) | 61,000 | 65,500 | 71,000 | 76,500 |
| 29 | - | (58,000) | 61,500 | 66,500 | 71,500 | 77,000 |
| 30 | - | (58,500) | 62,000 | 67,000 | 72,000 | 77,500 |
| 31 | - | (59,500) | 63,000 | 67,500 | 73,000 | 78,500 |

[33]The gross weights shown in parentheses are permitted only on state trunk highways and routes designated under section 169.832, subdivision 11.

| | | | | | | |
|---|---|---|---|---|---|---|
| 32 | - | (60,000) | 63,500 | 68,000 | 73,500 | 79,000 |
| 33 | - | - | 64,000 | 69,000 | 74,000 | 79,500 |
| 34 | - | - | 65,000 | 69,500 | 74,500 | 80,000 |
| 35 | - | - | 65,500 | 70,000 | 75,000 | - |
| 36 | - | - | 66,000 | 70,500 | 76,000 | - |
| 37 | - | - | 67,000 | 71,500 | 76,500 | - |
| 38 | - | - | 67,500 | 72,000 | 77,000 | - |
| 39 | - | - | 68,000 | 72,500 | 77,500 | - |
| 40 | - | - | 69,000 | 73,000 | 78,000 | - |
| 41 | - | - | 69,500 | (74,000) | 79,000 | - |
| 42 | - | - | 70,000 | (74,500) | 79,500 | - |
| 43 | - | - | 71,000 | (75,000) | 80,000 | - |
| 44 | - | - | 71,500 | (75,500) | - | - |
| 45 | - | - | 72,000 | (76,500) | - | - |
| 46 | - | - | 72,500 | (77,000) | - | - |
| 47 | - | - | (73,500) | (77,500) | - | - |
| 48 | - | - | (74,000) | (78,000) | - | - |
| 49 | - | - | (74,500) | (79,000) | - | - |
| 50 | - | - | (75,500) | (79,500) | - | - |
| 51 | - | - | (76,000) | (80,000) | - | - |

The Minnesota Statutes, Chapter 169 indicated the conditions for issuing a special permit:

*"**169.86 SPECIAL PERMIT TO EXCEED HEIGHT, WIDTH, OR LOAD; FEES***

*Subdivision 1.* ***Permit authorities; restrictions.*** *(a) The commissioner, with respect to highways under the commissioner's jurisdiction, and local authorities, with respect to highways under their jurisdiction, may, in their discretion, upon application in writing and good cause being shown therefore, issue a special permit, in writing, authorizing the applicant to move a vehicle or combination of vehicles of a size or weight of vehicle or load exceeding the maximum specified in this chapter, or otherwise not in conformity with the provisions of this chapter, upon any highway under the jurisdiction of the party granting such permit and for the maintenance of which such party is responsible…*

*Subd. 2.* ***Required information.*** *The application for a permit shall specifically describe in writing the vehicle or vehicles and loads to be moved and the particular highways and period of time for which a permit is requested…*

*Subd. 4.* ***Display and inspection of permit.*** *Every such permit shall be carried in the vehicle or combination of vehicles to which it refers and shall be open to inspection by any police officer or authorized agent of any authority granting such permit, and no person shall violate any of the terms or conditions of such special permit."*

The Minnesota Statutes, Chapter 169 provided information on an annual permit for overweight, or oversize and overweight, construction equipment, machinery, and supplies. The fees for the permit are shown in Table 13.

**Table 13 – Annual permit fees for overweight vehicles**

| Gross Weight (pounds) of Vehicle | Annual Permit Fee |
|---|---|
| 90,000 or less | $200 |
| 90,001 – 100,000 | $300 |
| 100,001 – 110,000 | $400 |
| 110,001 – 120,000 | $500 |
| 120,001 – 130,000 | $600 |
| 130,001 – 140,000 | $700 |
| 140,001 – 145,000 | $800 |

## 8.4 TYPES OF PERMITS ISSUED BY Mn/DOT FOR OVERWEIGHT TRUCKS

Mn/DOT provided the types of permits issued for overweight trucks in a December 19, 2007 letter to NTSB investigators (See Attachment 6 – Letter to the National Transportation Safety Board from the Minnesota Department of Transportation dated December 19, 2007). The following excerpts are taken from the letter:

> *"All trucks over legal loads defined by Formula B* (or commonly referred to as the federal bridge formula) *are required to get a permit. Minnesota permits are issued by the Mn/DOT Office of Freight and Commercial Vehicle Operations (OFCVO) for state owned bridges. Some responsibilities of the OFCVO are to issue permits, collect fees, record information, and communicate with the Bridge Office for special loads. The OFCVO uses a computer program called "Routebuilder NT" to process all permits.*
>
> *There are basically 3 kinds of permits issued for overweight trucks on state owned highways in Minnesota. The first is a self routing divisible annual permit for special commodities for loads up to 98 kips. The second is an annual permit for non divisible loads that do not exceed 145 kips. The last is a single trip permit for non divisible loads above the legal load.*
>
> *The divisible annual permit is for certain commodities that have legislature approval to go above legal loads. Garbage haulers, raw forest products and some agricultural harvest products are examples of divisible annual permits. The maximum weights differ by the haul product but the maximum is 98k on 6 axles. The commodity haulers are allowed to travel on all non interstate bridges unless it is posted with a permit restricted sign...*
>
> *...The annual non divisible permit allows trucks an unlimited number of trips. Annual non divisible permits are allowed up to 92k on 5 axles and 145k on 8*

*axles. The trucks either call in for their route or use our website that provides the weight and bridge restrictions. If the truck is below 84k on 5 axles or 112k on 8 axles the truck may self route from bridge information given by Mn/DOT.*

*Lastly there were approximately 28,000 single trip permits issued last year for overweight trucks. There is no maximum weight for single trip permits other than bridge capacity. All single trip permits have a defined route determined by OFCVO along with any restrictions…*

*…Routebuilder is able to process almost all the annual permits and the vast majority of single trip permits for overweight automatically. Based on axle weights and spacing Routebuilder attempts to classify the truck into a standard permit truck A, B, C or over C. The A truck is 104k at 46', the B is 136k at 49' and the C is combination of three trucks 159k at 57', 207k at 93' and 259k at 117'. All state owned bridges have their capacities related to the 3 standard permit trucks. If the truck is classified as an A, B or C the permit is compared to the predetermined standard truck capacity for each bridge it crosses. If the weight is over a C or Routebuilder and OFCVO permit technician can't accurately classify the truck due to concentrated axle groups, the information is sent to the Bridge Office for review. All trucks over legal weights are reviewed either directly by the Bridge Office or indirectly by the criteria the Bridge Office set for standard permit trucks used by Routebuilder.*

*Permit reviews are done by experienced bridge engineers in the Load Rating Unit at the Bridge Office. Permit reviews are typically processed within hours but could take several days or weeks depending on the complexity of permit. There are times when especially heavy permits take extensive coordination between the Bridge Office, OFCVO and the hauler to find a safe practical route.*

*For bridges that have rating factors above 1.0 for a permit there are no restrictions placed on the driver of the truck. If the rating factor is less than one, there are additional methods to decrease the trucks effect on the bridge. By occupying 2 lanes on a bridge the truck eliminates the possibility of a heavy adjacent truck. Also by limiting the speed, the truck reduces the dynamic forces of impact. If the rating factor is still below 1.0 with restrictions like eliminating an adjacent truck and/or reducing the speed, the permit is denied for that bridge and a new route must be chosen.*

*The Bridge Office maintains a database of all state owned bridges which includes postings and the standard permit truck restrictions if any. All state owned bridges have been analyzed for the standard permit trucks or posting trucks and these are updated as conditions change from deterioration to increased dead load. The Rating Unit is in the process of switching to a software program called Virtis from BARS which is being phased out. All the state owned bridges are being input in and analyzed by Virtis except the curved steel girders, concrete boxes, arches, tunnels, post tensioned boxes and trusses. Curved steel, concrete boxes* *and rigid frames are load rated either with BARS, another software package or by hand calculations."*

8.5 Mn/DOT DRAFT BRIDGE RATING MANUAL, JUNE 2007

Mn/DOT provided a copy of the LRFD Bridge Design Manual, Chapter 15 Bridge Rating, dated June 2007[34] Draft (See Attachment 21 – Mn/DOT LRFD Bridge Design Manual, Chapter 15 Bridge Rating, dated June 2007 Draft). The manual indicated the following:

*"<u>INTRODUCTION</u>*
*Bridge ratings are administered and performed by the Bridge Rating Unit of the Mn/DOT Bridge Office. Bridge ratings may also be performed by other qualified engineers.*

*All bridges in Minnesota open to the public, with spans of 10 feet and more are rated. This includes all county and local bridges. However, bridges that carry pedestrians, recreational traffic, or railroad trains need not be rated. Culverts, with spans of 10 feet or more, are also rated, but by a different method. See the Culvert section of this chapter for more information.*

*Rating results are kept on file, and key information is entered in the Pontis database. From there annual reports are prepared and sent to the FHWA.*

*Bridge Ratings are calculated in accordance with the AASHTO Manual for Condition Evaluation of Bridges (MCE). This manual refers the user to the AASHTO Standard Specifications for Highway Bridges for much additional needed information...*

*<u>GLOSSARY</u>*
*Design Load Rating: The AASHTO HS truck and lane loads are used for the live load. The final rating is usually expressed relative to HS20. This is usually calculated at both the inventory and operating levels.*

*Legal Load Rating: (Sometimes called Posting Rating.) The live load is one or more of the "legal trucks". If the RF is less than 1.00 (or another specified amount), the bridge will be posted.*

*RF: Rating Factor: The result of calculating the rating equation, MCE 6-1a. Generally RF≥1.0 indicates that the member or bridge has sufficient capacity for the equated live load and is acceptable; and RF<1.0 indicates overstress and requires further action. The RF may be converted to a weight by applying the equation, MCE 6-1b. An RF is always associated with a particular live load...*

*<u>GENERAL</u>*

[34] LRFD Bridge Design Manual, Minnesota Department of Transportation Bridge Office, Chapter 15 Bridge Rating, June 2007 Draft, pages 15-2 through 15-10 including attachments.

*Bridges are rated at two different stress levels, Inventory level and Operating level. The Operating level is used for load posting and for evaluation of overweight permits.*

*In almost all cases only the primary load carrying members of the superstructure are rated. Decks or piers may have to be investigated in unusual circumstances such as severe deterioration. Unusually heavy permit loads may also require investigation of the deck and piers.*

*When rating a bridge, the final overall bridge rating should be the rating of the weakest point of the weakest member within the bridge. This is recorded on the cover sheet of the rating form.*

*The weakest link may change with different rating vehicles. This is because rating vehicles of different weights, axle spacings, and/or lengths have different effects on different members and spans. The identification of the controlling member, location, and limit state for each rated vehicle is given on page two (or three) of the rating forms.*

*Generally ratings are calculated for shear and for bending moment, and at the tenth points of each span and other critical points as needed. Other force effects that are sometimes checked are axial and torsion...*

*LOADS*

*For steel bridges, account for the extra dead loads such as welds, splices, bolts, connection plates, etc. This generally ranges from 2% to 5% of the main member weight.*

*Design ratings are calculated and reported in terms of HS20. Thus with the HS20 truck as the live load in the denominator of the rating equation and if the resulting rating factor is 1.17, the rating would be recorded as HS23.4…*

*RATING NEW BRIDGES*

*New bridges are to be rated anytime after the plan is completed and before the bridge is opened to traffic. The results are then turned in to the Bridge Management Unit for entering in Pontis.*

*For Mn/DOT bridges, the records remain inactive until Bridge Management is informed that the bridge has been opened to traffic.*

*If any changes are made to the bridge during construction that would affect the rating, these changes should be reported to the Bridge Ratings Unit (or the person who did the original rating), and also be recorded on the as built plans. This includes strand pattern changes for prestressed beams. The bridge rating is then recalculated.*

*RERATING EXISTING BRIDGES*

*A new bridge rating should be calculated whenever a change occurs that would affect the rating. The most commonly encountered types of changes are:*

- *A modification that changes the dead load on the bridge (For example: a deck overlay)*
- *Damage that alters the structural capacity of the bridge (For example: being hit by an oversize load)*
- *Deterioration that alters the structural capacity of the bridge (For example: rust, corrosion or rot) Scheduled inspections are usually the source of this information.*
- *Settlement or movement of a pier or abutment.*
- *Repairs or remodeling.*
- *A change in the AASHTO Rating Specification.*
- *An upgrading of the rating software.*
- *A change in laws regulating truck weights.*

*A new rating should be completed, signed, dated, and filed, as outlined in the Forms and Documentation Section of this chapter. This most recent rating then supercedes any and all preceding ratings.*

*OVERWEIGHT PERMITS*

*Maximum vehicle weights are defined in Minnesota Statutes. Under certain conditions, trucks may obtain permits to travel at greater weights.*

*Overweight and overdimension permits are issued by the Office of Freight and Commercial Vehicle Operations.*

*The OFCVO issues annual permits for trucks weighing up to a maximum of 145,000 pounds. A holder of an annual permit may make an unlimited number of trips during the year of the permit. The trucker may make his own judgment of which weight class (A, B, or C) his truck fits, or he may ask the permit office to determine the weight class. The permit office sometimes forwards these to the bridge office. These are commonly called "general checks."*

*If the initial RF for a permit truck is less than 1.0, the truck may still be allowed to cross the bridge under a restriction. Overweight Permit Restrictions are shown in Figure 15-R-xx.*

*A truck traveling under an overweight permit may not cross a load posted bridge.*

*Standard Permit Vehicle A: 104 kip, 46 feet length*
*Standard Permit Vehicle B: 136 kip, 49 feet length*
*Standard Permit Vehicle C: 159 kip, 57 feet length*

*<u>OVERWEIGHT PERMIT RESTRICTIONS FOR BRIDGES</u>*
*Restriction Code 1: No restrictions to drive over bridge.*

*Restriction Code 2: Drive on the centerline between two lanes, in a manner that prevents any other vehicle from occupying a part of either lane on either side of the permit vehicle. Drive in the center of a single lane bridge.*

*Restriction Code 3: Drive at a speed of 10 miles per hour or less.*

*Restriction Code 4: Combine both restrictions 2 and 3 above; vehicle shall straddle two lanes at maximum speed of 10 miles per hour.*

*Restriction Code 5: More specific instructions must be attached.*

*Restriction Code 6: More specific instructions must be attached.*

*Restriction Code 7: More specific instructions must be attached.*

*Restriction Code X: This overweight permit vehicle is **NOT ALLOWED** on this bridge."*

## 8.6 DECEMBER 1995 BRIDGE RATING AND LOAD POSTING REPORT FOR I-35W BRIDGE (BRIDGE #9340)

The Bridge Rating and Load Posting Report, dated December 14, 1995 and certified by a registered professional engineer in the State of Minnesota, indicated the inventory rating was HS20 (or 36 tons) and the operating rating was HS33 (or 59.4 tons) (See Attachment 22 – Mn/DOT Bridge Rating and Load Posting Report dated December 14, 1995). The posting of the bridge was not required. Table 14 shows the critical sections of the I-35W Bridge with the inventory rating, operating rating, and the date shown on the bridge rating sheet. The controlling section of the bridge was identified as S01, the south approach, consisting of 5 spans (spans 1 through 5).

**Table 14 – Critical sections of I-35W Bridge (Bridge #9340) with the inventory rating, operating rating, and the date shown on the bridge rating sheet**

| Critical Section | Description | Inventory Rating | Operating Rating | Date shown on the bridge rating sheet |
|---|---|---|---|---|
| S01 | South approach 5 spans (1 through 5) | HS20 | HS33 | 12-14-95 |
| S02 | North approach 3 spans (9 through 11) | HS27 | HS45.5 | 12-14-95 |
| S03 | Continuous voided slab 3 spans (12 through 14) | HS20.2 | HS33.6 | 12-14-95 |
| S04, S05[35] | Truss Stringers 3 spans (6 through 8) | HS22.8 | HS38.1 | 8-18-97 |

Mn/DOT sent a letter to the NTSB dated September 21, 2007 identifying the specific members that governed the weakest point of the bridge (See Attachment 9 – Letter to the National Transportation Safety Board from the Minnesota Department of Transportation dated September 21, 2007):

> *"This controlling rating is from the SB roadway, beam line G13, at midspan in the fourth of the five continuous spans (107.25 ft). The limit state is tension stress in the bottom flange due to bending moment. The rating program used here, BARS, is a line analysis program. Member G13 would have been selected as a good representative because of its longer length in span one and it is the first interior beam."*

The method of load rating used for the December 1995 Bridge Rating and Load Posting Report was the load factor method. The design load (or live load category for which the bridge was designed) was HS20 (or 36 tons). The permit codes shown on the December 1995 Bridge Rating and Load Posting Report were the following:

- Standard Permit Vehicle A (104 kip, 46 feet length): 1 (no restrictions to drive over the bridge),
- Standard Permit Vehicle B (136 kip, 49 feet length): 1 (no restrictions to drive over the bridge), and

[35]The inventory rating and operating rating for critical sections S04 and S05 are the same since they both consist of truss stringers (S04 represents 4 span continuous at 38 feet length and S05 represents 6 span continuous at 38 feet length). The inventory rating and operating rating for S04 is given.

- Standard Permit Vehicle C (159 kip, 57 feet length): 1 (no restrictions to drive over the bridge)

The data used for the basis of the December 1995 Bridge Rating and Load Posting Report was a computer program called BARS. Mn/DOT acknowledged in a September 21, 2007 letter to NTSB investigators that the BARS program is an old program which is being phased out (See Attachment 9 – Letter to the National Transportation Safety Board from the Minnesota Department of Transportation dated September 21, 2007):

> *"BARS is an old program which is being phased out."*

Mn/DOT acknowledged in a December 19, 2007 letter to NTSB investigators that the BARS program is being replaced with Virtis (a specialized bridge load rating program developed by FHWA and AASHTO) (See Attachment 6 – Letter to the National Transportation Safety Board from the Minnesota Department of Transportation dated December 19, 2007):

> *"The Rating Unit is in the process of switching to a software program called Virtis from BARS which is being phased out. All the state owned bridges are being input in and analyzed by Virtis except the curved steel girders, concrete boxes, arches, tunnels, post tensioned boxes and trusses. Curved steel, concrete boxes and rigid frames are load rated either with BARS, another software package or by hand calculations."*

Mn/DOT acknowledged in a January 8, 2008 letter to NTSB investigators that the BARS and Virtis bridge load rating programs do not have the capability of analyzing connections (i.e. gusset plates) (See Attachment 23 – Letter to the National Transportation Safety Board from the Minnesota Department of Transportation dated January 8, 2008):

> *"Neither BARS nor Virtis rating software include an analysis of connections."*

The Mn/DOT bridge rating engineer that performed the rating and load posting analysis used a December 11, 1995 BARS computer printout (See Attachment 24 – Mn/DOT BARS load rating computer printout dated December 11, 1995) that calculated the inventory rating and operating rating for critical sections S01, S02, and S03. The dead load used in the December 11, 1995 BARS computer printout was 358 pounds per linear foot for critical sections S01 and S02, and 47 pounds per linear foot for critical section S03.

The same Mn/DOT bridge rating engineer calculated a new dead load on the bridge on August 18, 1997. The new dead load was to reflect the weight of a new solid extension rail and double J-rail to be constructed in a January 1998 letting contract. The Mn/DOT bridge rating engineer used the new dead load to re-calculate the inventory ratings and operating ratings as shown in an August 18, 1997 BARS computer printout (See Attachment 25 – Mn/DOT BARS load rating computer printout dated August 18, 1997). The calculations performed by the Mn/DOT bridge rating engineer show the new dead load as 487 pounds per linear foot for critical sections S01, S02, S04, and S05. The 47 pounds per linear foot for critical section S03 remained the same.

Table 15 shows a comparison of the inventory ratings and operating ratings as shown on the December 1995 Bridge Rating and Load Posting Report, and what was actually calculated in the December 11, 1995 BARS and August 18, 1997 BARS computer printouts. The inventory rating and operating rating went down for critical sections S01 and S02 as a result of the new dead load, however, it was not reported on the bridge rating and load posting report. Instead, the inventory rating and operating rating for critical sections S01 and S02 were taken from the December 11, 1995 BARS computer printout. The inventory rating and operating rating remained the same for critical section S03 because no change occurred to the dead load. Had the inventory rating and operating rating for critical sections S01 and S02 been used from the August 18, 1997 BARS computer printout, the posting of the bridge would still not be required.

**Table 15 – Comparison of the inventory ratings and operating ratings as shown on the December 1995 Bridge Rating and Load Posting Report, and what was actually calculated in the December 11, 1995 BARS and August 18, 1997 BARS computer printouts**

| Critical Section | December 1995 Bridge Rating and Load Posting Report; Inventory Rating (IR) Operating Rating (OR) | December 11, 1995 BARS computer printout | | August 18, 1997 BARS computer printout | |
|---|---|---|---|---|---|
| | | Ratings | Dead Load (lb/ft) | Ratings | Dead Load (lb/ft) |
| S01 | HS20 (IR)<br>HS33 (OR) | HS19.77 (IR)<br>HS32.95 (OR) | 358 | HS18.93 (IR)<br>HS31.55 (OR) | 487 |
| S02 | HS27 (IR)<br>HS45.5 (OR) | HS27.32 (IR)<br>HS45.53 (OR) | 358 | HS26.69 (IR)<br>HS44.48 (OR) | 487 |
| S03 | HS 20.2 (IR)<br>HS 33.6 (OR) | HS20.17 (IR)<br>HS33.61 (OR) | 47 | HS20.17 (IR)<br>HS33.61 (OR) | 47 |
| S04, S05[36] | HS 22.8 (IR)<br>HS38.1 (OR) | Not Calculated | | HS22.87 (IR)<br>HS38.12 (OR) | 487 |

Mn/DOT acknowledged in a November 2, 2007 email to NTSB investigators the reason why the new bridge rating was not officially documented (See Attachment 26 – Email to the National Transportation Safety Board from the Minnesota Department of Transportation dated November 2, 2007):

> *"It appears that a new rating was computed with BARS in August 1997, before the construction work was done on the bridge. The construction contract was bid on March 27, 1998, with work performed during the 1998 construction season. Apparently the follow up to officially document and record the rating did not occur after construction was completed."*

[36]The inventory rating and operating rating for critical sections S04 and S05 are the same since they both consist of truss stringers (S04 represents 4 span continuous at 38 feet length and S05 represents 6 span continuous at 38 feet length). The inventory rating and operating rating for S04 is given.

The email indicated that the former Mn/DOT bridge rating engineer retired on July 8, 1998.

### 8.7 SEPTEMBER 1979 BRIDGE RATING AND LOAD POSTING REPORT FOR I-35W BRIDGE (BRIDGE #9340)

The Bridge Rating and Load Posting Report, dated September 17, 1979 and certified by a registered professional engineer in the State of Minnesota, indicated the inventory rating was HS15.9 (or 28.6 tons) and the operating rating was HS30.6 (or 55.1 tons) (See Attachment 27 – Mn/DOT Bridge Rating and Load Posting Report dated September 17, 1979). The posting of the bridge was not required.

Mn/DOT acknowledged in an October 10, 2007 email to NTSB investigators that the September 1979 rating used the allowable stress method and the December 1995 rating used the load factor method (See Attachment 28 – Email to the National Transportation Safety Board from the Minnesota Department of Transportation dated October 10, 2007). Mn/DOT indicated in the email that the reason the September 1979 rating is lower than the December 1995 rating is because the load factor method typically yields higher rating numbers. Mn/DOT indicated in the email that they did not know from the September 1979 rating sheet which portion of the bridge controlled the rating. Mn/DOT also indicated in the email that a BARS computer printout was not attached to the September 1979 rating sheet in the bridge management file.

> *"…This was in the Bridge Management file for Bridge 9340, there was no computer output attached…*
>
> *…We also do not know from this sheet what portion of the bridge controlled the rating.*
>
> *In 1979 this rating would have been done using the Allowable Stress Method for ratings. The rating done in the 1990's would have used the Load Factor Design Method, which typically yields higher rating numbers. That is the reason the 1979 rating is lower than 1995."*

Mn/DOT acknowledged in a November 2, 2007 email to NTSB investigators that they do not have a policy on retention of bridge rating reports (See Attachment 26 – Email to the National Transportation Safety Board from the Minnesota Department of Transportation dated November 2, 2007):

> *"There is no policy in the Bridge Ratings Unit (where BARS reports and other supporting calculations are filed) to retain old ratings after a new one is computed. In our Bridge Management section, they do leave the old rating in the files when a new rating is given to them. This usually involves one or two pages. The 1979 rating is in the Bridge Management file."*

## 8.8 FHWA TURNER-FAIRBANK HIGHWAY RESEARCH CENTER REPORT ASSESSMENT OF THE LOAD RATING RECORDS FOR MINNESOTA BRIDGE NO. 9340 (I-35W OVER THE MISSISSIPPI RIVER)

The FHWA Turner-Fairbank Highway Research Center prepared a report entitled Assessment of the Load Rating Records for Minnesota Bridge No. 9340 (I-35W over the Mississippi River) dated June 30, 2008 (See Attachment 29 - FHWA Turner-Fairbank Highway Research Center Report Assessment of the Load Rating Records for Minnesota Bridge No. 9340 (I-35W over the Mississippi River) dated June 30, 2008). The findings of the report include the following:

*"**FINDINGS***

- *No information on load rating of the truss portion of the structure was found in the documentation supplied for any of the load ratings conducted. The load rating file should include an analysis supporting the current load rating for the entire bridge, including the deck truss, either from an initial analysis concluding that the truss was not critical to future ratings or from a rating prompted by a change in conditions or deterioration. The influence lines that were included for all truss members in the original design documents may have been used initially to verify that the rating was controlled by the deck stringer system; however, there is nothing within the documentation provided to support this assumption. A re-rating was performed on the approach spans in 1995, and again in 1997; however, no information was included pertaining to a re-rating of the truss structure. The re-rating in 1997 was warranted due to an increase in dead load resulting from the change in bridge barrier type.*

- *The only document retained from the 1979 load rating was the Rating and Load Posting Report Sheet. The Report Sheet indicates a reduction in capacity of approximately 20% from design values. While no supporting documentation was reviewed, it can be inferred from calculation and other information that the reduction in rating was due to the added weight of the 1977 bridge overlay.*

- *The retained records for the load ratings conducted in 1995 and in 1997 on the approach spans are incomplete. These ratings were conducted on the interior G13 girder. It is unclear if this is the controlling girder line and unknown whether the engineer considered other girder lines.*

- *The dead load calculations retained from the 1997 rating contained minor errors. The height for the exterior barrier installed in the 1990's should have been 2'-8" instead of 2'-0" and the width should have been 10 inches instead of 9 inches. Also, diaphragms, lamp posts, and existing metal posts were not included in the dead load calculations. Although the overall significance of these items may be minimal, a load rating analysis should accurately account for all existing dead load conditions applied to the structure and include a narrative describing what assumptions were made in determining the applied dead load.*

- *The inspection reports indicate that several of the exterior approach span girders, primary truss and floor truss members, and primary truss connections exhibited some section loss due to corrosion that was not addressed in either a narrative summary or in re-rating calculations. A load rating analysis should take into consideration the loss of capacity resulting from deterioration of all load carrying structural elements or the file should include a discussion detailing the reasons why the deterioration was considered negligible.*

- *The most recent "Load Rating Summary" sheet is not correct. In the 1997 calculations provided, the girders in the south approach governed with an HS18.9 Inventory Rating and a HS31.5 Operating Rating (ratings in this document are reported in Customary U.S. Units and are based on an HS20 live load rating vehicle and the load factor rating method). The controlling ratings shown on the most recent Load Rating Summary sheet are HS20.0 for Inventory and HS33.0 for Operating. This error seems to have resulted from not appropriately updating the information included on the 1995 load rating summary sheets. It appears that the stringer calculations conducted in 1997 were simply appended to the 1995 Load Rating Summary sheet and no new summary sheet was generated despite the increase in bridge rail dead load which resulted in about a 5% reduction in load carrying capacity.*

- *According to the 1979 Load Rating Summary sheet, the Inventory Rating was HS15.9 and the Operating Rating was HS30.6. According to the 1995 Load Rating Summary Sheet, the Inventory Rating was HS20 and the Operating Rating was HS33. While it is most likely that the variation in ratings between 1979 and 1995 was the result of transitioning from the ASR method to the LFR method that took place during that time period, no documentation was found that provided that explanation.*

*It is important to note that despite the omissions and inconsistencies of the documentation, the results for all of the ratings conducted indicate that the I-35W Bridge was capable of safely carrying the live load for which it was designed."*

## 9. SUFFICIENCY RATING FORMULA

The sufficiency rating formula can be found in FHWA's "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges"[37]. The sufficiency rating formula is a method of evaluating data by calculating four (4) separate factors to obtain a numeric value which is indicative of bridge sufficiency to remain in service. The result of this method is a percentage in which 100 percent would represent an entirely sufficient bridge and zero percent would represent an entirely insufficient bridge. A bridge with a sufficiency rating of 80 or less is eligible for federal bridge rehabilitation funding. A bridge with a sufficiency

[37]Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges, U.S. Department of Transportation, Federal Highway Administration, Office of Engineering Bridge Division, December 1995.

rating of less than 50 is eligible for federal bridge replacement funding. The I-35W Bridge (Bridge #9340) had a sufficiency rating of 50.

The sufficiency rating formula is calculated by the following equation:

$$\text{Sufficiency Rating} = S_1 + S_2 + S_3 - S_4$$

Where:

$S_1$ = Structural Adequacy and Safety of the bridge ($S_1$ = 55% maximum)
$S_2$ = Serviceability and functional obsolescence of the bridge ($S_2$ = 30% maximum)
$S_3$ = Essentiality of the bridge for public use ($S_3$ = 15% maximum)
$S_4$ = Special reductions for the bridge ($S_4$ = 13% maximum)

$S_1$, $S_2$, $S_3$, and $S_4$ are each calculated by a formula that takes into account the following structure inventory and appraisal data:

$S_1$ = Structural Adequacy and Safety of the bridge

- Superstructure (Item #59)
- Substructure (Item #60)
- Culverts (Item #62)
- Inventory Rating (Item #66)

$S_2$ = Serviceability and functional obsolescence of the bridge

- Lanes on Structure (Item #28)
- Average Daily Traffic (Item #29)
- Approach Roadway Width (Item #32)
- Structure Type Main (Item #43)
- Bridge Roadway Width (Item #51)
- Vertical Clearance over Deck (Item #53)
- Deck Condition (Item #58)
- Structural Evaluation (Item #67)
- Deck Geometry (Item #68)
- Underclearances (Item #69)
- Waterway Adequacy (Item #71)
- Approach Roadway Alignment (Item #72)
- Defense Highway Designation (Item #100)

$S_3$ = Essentiality of the bridge for public use

- Detour Length (Item #19)
- Average Daily Traffic (Item #29)
- Defense Highway Designation (Item #100)

$S_4$ = Special reductions for the bridge

- Detour Length (Item #19)
- Traffic Safety Features (Item #36)
- Structure Type Main (Item #43)

## 9.1 INSPECTION AND MANAGEMENT OF BRIDGES WITH FRACTURE-CRITICAL DETAILS

The Transportation Research Board (TRB) produced a National Cooperative Highway Research Program (NCHRP) Synthesis Report 354 entitled Inspection and Management of Bridges with Fracture-Critical Details[38] dated 2005. The report indicated the following:

> *"Presently, material selection, design, and fabrication of steel bridges are governed by*
>
> - *AASHTO LRFD Bridge Design Specifications and*
> - *AASHTO/AWS-D1.5, Bridge Welding Code.*
>
> *In addition to CVN requirements, these provisions restrict the choice of details as well as control weld flaws and other crack-like defects. These provisions have reshaped industry practices and result in an acceptably low probability of fatigue cracking and brittle fracture in new bridges.*
>
> *However, many older steel bridges built before the implementation of modern fatigue design provisions in the mid-1970s possess poor fatigue details, such as cover plates that can develop fatigue cracks, which if not repaired, can grow and lead to fracture of the member and possible collapse of part or all of the bridge.*
>
> *Other factors that make these older bridges susceptible to fracture include:*
>
> - *Marginal fracture toughness of the steel and weld metal;*
> - *Detailing, fabrication quality, and shop inspection below modern standards;*
> - *Severe corrosion problems, especially at open or failing expansion joints:*
> - *Higher traffic volumes and truck weights than the bridge was originally designed to handle.*
>
> *In light of these factors, periodic in-service inspection is particularly important for older bridges to provide an opportunity to detect cracks and corrosion before they grow to a critical size. In 1970, partly in reaction to the collapse of the Point Pleasant Bridge over the Ohio River, the NBIS was established. Title 23, Code of*

[38] Robert J. Conner, Robert Dexter, and Hussam Mahmoud, National Cooperative Highway Research Program (NCHRP) Synthesis Report 354, Inspection and Management of Bridges with Fracture-Critical Details, Transportation Research Board, Washington, D.C., 2005, pages 7 and 8.

*Federal Regulations, Part 650, Subpart C sets forth the NBIS for all bridges of more than 20 feet span on all public roads. Section 650.3 specifies inspection procedures and frequencies, indicates minimum qualifications for personnel, and states reporting, inventory, load posting, and inspection recordkeeping requirements. The current NBIS mandates a 2-year inspection interval for all highway bridges carrying public roads.*

*However, modern steel bridges are not nearly as susceptible to fracture as older bridges. As a result, the ways modern bridges are managed could possibly be evaluated differently than older bridges. This could be studied further, with considerable potential benefits.*

*For example, problems with severe corrosion have been reduced. In the last 20 years, durability of weathering steel and coating systems has improved. Expansion joints have been improved if not eliminated through the use of continuous jointless bridges.*

*In addition, there have been few if any cases where weld defects or low-toughness steel has been an issue for modern steel bridges, owing primarily to improvements in details, fabrication practices, and fracture toughness of the steel and weld metal. If spontaneous fracture from weld defects is rules out, then fracture can only occur if preceded by fatigue. Therefore, in this case, it is essentially sufficient to control fatigue to prevent fracture.*

*Distortion-induced fatigue cracking, discussed further in Appendix A, continued as a fatigue problem in typical plate girder bridges designed before the mid-1980s. A common example of distortion-induced fatigue cracking is web-gap cracking, which occurs in the gap when a connection plate is not attached to a flange and is subject to out-of-plane distortion. This problem was corrected in 1985 by a change in AASHTO specifications that mandated the attachment of the connection plate to both flanges.*

*Hence, it is important to distinguish three different age ranges of steel bridges:*

1. *Steel bridges built before the implementation of modern fatigue design provisions in the mid-1970's.*

2. *Steel bridges designed after the mid-1970's, but before 1985, which have fewer fatigue problems but remain susceptible to distortion-induced fatigue.*

3. *Modern steel bridges designed after 1985 that should not be susceptible to fatigue at all."*

## 10. A NATIONAL LOOK AT HOW LONG DECK TRUSS BRIDGES HAVE BEEN STRUCTURALLY DEFICIENT

The Federal Highway Administration (FHWA) Office of Bridge Technology sorted the National Bridge Inventory, at the request of NTSB investigators, to determine how long steel deck truss bridges have been structurally deficient. The FHWA Office of Bridge Technology determined the appropriate year to start the sort was 1990, since the data in the inventory received from the states prior to 1990, was found to be inconsistent. The FHWA Office of Bridge Technology provided the data contained in Table 16 on February 7, 2008.

**Table 16 – How long steel deck truss bridges have been structurally deficient since 1990**

| State | Number of Steel Deck Truss Bridges | Number of Steel Deck Truss Bridges that are Structurally Deficient | How long Steel Deck Truss Bridges have been Structurally Deficient since 1990[39] | | |
|---|---|---|---|---|---|
| | | | 0-5 years | 6-10 years | 11-17 years |
| Alabama | 2 | 1 | | 1 | |
| Alaska | 7 | 3 | | | 3 |
| Arizona | 7 | 3 | 1 | 2 | |
| Arkansas | 11 | 3 | | 1 | 2 |
| California | 50 | 22 | 1 | 9 | 12 |
| Colorado | 6 | 1 | 1 | | |
| Connecticut | 5 | 2 | | 2 | |
| Delaware | 1 | 0 | | | |
| District of Columbia | 0 | 0 | | | |
| Florida | 0 | 0 | | | |
| Georgia | 2 | 0 | | | |
| Hawaii | 2 | 0 | | | |
| Idaho | 7 | 2 | | 1 | 1 |
| Illinois | 17 | 3 | | 1 | 2 |
| Indiana | 9 | 5 | 2 | 1 | 2 |
| Iowa | 9 | 8 | 2 | | 6 |
| Kansas | 13 | 6 | 1 | 3 | 2 |
| Kentucky | 9 | 2 | 1 | | 1 |
| Louisiana | 2 | 0 | | | |
| Maine | 6 | 1 | 1 | | |
| Maryland | 9 | 0 | | | |
| Massachusetts | 18 | 4 | 1 | 2 | 1 |
| Michigan | 4 | 2 | 1 | 1 | |
| Minnesota | 4 | 1 | | | 1 |

[39]FHWA does not consider a bridge structurally deficient if the bridge has been constructed or had major reconstruction within the past 10 years. The list does not include how long steel deck truss bridges have been functionally obsolete since 1990.

| | | | | | |
|---|---|---|---|---|---|
| Mississippi | 0 | 0 | | | |
| Missouri | 4 | 1 | | 1 | |
| Montana | 9 | 2 | | 1 | 1 |
| Nebraska | 2 | 1 | 1 | | |
| Nevada | 1 | 0 | | | |
| New Hampshire | 3 | 1 | | | 1 |
| New Jersey | 8 | 5 | 2 | 1 | 2 |
| New Mexico | 5 | 0 | | | |
| New York | 32 | 6 | | | 6 |
| North Carolina | 1 | 0 | | | |
| North Dakota | 0 | 0 | | | |
| Ohio | 16 | 3 | | 2 | 1 |
| Oklahoma | 10 | 3 | | | 3 |
| Oregon | 37 | 12 | 3 | 6 | 3 |
| Pennsylvania | 48 | 16 | 2 | 4 | 10 |
| Rhode Island | 0 | 0 | | | |
| South Carolina | 0 | 0 | | | |
| South Dakota | 2 | 1 | | | 1 |
| Tennessee | 6 | 1 | 1 | | |
| Texas | 8 | 2 | 2 | | |
| Utah | 1 | 1 | | | 1 |
| Vermont | 8 | 6 | 4 | 2 | |
| Virginia | 9 | 6 | 1 | | 5 |
| Washington | 22 | 3 | 2 | 1 | |
| West Virginia | 15 | 2 | | | 2 |
| Wisconsin | 14 | 3 | 2 | 1 | |
| Wyoming | 3 | 1 | | | 1 |
| Puerto Rico | 2 | 0 | | | |
| | | | | | |
| **Totals** | **466** | **145** | **32** | **43** | **70** |

Table 16 revealed, on a national basis, how long steel deck truss bridges have been structurally deficient for 3 time periods (0-5 years, 6-10 years, and 11-17 years). Approximately 50% (or 70 out of a total of 145) of all steel deck truss bridges have been structurally deficient for 11 to 17 years.

## 11. MN/DOT POLICIES AND PROCEDURES WITH RESPECT TO CONSTRUCTION LOADING ON BRIDGES

Mn/DOT provided the policies and procedures with respect to construction loading on bridges in a December 19, 2007 letter to NTSB investigators (See Attachment 6 – Letter to the National Transportation Safety Board from the Minnesota Department of Transportation dated December 19, 2007). The following excerpts are taken from the letter:

*"Mn/DOT policies and procedures with respect for construction loading are as follows;*

*Since 1968 the Mn/DOT Standard Specifications for Construction manual contains language on limiting loads in Section 1513. Section 1513 in the current 2005 manual states that a contractor shall comply with the same load restrictions as normal legal traffic. The restrictions are for completed structures or those under construction. The legal limits are defined per Minnesota Statute of the Highway Traffic Regulation Act Chapter 169. Section 1513 from the 1968 and 2005 manuals are attached to this document as Figure A and B, respectively. Also per Section 2401.3G (Figure C) the contractor shall not prematurely load newly placed concrete elements until proper curing is completed.*

*The contractor can request to place larger than legal loads on a new or remodeled bridge with Mn/DOT Construction Project Engineers approval. Although not a written policy, when a contractor proposes a load that exceeds legal loads, it is a practice for the Mn/DOT Construction Project Engineer to consult with the Regional Construction Engineer in the Bridge Office. The construction loading information is provided to the Load Rating Unit or Design Unit for evaluation to determine if the loading is acceptable or if any special procedures such as use of the load distribution mats are required. Some examples of loads that exceed legal loads are mobile cranes or heavy earth moving equipment."*

The Mn/DOT Standard Specifications for Construction Section 1513[40] indicated the following:

*"**Restrictions on Movement of Heavy Loads and Equipment***

*The hauling of materials and the movement of equipment to and from the Project and over completed structures, base courses, and pavements within the Project that are open for use by traffic and are to remain a part of the permanent movement, shall comply with the regulations governing the operation of vehicles on the highways of Minnesota, as prescribed in the Highway Traffic Regulation Act.*

*The Contractor shall comply with legal load restrictions, and with any special restrictions imposed by the Contract, in hauling materials and moving equipment over structures, completed upgrades, base courses, and pavements within the Project that are under construction, or have been completed but have not been accepted and opened for use by traffic.*

*The Contractor shall have a completed Weight Information Card in each vehicle used for hauling bituminous mixture, aggregate, batch concrete, and grading*

[40]Standard Specifications for Construction, 2005 Edition, Minnesota Department of Transportation, Division I, Section 1513 Restrictions on Movement of Heavy Loads and Equipment, page 49.

*material (including borrow and excess) prior to starting work. This card shall identify the truck or tractor and trailer by Minnesota or prorated license number and shall contain the tare, maximum allowable legal gross mass, supporting information, and the signature of the owner. The card shall be available to the Engineer upon request. All Contractor-related costs in providing, verifying, and spot checking the cab card information (including weighing trucks on certified commercial scales, both empty and loaded) will be incidental, and no compensation other than for Plan pay items will be made.*

*Equipment mounted on crawler tracks or steel-tired wheels shall not be operated on or across concrete or bituminous surfaces without specific authorization from the Engineer. Special restrictions may be imposed by the Contract with respect to speed, load distribution, surface protection, and other precautions considered necessary.*

*Should construction operations necessitate the crossing of an existing pavement or completed portions of the pavement structure with equipment or loads that would otherwise be prohibited, approved methods of load distribution or bridging shall be provided by the Contractor at no expense to the Department.*

*Neither by issuance of a special permit, nor by adherence to any other restrictions imposed, shall the Contractor be relieved of liability for damages resulting from the operation and movement of construction equipment."*

The Minnesota Statutes, Chapter 169 indicated the following regarding load limits on bridges:

*"**169.84 LOAD LIMIT ON BRIDGE***

*Subject to the limitations upon wheel and axle loads prescribed in this chapter, the gross weight of any vehicle or combination of vehicles driven onto or over a bridge on any highway shall not exceed the safe capacity of the bridge, as may be indicated by warning posted on the bridge or the approaches thereto."*

The AASHTO Standard Specifications for Highway Bridges, 17th Edition, contained language regarding the application of construction loads. The AASHTO Standard Specifications for Highway Bridges[41] indicated the following:

*"**Construction Loads***

*Otherwise, loads imposed on existing, new or partially completed portions of structures due to construction operations shall not exceed the load-carrying capacity of the structure, or portion of structure, as determined by the Load Factor Design methods of AASHTO using Load Group IB. The compressive*

[41]Standard Specifications for Highway Bridges, 17th Edition – 2002, American Association of State Highway and Transportation Officials, 8.15.3 Construction Loads, page 547.

*strength of concrete ( $f_c^{'}$ ) to be used in computing the load-carrying capacity shall be the smaller of the actual compressive strength at the time of loading or the specified compressive strength of the concrete."*

The AASHTO LRFD Bridge Design Specifications, 4th Edition, contained language regarding load factors for construction loads. The AASHTO LRFD Bridge Design Specifications[42] indicated the following:

*"**Load Factors for Construction Loads*** **(Commentary Section)**

*The load factors presented here should not relieve the contractor of responsibility for safety and damage control during construction.*

*Construction loads are permanent loads and other loads that act on the structure only during construction. Construction loads include the weight of equipment such as deck finishing machines or loads applied to the structure through falsework or other temporary supports. Often the construction loads are not accurately known at design time; however, the magnitude and location of these loads considered in the design should be noted on the contract documents."*

### 11.1 NTSB REQUEST FOR MN/DOT TO PERFORM LOAD RATING FOR CONSTRUCTION MATERIAL AND EQUIPMENT THAT WAS PLACED ON BRIDGE

In an October 15, 2007 email, NTSB investigators requested Mn/DOT perform a load rating based on the weight of the construction material and equipment on the bridge prior to the collapse (See Attachment 30 – Email to the Minnesota Department of Transportation from the National Transportation Safety Board dated October 15, 2007). NTSB investigators requested that Mn/DOT perform the rating using standard procedures and not refer to any information learned about the bridge since the collapse.

Mn/DOT responded to the NTSB request in a November 9, 2007 email transmitting a report entitled "Load Rating of Bridge 9340 with Construction Loads" (See Attachment 31 – Mn/DOT transmittal of report to NTSB entitled Load Rating of Bridge 9340 with Construction Loads). The report indicated that a proposal to place construction material and equipment on the bridge would likely have been rejected by Mn/DOT. However, a detailed analysis of the proposal would have shown that all rating factors were acceptable. The following excerpts are taken from the report:

*"Had this proposal been forwarded to us from the contractor at the start of the overlay contract, we would likely have rejected it, before doing any analysis for the loads. We would have questioned if there were alternate locations for stockpiling the materials. This loading is immediately seen to be much larger*

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[42]LRFD Bridge Design Specifications, 4th Edition – 2007, American Association of State Highway and Transportation Officials, 3.4.2 Load Factors for Construction Loads (Commentary Section), page 3-14.

*than design loads. For example, the HS 20 design lane load is 0.64 k / ft. The rock and sand piles weigh about four times as much as this, spread over a width of 14 ft., just slightly more than a design lane…*

*…In conclusion, the most direct response to your question is we would have likely denied the contractors request based on a quick and fairly simple review. However a more rigorous analysis shows all rating factors to be above one.”*

### 11.2 MN/DOT REVISED ITS STANDARD SPECIFICATIONS FOR CONSTRUCTION TO ADDRESS THE STORAGE OF CONSTRUCTION MATERIALS ON BRIDGES

On September 18, 2008 Mn/DOT revised its Standard Specifications for Construction, Section 1513, to address the storage of construction materials on bridges. The revisions indicated the following:

*“Unless specifically allowed in the Contract, or approved by the Engineer, all construction material and/or equipment which might be temporarily stored or parked on a bridge deck while the bridge is under construction will be limited by this specification. These requirements are intended to limit construction loads to levels commensurate with the typical design live load. The storage of materials and equipment as a whole will be limited to all of the following:*

- *Combinations of vehicles, materials, and other equipment are limited to a maximum weight of 31,702 kg/100 $m^2$ (65,000 lbs./1000 $ft^2$).*
- *Material stockpiles (including but not limited to pallets of products, reinforcing bar bundles, aggregate piles) are limited to a maximum weight of 12,200 kg/10 $m^2$ (25,000 lbs./100 $ft^2$).*
- *Combinations of vehicles, materials, and other equipment are limited to a maximum weight of 90,700 kg (200,000 lbs.) per span.*

*The Contractor may submit alternate loadings to the Project Engineer 30 Calendar days prior to placement. Any submittals will require the calculations be certified by a Professional Engineer.”*

## 12. PROPOSAL FOR STEEL PLATING OF 52 FRACTURE CRITICAL TRUSS MEMBERS

The URS Corporation proposed to strengthen 20 fracture critical main truss members of the bridge in July 2006. The URS Corporation later increased the number from 20 to 52 fracture critical main truss members as one of three equally viable retrofit approaches proposed in a January 2007 executive summary of a Fatigue Evaluation and Redundancy Analysis Draft Report for Bridge #9340 (See Attachment 13 – URS Corporation Fatigue Evaluation and Redundancy Analysis Draft Report for Bridge #9340, Executive Summary, dated January 2007).

The URS Corporation proposal was to bolt high strength steel plates onto the sides of fracture critical main truss members to fully replace the strength of the critical member should it crack, and also making the member internally redundant (less susceptible to failure) if the crack became critical. URS recommended steel plating 20 members initially (later the number was increased to 52) in order to prevent possible failure[43]. A rough estimate for the steel plating of 20 members was $1 to $1.25 million dollars.

Table 17 shows a timeline for Mn/DOT's decision-making process to either steel plate or inspect the fracture critical main truss members. Mn/DOT had plans to steel plate the fracture critical main truss members from July 24, 2006 through December 19, 2006. A decision was made by Mn/DOT on January 17, 2007 to switch to non-destructive examination (NDE) inspection based on a proposal made by the URS Corporation. Mn/DOT acknowledged the main reasons for the switch were the expectation inspection staff could locate with visual and ultrasonic testing the critical crack size URS identified, plus the concern for introducing a defect during drilling and attachment of the plates. A copy of the minutes of meetings, emails, and handwritten notes from July 24, 2006 through May 2007 was obtained by NTSB investigators (See Attachment 32 – Copy of minutes of meetings, emails, and handwritten notes from July 24, 2006 through May 2007 as it relates to proposal for steel plating of all 52 fracture critical truss members).

**Table 17 – Mn/DOT's decision-making process to either steel plate or inspect the fracture critical main truss members**

| Date | Document / Discussion |
|---|---|
| July 24, 2006 | Minutes of meeting discussed possible future plating contract. |
| October 4, 2006 | Inquiry from Metro project manager about cost of plating. |
| October 16, 2006 | Email indicated plating estimate was $1 to $1.25 million. |
| November 1, 2006 | Email indicated Metro decision to fund $1.5 million plating project with a scheduled letting for construction in January 2008. |
| November 7, 2006 | Email indicated a monitoring system should be considered if URS believed the plating was a risk. |
| November 14, 2006 | Handwritten notes on email indicated the state bridge engineer discussed monitoring option with URS. URS expressed confidence in plating option. Mn/DOT decided to remain with plating plan rather than pursue an uncertain monitoring system. |
| November 21, 2006 | Email indicated URS was asked to prepare plans and special provisions for an October 2007 plating contract letting. |
| November 27, 2006 | Email indicated further discussions with URS on scope of work. |
| December 4, 2006 | Minutes of meeting indicated Metro, Bridge, and URS discussed actions needed to meet October 2007 plating contract letting. |
| December 19, 2006 | URS informed Mn/DOT that there are alternatives to plating to consider. January draft of eventual 2007 executive summary, with 3 equally viable options was sent. |

[43]Plating provides internal redundancy to individual members of the truss to which it is applied. Although it could have possibly prevented collapse due to fracture of one of the truss members, the plating would not have altered or prevented the failure of the gusset plates.

| | |
|---|---|
| December 19, 2006 | Discussed plating job at Associated General Contractor (AGC) meeting to ask input from interested contractors. |
| December 21, 2006 | Email indicated that in depth inspection and non-destructive examination (NDE) option proposed by URS may be feasible since Mn/DOT should be able to detect the size of the crack. |
| December 28, 2006 | Email indicated that packages were sent of scope, truss elevation, and typical plating detail to the Associated General Contractor (AGC) for comments. |
| January 9, 2007 | Email indicated that the Associated General Contractor (AGC) invited member contractors to give input on the plating contract. |
| January 10, 2007 | Email indicated a proposed meeting to discuss how non-destructive examination (NDE) can be used to limit proposed areas of plating. URS discussed the possibility of not plating any members. |
| January 12, 2007 | Email indicated conference call with URS scheduled for January 17, 2007. |
| January 17, 2007 | Handwritten notes of state bridge engineer on email indicated detailed review of fatigue using fracture mechanics. A decision was made to do non-destructive examination (NDE) of south span in 2007. If confident of visual and ultrasonic testing, than proceed to north span. If not confident, go with plating repair. |
| January 18, 2007 | Email indicated that Mn/DOT would reschedule October 2007 plating contract letting until FY2009 after an evaluation of in-depth visual and non-destructive examination (NDE) inspection methods and results are completed. |
| March 2007 | Contract signed with URS to review Mn/DOT's inspection results and to develop repair plans and specifications if plating necessary. |
| May 2007 | Mn/DOT and Metro inspection teams perform in-depth and non-destructive examination (NDE) inspection of half of the critical members identified in the URS report. |
| August 20, 2007 | A meeting was scheduled, however was not held due to the bridge collapse, to review methods and results and to determine if inspections should continue in lieu of plating, or to proceed with plating. |

NTSB investigators requested a copy of the May 2007 field notes and photographs taken by the Mn/DOT inspection team (See Attachment 33 – Field notes taken by Mn/DOT Inspection Team dated May 2007). The inspection used visual and non-destructive examination (NDE) methods. The field notes were not written into a final report since the bridge collapse occurred before the scheduled August 20, 2007 meeting. The inspection focused on all 26 members of interest on the west truss and several members on the south end of the east truss. The field notes and photographs taken by the Mn/DOT inspection team did not reveal any significant cracks in the fracture critical main truss members.

## 13. WASHOUT HOLE DISCOVERED BEFORE COLLAPSE

A washout hole was discovered before the collapse by Mn/DOT inspectors in December of 2006. The washout hole was located between Pier 4 and 5 and estimated to be 4 feet by 6 feet in plan and 2 feet deep. Mn/DOT provided the following information to NTSB investigators regarding the washout hole (See Attachment 34 – Mn/DOT correspondence and photographs of washout hole between Pier 4 and 5):

> *"The erosion was noted in December of 2006 and some temporary repairs were made in January of 2007. Maintenance crews came back in early July 2007 to conduct further investigation and complete repair. First thought was that it might be a "sinkhole" which is generally caused by something creating a void underground, and then having the earth fall into that void, causing a hole on the surface. After further investigation, maintenance crews determined that it was a washout caused by water draining from the bridge on to an angled pavement bed below the bridge. As the water hit the concrete, it began to work its way into cracks, and eventually began to wash away earth beneath the concrete. The hole in picture two was caused because the earth kept washing away and eventually it pulled the earth from underneath the blacktop, causing the hole to form. Mn/DOT maintenance began a repair in July by putting five yards of a very moist concrete into the hole (a yard is equivalent to 9 wheelbarrows). They found that the concrete, which was fairly runny, began to seep out of spots at the bottom of the slope. This helped confirm that it was indeed a washout and not a sinkhole. The repair continued with a number of loads of rock and more concrete. The last time the crew was working on the washout repair was July 25."*

## 14. DESIGN SPECIFICATIONS FOR GUSSET PLATES

The I-35W Bridge (Bridge #9340) was designed by the engineering consultant firm of Sverdrup & Parcel and Associates, Inc. Sverdrup & Parcel and Associates, Inc. was a predecessor company of Sverdrup Corporation, a company acquired by Jacobs Engineering Group Inc. in 1999. NTSB investigators requested the detailed calculations for the main truss gusset plates of the I-35W Bridge from Jacobs Engineering Group, Inc. and Mn/DOT. Although copies of the original design and fabrication drawings for the bridge were provided to NTSB investigators, the detailed calculations for the main truss gusset plates could not be located.

The general notes[44] on the 1965 construction drawings for the I-35W Bridge indicated the design data conform to the 1961 AASHO (American Association of State Highway Officials) Design Specifications and the 1961 and 1962 Interim Specifications modified by Minnesota Highway Department standards on allowable stresses:

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[44]State of Minnesota Department of Highways, Construction Plan for Bridge 9340, Trunk Highway No. 35W between Washington Ave. on the west bank of the Mississippi River to University Ave. on the east bank of the Mississippi River, Minnesota Project No. I-IG 35W-3 (58)112, June 18, 1965, Sheet No. 2 of 94.

*"Design Data*

*1961 A.A.S.H.O. Design Specifications and 1961 and 1962 Interim Specifications.*

*H20-S16-44 Loading and alternate loading designated in P.P.M. 20-4, Section 4C.*

*Allowable Design Stresses:*

$f_c = 1{,}600$ *psi n=8 (Based on* $f'_c = 4{,}000$ *psi)*

$f_s = 20{,}000$ *psi Intermediate grade reinforcement*

$f_s = 20{,}000$ *psi Structural Steel M.H.D. 3306*

$f_s = 27{,}000$ *psi Structural Steel M.H.D. 3309*

$f_s = 45{,}000$ *psi Structural Steel M.H.D. 3318*

*Structural Steel M.H.D. 3310 as follows:*

$f_s = 27{,}000$ *psi ¾" thickness and under*

$f_s = 24{,}000$ *psi over ¾" to 1½" thickness incl.*

$f_s = 22{,}000$ *psi over 1½" to 4" thickness incl."*

The 1961 AASHO Standard Specifications for Highway Bridges[45] indicated the requirements for designing a gusset plate:

*"The gusset plates shall be of ample thickness to resist shear, direct stress, and flexure, acting on the weakest or critical section of maximum stress."*

In addition, the 1961 AASHO Standard Specifications for Highway Bridges indicated the requirements for stiffening a gusset plate:

*"If the unsupported edge of a gusset plate exceeds the following number of times its thickness, the edge shall be stiffened:*

*60 for carbon steel.*
*50 for silicon steel.*
*48 for low-alloy steel.*
*45 for nickel steel."*

The requirements for designing and stiffening a gusset plate have not changed significantly from the 1961 AASHO Standard Specifications for Highway Bridges to the current 2002 American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges.

[45] Standard Specifications for Highway Bridges, 8th Edition – 1961, American Association of State Highway Officials, 1.6.34 Gusset Plates, page 77.

NTSB investigators determined through interviews with leading experts in the bridge industry, the design methodology for gusset plates is normally very conservative, with the result that a properly designed gusset plate should generally be stronger than the beams it connects.

The 2007 AASHTO LRFD Bridge Design Specifications, Fourth Edition[46], provided the following information regarding the design of connections and splices:

*"Except as specified otherwise, connections and splices for primary members shall be designed at the strength limit state for not less than the larger of:*

- *The average of the flexural moment-induced stress, shear, or axial force due to the factored loadings at the point of splice or connection and the factored flexural, shear, or axial resistance of the member or element at the same point, or*

- *75 percent of the factored flexural, shear, or axial resistance of the member or element."*

The 1999 Structural Steel Designer's Handbook, Third Edition[47], and the 1994 Structural Steel Designer's Handbook, Second Edition[48], provided the following information regarding the design of gusset plates:

*"At every joint in a truss, working lines of the intersecting members preferably should meet at a point to avoid eccentric loading. While the members may be welded directly to each other, most frequently they are connected to each other by bolting to gusset plates. Angle members may be bolted to a single gusset plate, whereas box and H shapes may be bolted to a pair of gusset plates.*

*A gusset plate usually is a one-piece element. When necessary, it may be spliced with groove welds. When the free edges of the plate will be subjected to compression, they usually are stiffened with plates or angles. Consideration should be given in design to the possibility of the stresses in gusset plates during erection being opposite in sense to the stresses that will be imposed by service loads.*

*Gusset plates are sometimes designed by the method of sections based on conventional strength of materials theory. The method of sections involves investigation of stresses on various planes through a plate and truss members. Analysis of gusset plates by finite-element methods, however, may be advisable where unusual geometry exists.*

[46] LRFD Bridge Design Specifications, 4th Edition – 2007, American Association of State Highway and Transportation Officials, 6.13 Connections and Splices, page 6-193.
[47] Roger L. Brockenbrough and Frederick S. Merritt, Structural Steel Designer's Handbook, Third Edition, 1999, 13.12 Truss Joint Design Procedure, page 13.35.
[48] Roger L. Brockenbrough and Frederick S. Merritt, Structural Steel Designer's Handbook, Second Edition, 1994, 12.10 Truss Joint Design, page 12.27.

*Transfer of member forces into and out of a gusset plate invokes the potential for block shear around the connector groups and is assumed to have about a 30 degree angle of distribution with respect to the gage line, as illustrated in Fig. 12.9."*

The 1972 Structural Steel Designer's Handbook[49] provided the following information regarding the design of gusset plates:

*"Except when pin-connected, main truss members preferably should be connected with gusset plates. To avoid eccentricity, fasteners connecting each member should be symmetrical about the axis of each. It is desirable that the fasteners develop the full capacity of each element of the member.*

*Thickness of a gusset plate should be adequate for resisting shear, direct stress, and flexure at critical sections, where these stresses are maximum. Reentrant cuts should be avoided. (Curves made for appearance, however, are permissible.)*

*If the ratio of length of unsupported edge to thickness of a gusset plate exceeds* $347/\sqrt{F_y}$*, where* $F_y$ *= steel yield strength, ksi, the edge should be stiffened."*

The 1972 Design in Structural Steel[50] textbook provided the following information regarding the design of gusset plates:

*"A plate used to connect the members of a truss at a joint or to perform a similar function in another type of structure is called a gusset plate. A typical gusset plate in a riveted or bolted joint is illustrated in Fig. 7-12. Gusset plates are not really designed in the sense that on analysis they satisfy certain design criteria. Instead, the number of fasteners required in each of the connecting members is determined and the plate is simply made large enough to accommodate them."*

The 1920 Design of Highway Bridges of Steel, Timber and Concrete[51] textbook provided the following information regarding the design of gusset plates:

*"The gusset plates will be made at least thick enough to develop in bearing, the strength of the rivets in single shear...*

*...The plates must be of sufficient size to contain the necessary rivets and to carry the stresses transmitted from the members."*

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[49]Frederick S. Merritt, Structural Steel Designer's Handbook, 1972, 12-11 Gusset Plates, page 12-12.
[50]Carl L. Shermer, Design in Structural Steel, Ohio University, 1972, 7-6 Riveted or Bolted Gusset-Plate Connections, page 210.
[51]Milo S. Ketchum, The Design of Highway Bridges of Steel, Timber and Concrete, Second Edition, 1920, Design of Joints, page 223.

### 14.1 FHWA TURNER-FAIRBANK HIGHWAY RESEARCH CENTER INTERIM REPORT, ADEQUACY OF THE U10 & L11 GUSSET PLATE DESIGNS, JANUARY 11, 2008

The FHWA Turner-Fairbank Highway Research Center prepared an interim report entitled Adequacy of the U10 & L11 Gusset Plate Designs for the Minnesota Bridge No. 9340 (I-35W over the Mississippi River) dated January 11, 2008 (See FHWA Turner-Fairbank Highway Research Center Interim Report Adequacy of the U10 & L11 Gusset Plate Designs for the Minnesota Bridge No. 9340 (I-35W over the Mississippi River) dated January 11, 2008). The following excerpts are taken from the interim report:

> *"One of the tasks performed by the FHWA team was a review and assessment of the original bridge design calculations by Sverdrup & Parcel. This report will focus on the findings of this assessment unique to the gusset plate design methodology used for the primary truss and more specifically the design of the gusset plates at locations U10 and L11. The initial onsite investigation of the collapsed structure identified the failure of the U10 gusset plates as occurring early in the event. The L11 gusset plates are detailed similarly to those at U10…*
>
> *…INTERPRETATION OF RESULTS…*
>
> *…The gusset plates at U10 and L11 consistently failed the D/C ratio checks conducted and the U10 gussets also violated the unsupported edge limitations. The capacity inadequacies were considerable for all conditions investigated with the plate providing approximately one-half of the resistance required by the design loadings."*

## 15. CONCRETE CORE TESTING OF BRIDGE DECK AFTER BRIDGE COLLAPSE

After the collapse, Mn/DOT conducted concrete core testing to determine the average thickness of the bridge deck (See Attachment 35 – Mn/DOT concrete core testing of bridge deck dated August 23, 2007). Table 18 shows the average thickness of the bridge deck on the truss spans that corresponds to the direction of travel, lane assignment, and re-decking status at the time of the collapse.

**Table 18 – Results of concrete core testing of bridge deck on the truss spans after the collapse**

| Direction of Travel | Lane Assignment | Re-decking Status at the time of the collapse | Average Thickness |
|---|---|---|---|
| Southbound | Outside Half | New overlay completed | 8.85 inches |
| Southbound | Inside Half | Existing overlay milled, no new overlay in place | 6.98 inches |
| Northbound | Outside Half | New overlay completed | 8.79 inches |
| Northbound | Inside Half | Existing overlay in place | 8.69 inches |

## E. ATTACHMENTS

Attachment 1 – National Bridge Inventory (NBI) forms for the I-35W Bridge (Bridge #9340) from 1983 through 2007

Attachment 2 – Letter to the National Transportation Safety Board from the Minnesota Department of Transportation dated April 22, 2008

Attachment 3 – Minnesota Department of Transportation Bridge Inspection Manual dated May 2007

Attachment 4 – Mn/DOT Bridge Inspection Reports dated August 5, 1990 and October 18, 1993

Attachment 5 – Mn/DOT Fracture Critical Bridge Inspection In-Depth Report dated June 2006

Attachment 6 – Letter to the National Transportation Safety Board from the Minnesota Department of Transportation dated December 19, 2007

Attachment 7 – FHWA's Apportionment Process for Highway Bridge Program (HBP) Funds

Attachment 8 – Mn/DOT Fracture Critical Bridge Inspection Annual Report dated June 2004

Attachment 9 – Letter to the National Transportation Safety Board from the Minnesota Department of Transportation dated September 21, 2007

Attachment 10 – Mn/DOT Bridge Inspection Report dated June 15, 2006

Attachment 11 – Mn/DOT Guidelines for In-Depth Inspection of Failure Critical and Other Non-Redundant Bridges and for Underwater Inspections dated July 19, 2007

Attachment 12 – Mn/DOT Technical Memorandum regarding "Critical Deficiencies" found during bridge inspections dated July 20, 2005

Attachment 13 – URS Corporation Fatigue Evaluation and Redundancy Analysis Draft Report for Bridge #9340, Executive Summary, dated January 2007

Attachment 14 – URS Corporation Fatigue Evaluation Redundancy Analysis Draft Report for Bridge #9340 dated July 2006

Attachment 15 – URS Corporation Fatigue Evaluation Bridge 9340 35W over Mississippi River Second Inspection Report dated November 17, 2003

Attachment 16 – URS Corporation Fatigue Evaluation Bridge 9340 35W over Mississippi River Initial Inspection Report dated June 9, 2003

Attachment 17 – University of Minnesota Fatigue Evaluation of the Deck Truss of Bridge 9340 Final Report dated March 2001

Attachment 18 – Mn/DOT Memorandums documenting cracks in the north approach span girder near Pier 9 from October 1998 through November 2000

Attachment 19 – Ayres Associates Underwater Bridge Inspection Report for Bridge #9340 dated December 8, 2004

Attachment 20 – Email to the National Transportation Safety Board from the Minnesota Department of Transportation dated January 10, 2008

Attachment 21 – Mn/DOT LRFD Bridge Design Manual, Chapter 15 Bridge Rating, dated June 2007 Draft

Attachment 22 – Mn/DOT Bridge Rating and Load Posting Report dated December 14, 1995

Attachment 23 – Letter to the National Transportation Safety Board from the Minnesota Department of Transportation dated January 8, 2008

Attachment 24 – Mn/DOT BARS load rating computer printout dated December 11, 1995

Attachment 25 – Mn/DOT BARS load rating computer printout dated August 18, 1997

Attachment 26 – Email to the National Transportation Safety Board from the Minnesota Department of Transportation dated November 2, 2007

Attachment 27 – Mn/DOT Bridge Rating and Load Posting Report dated September 17, 1979

Attachment 28 – Email to the National Transportation Safety Board from the Minnesota Department of Transportation dated October 10, 2007

Attachment 29 – FHWA Turner-Fairbank Highway Research Center Report Assessment of the Load Rating Records for Minnesota Bridge No. 9340 (I-35W over the Mississippi River) dated June 30, 2008

Attachment 30 – Email to the Minnesota Department of Transportation from the National Transportation Safety Board dated October 15, 2007

Attachment 31 – Mn/DOT transmittal of report to NTSB entitled Load Rating of Bridge 9340 with Construction Loads

Attachment 32 – Copy of minutes of meetings, emails, and handwritten notes from July 24, 2006 through May 2007 as it relates to proposal for steel plating of all 52 fracture critical truss members

Attachment 33 – Field notes taken by Mn/DOT Inspection Team dated May 2007

Attachment 34 – Mn/DOT correspondence and photographs of washout hole between Pier 4 and 5

Attachment 35 – Mn/DOT concrete core testing of bridge deck dated August 23, 2007

*Dan Walsh /s/*

**Dan Walsh, P.E.**
**Highway Accident Investigator**