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Incorporation of Fatigue Detail Classification of Steel Bridges into the Minnesota Department of Transportation Database

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16. Abstract (Limit: 200 words)

This report addresses the fatigue and fracture susceptibility of the Minnesota Department of Transportation (Mn/DOT) steel bridge inventory and provides a framework for enumerating fracture and fatigue sensitive details present in steel bridges. It also provides a method for rating the details in terms of their overall frequency and consequence of cracking. The research examines eighteen details identified as possible cracking locations, and a composite rank number is computed based on the details it possesses. The report includes a collection of case studies on cracking due to fatigue and fracture, as well as a timeline of changes to the American Association of State Highway and Transportation Officials Specifications for Highway Bridges and the Minnesota Standard Specifications for Highway Construction. Frequency of occurrence of cracking was defined on the basis of a national survey of state Departments of Transportation, from which the experiences regarding steel bridge cracking were collected from fifteen states. The results are organized in a comprehensive table correlating geometric constraints to rank numbers. The report concludes with a program that calculates a composite rank number for each bridge based upon distribution and rank of the individual details present in the bridge.

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Incorporation of Fatigue Detail Classification of Steel Bridges into the Minnesota Department of Transportation Database

Final Report

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EXECUTIVE SUMMARY

In order for the Minnesota Department of Transportation to estimate future spending and rehabilitation needs, it is important to establish the overall fatigue and fracture susceptibility of its bridge inventory. This report provides a framework for enumerating fracture and fatigue sensitive details present in steel bridges. It also provides a method for rating the details in terms of their overall frequency and consequence of cracking. The research includes thorough examination of eighteen details identified as possible cracking locations. A composite rank number which represents overall bridge susceptibility to fatigue and fracture is computed based on the details it possesses, enabling organization of bridges by vulnerability.

The research conducted includes a collection of case studies on cracking and predictive formulas, as well as a timeline of changes to the American Association of State Highway and Transportation Officials (AASHTO) Specifications for Highway Bridges and the Minnesota Standard Specifications for Highway Construction. Historic examples of cracking were assembled through examination of bridge plans and inspection reports. Frequency of occurrence was gathered by a national survey of the Departments of Transportation which collected data on the occurrence of steel bridge cracking from fifteen states across the nation. The most common problems were further analyzed with results from academic studies and then subdivided into different rank groups based on geometry.

The result is a comprehensive table correlating geometric constraints to rank numbers. A year is also given in the table which represents when use of a detail was ended, restricted by code, or eliminated by code. The project concludes with the implementation of a program into the Minnesota Department of Transportation (Mn/DOT) bridge database, which calculates a composite rank number for each bridge based upon distribution and rank of the individual details present in the bridge.

It was noted that little fracture and fatigue research has actually been performed in-situ on bridges (as opposed to laboratory models). Therefore, the ranking developed in this study is qualitative in nature and largely based on past experience. The only detail for which stresses can be easily calculated is the transverse stiffener web gap, based on a method developed by the University of Minnesota. These formulas require geometrical information that must be gathered, as well as accurate traffic counts for each bridge. Simplifications can be made to these formulas as well as those found in NCHRP 299 – "Fatigue Evaluation Procedures for Steel Bridges" to estimate fatigue life, but after simplifications, the estimations are comparable to the ranking developed in this report.

It is recommended that some form of the program developed here be used for actual implementation into Minnesota Department of Transportation bridge databases to gain a better understanding of the fatigue and fracture susceptibility of the bridge inventory. Comprehensive examination of bridge plans and inspections are the most efficient methods to collect information necessary for input into the program. Inspection records are necessary because some of the details are not shown in bridge plans as they result from improper field practices.

CHAPTER 1– INTRODUCTION

1.1 Overview

Fatigue in steel bridges is a major concern nationwide, as many bridges are approaching an age when fatigue life of certain details will be reached. Periodic inspections of these bridges may lead to the discovery of cracks requiring repair or retrofit. It is important therefore, to gain an understanding of how fatigue details reduce overall bridge inventory life. In doing so, fatigue and fracture susceptible bridges can be enumerated to help estimate future spending and rehabilitation needs.

Besides advancing steel bridge management and transferring valuable technology to the Minnesota Department of Transportation (Mn/DOT), implementation of procedures in this report can increase safety and reliability of the steel bridge inventory in Minnesota by facilitating the identification of bridges that are vulnerable to fatigue and fracture problems. This research will enable enhanced productivity and greater economy in bridge inspection, maintenance, and evaluation operations by allowing Mn/DOT staff to more accurately identify bridges with fatigue and fracture problems and assess their severity.

A system is developed for identification and gross ranking of bridges with high, medium, or low need for preventative maintenance or special inspection. It consists of a list of flagged fracture or fatigue susceptible details. It also includes a program which acquires data, such as geometry of certain details, and outputs a rank which classifies the severity and frequency of possible cracking for each detail. Bridges can then be ordered depending on which details they possess and their frequency of cracking. Below is a list of the major tasks necessary to rank bridges:

1. Identification of details with susceptibility to fracture and fatigue
2. Examination of case histories on fatigue and fracture in Minnesota and the United States and past research to discover the best approach to increase accuracy for the most common details
3. Examination of code changes regarding fatigue and fracture
4. Development of a procedure for an intern or field inspector to review the entire suite of bridges and collect information about details
5. Allocation of space in databases to contain necessary detail information
6. Development of a program to rank flagged details with fatigue or fracture susceptibility
7. Implementation of a program to output rank numbers to database

Flagged details include all of the fracture and fatigue susceptible details considered in this study and given a rank number. To rank the severity of flagged details, a comprehensive background of current knowledge was gathered. First, an extensive review of literature was performed, beginning with history of details that have shown a propensity for cracking. Next, a timeline was created, listing when problematic details were addressed in bridge design codes to identify which bridges were built after such changes and are thus exempt. By using geometries from actual bridges and combining this information with crack histories and past research, factors affecting cracking were identified.

To set the rank number for each type of detail, qualitative classification of acquired data was necessary rather than quantitative methods. A sample set of bridge plans was studied to discover what information could be gathered from them. A procedure was developed for extrapolation of information from plans, entry into the ranking program, and output to a database compatible format.

Research previously conducted on the topic of fatigue life estimation has left many loose ends in terms of applicability. Past research may have identified useful equations for estimating stresses based on parameters describing bridge geometry, material properties and loading; however, many times not all of these parameters are available. Furthermore, some procedures are far too complicated for applying them systematically to a large suite of bridges, and as such, simplifications must be made.

Use of the program and application of results is at the discretion of Mn/DOT. The conclusions in this report are based upon information obtained from existing research, evaluations, and code recommendations. No further experimentation was performed; therefore, the limitations of the conclusions depend upon the reliability of past work and experience. This report forms the basis for implementation of a ranking system for bridge management needs. This includes assessing the scope of individual bridge preservation projects, as well as identifying bridges most likely to fatigue. This report may be used for identification of details requiring special attention during inspections. It is not a substitute for periodic inspections, nor should the ranking be used as an evaluation tool for bridge safety.

1.2 Chapter Summaries

Chapter 2 – Provides the background information on fatigue details:

- identification of fatigue details by AASHTO *S-N* category, distortional fatigue summary, and case studies analyzed by Fisher (1984)
- NCHRP Report 299 fatigue life estimation procedure
- distortional stress approximations from University of Minnesota research

Chapter 3 – Provides the background information on fracture details:

- fracture code changes from the "Proposed Control Plan for New Bridges with Fracture Critical Members and the Inspection of Fracture Critical Bridge Members" (FHWA 1978)
- fracture case studies by Fisher (1984)

Chapter 4 – Provides information on geometric constraints for the five most common details:

- code language that classifies geometries based on *S-N* category
- other academic research that identifies effect of geometry

Chapter 5 – Specifies cracking case histories for Mn/DOT and the United States:

- Mn/DOT inspections review
- bridge plan review

- survey of other state Departments of Transportation (DOTs) for collection of frequency of cracking and any details not previously identified

Chapter 6 – Summarizes changes to bridge codes involving fracture and fatigue since 1964:

- general summary of American Association of State Highway and Transportation Officials (AASHTO) code
- specific wording changes to AASHTO and Minnesota codes
- timeline representing the changes

Chapter 7 – Provides the overall classification for flagged details:

- rank numbering is identified and described
- details are separated into geometric categories and rank is assigned
- a classification table is developed to summarize all information

Chapter 8 – Describes the program used for connecting geometry to bridge rank:

- blank spreadsheet for input is developed
- functioning of the program is described
- program is included
- examples of details are subjected to ranking program

Chapter 9 – Conclusions are drawn to summarize report:

- summary of information collected conclusions drawn
- recommendations for use and further research

Appendix A – Table of dates during which various types of steel were used
Appendix B – Rank number calculations for web gap detail
Appendix C – Responses from survey of DOTs

CHAPTER 2 – FATIGUE BACKGROUND

The process of discovering a reliable method for assessing the potential for bridge damage and remaining life requires a broad background into studies conducted on the subject. Gathering information on past instances of steel bridge cracking and compiling the affects of geometric factors is crucial. Only after gaining a broad understanding of these areas, can a classification system be formulated to rank details according to their probable necessity for repair. This chapter outlines some of the research performed, proposed design criteria, and established procedures, which create the foundation of classifications presented in later chapters.

Research of fatigue has been performed for many decades, in an attempt to quantify the sustainability of structures under repeated loading. Fatigue almost always precedes fracture and can therefore help to predict where rapid crack growth may occur. A large body of research on fatigue has been performed at Lehigh University since the 1970's by Fisher (1981), who assisted in developing a fatigue control plan for the National Cooperative Highway Research Program (NCHRP) (FHWA 1978). Refinement of this plan was conducted over the course of the next twenty years, including retrofit of web gaps in 1985 and alterations to certain weld applications (FHWA 1978).

Geometric restrictions proposed in this research enabled subsequent bridges to be constructed without highly fatigue-susceptible details. The Minnesota Department of Transportation later adopted a plan to construct bridges with infinite fatigue life. Current research focuses on assessing stresses and sustainability of existing details. Work done by the University of Minnesota in 2000 and thereafter proposed and refined a method for approximating distortional stresses in cross-braces and diaphragms. These and other stress approximation methods could be used in the future, in conjunction with the American Association of State Highway and Transportation Officials (AASHTO) fatigue curve classification (Figure 2.1) to generate an approximate number of cycles to failure.

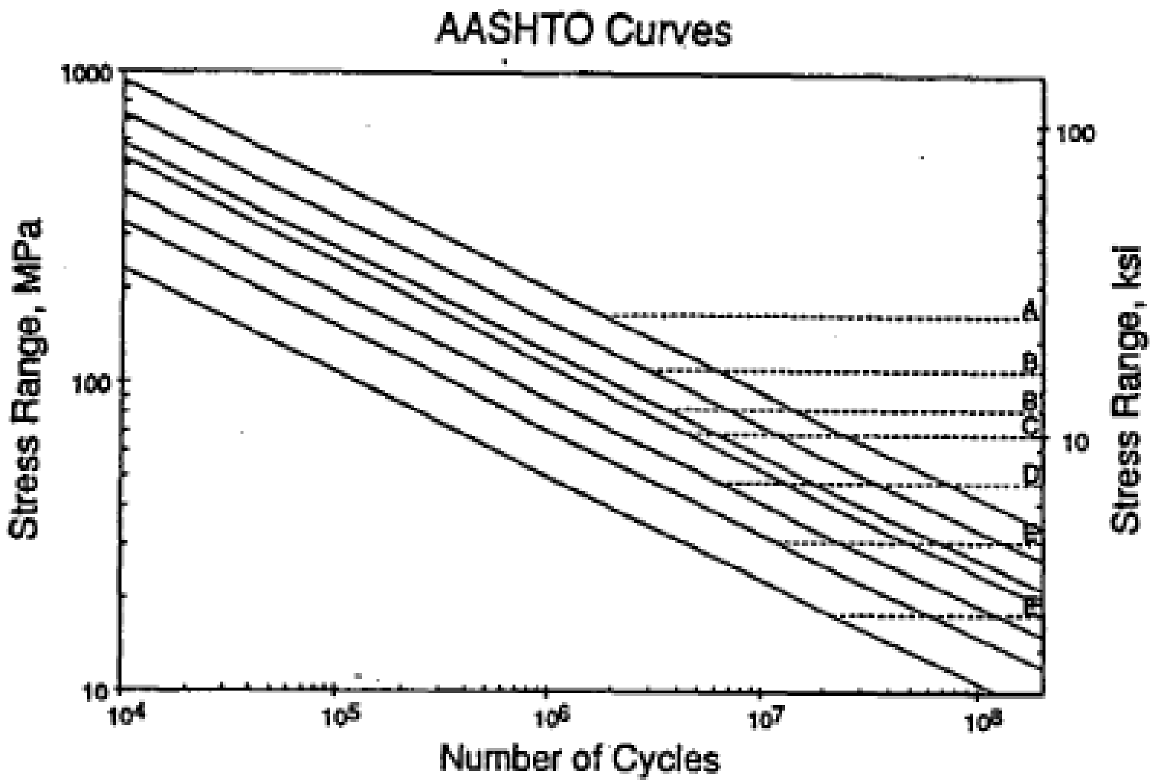


Figure 2.1 AASHTO *S-N* curves for detail classification A through E'

The AASHTO Fatigue Curves classify geometries of material and welds by A through E', in order of decreasing fatigue resistance. Use of the curves necessitates correct classification of each detail, as well as obtaining an appropriate stress range.

2.1 Identification of Details

Recognizing which details have fatigue issues is crucial to attempt classification. A fairly comprehensive list of details with their AASHTO *S-N* category curve can be found in many sources, including the Federal Highway Administration (FHWA) Bridge Inspector's Manual, or the American Institute of Steel Construction (AISC) Manual of Steel Construction, Load and Resistance Factor Design (LRFD). A summary of these is contained in section 0.

Differential displacement induced fatigue has caused more cracks in steel bridges then simple fatigue. As mentioned before, these "distortional fatigue" details have only been eliminated from design since 1985. Therefore, to anticipate the large number of distortional flaws, further research was performed at the University of Minnesota to develop approximate stress methods for these details. In doing so, this research has increased understanding of this problem sufficiently to allow distortional details to be rated with a similar confidence to non-distortional details. A basic overview of distortional fatigue is included in section 0.

Laboratory data on fatigue cracking does not cover all in-situ details due to the case-specific nature of the fatigue stress states, and current practices are derived usually from past experiences.

These, in majority, consist of in-situ bridge failures that have been analyzed post-failure. Section 0 outlines specific cases across the United States and Canada in which details have cracked.

2.1.1 AASHTO S-N Categories

The classification for rating which geometries are vulnerable to fatigue is found consistently in many sources. This section outlines some of the known problematic details including their classification category for the AASHTO curves. A complete classification summary for common geometries is given thereafter, in Tables 2.2 through 2.15.

Of the multitude of details present in bridges in the United States, some are more common than others. For example, eyebars and pin plates are rare details. These were discovered to be hazardous earlier than many details, and most have been replaced. On the other hand, fatigue sensitive cover plates and horizontal stiffener weld intersections are found in many bridges; only recently were they redesigned or eliminated from design.

Determination of which details to examine and which to ignore is an imperative portion of organizing critical details. Locating details with A or B classification on the AASHTO curves is unnecessary, since there are currently many C through F details present. Furthermore, sub-ranking rare details may prove wasteful, and coarse classification of the details could be more appropriate (Corwin 2002).

The most common susceptible detail is the flange cover plate used to increase moment capacity locally. Welds on partial length flange cover plates create huge stress concentrations that could develop cracks at the terminations of the weld returns or at the end of the cover plates. If the plate is wider than the flange *with* welds across ends, or the plate is narrower than the flange *with or without* welds across ends: then, if $t_f \leq 0.8in.$, then the category is E, or if $t_f > 0.8in.$, the he category is E'. Otherwise, the detail is E'. Placement of the termination within areas of high stress reversals was discouraged by AASHTO with the adoption of its fatigue design.

When horizontal connection plates for lateral bracing and vertical stiffeners intersect they are occasionally welded together. This, in turn, forms a built-in crack where butt-welds do not achieve full penetration (Figure 2.2). This is a common and critical detail that must be examined. Stress concentrations at the ends of horizontal connection plates, whether connected to the bottom of the web or the tension flange, are a category E detail if longer than 4 inches. They are classified by AASHTO as tension members even though they are secondary elements.

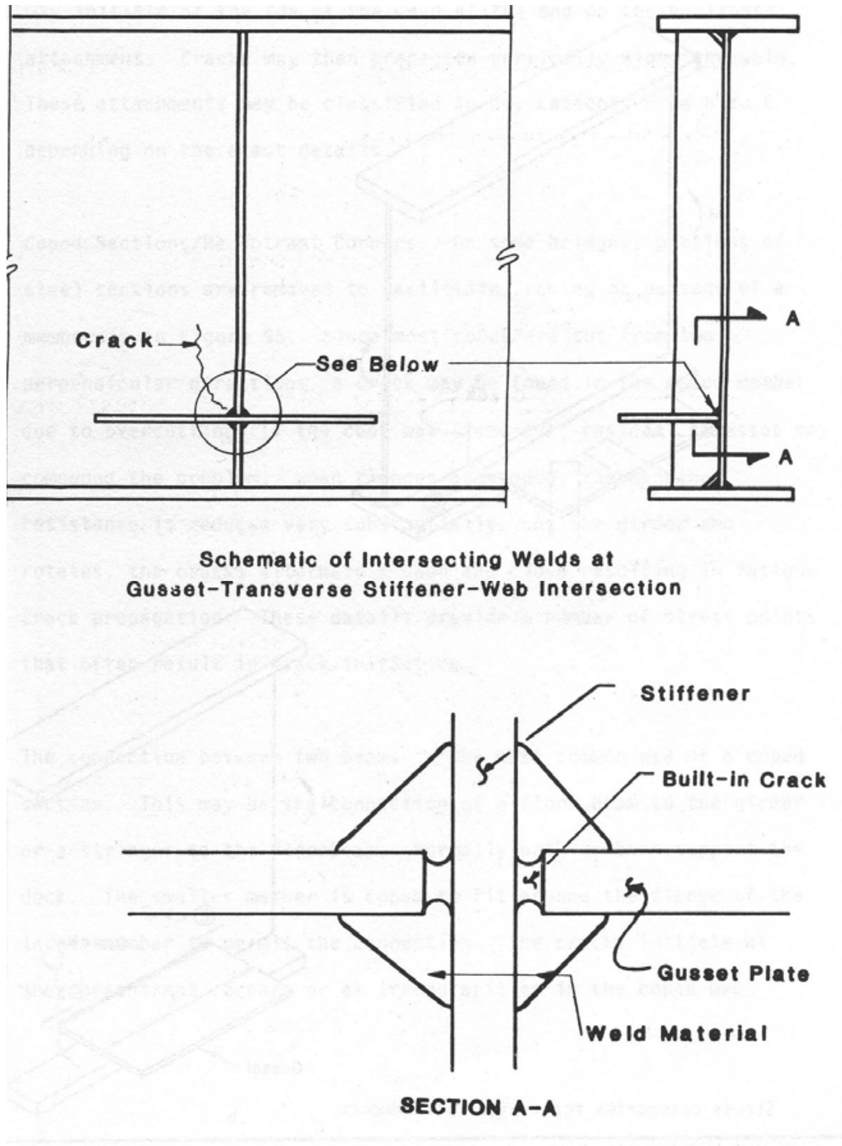


Figure 2.2 Intersecting stiffeners weld detail (Fisher 1981)

Overcut copes have a large increase in stress at the end of the cut. Any detail with this problem should be examined since it acts as an initial fissure.

Another common fatigue issue is the backing bar left in place after the completion of a groove weld. If left in place, a backing bar can create a stress concentration similar to a notch between the bar and the welded metal. Since the primary stress fluctuation is usually perpendicular to the bar, a fatigue crack can easily form. The current Bridge Welding Code, AWS D1.5 required the removal of backing bars, but structures built prior to 1994 did not require removal (FHWA 1986).

Other common bridge details are listed in Table 2.1, and detailed descriptions are provided in Tables 2.2 to 2.15.

Table 2.1 AASHTO *S-N* categories for common details

Description	*S-N* category
Riveted/mechanically fastened joints (non-high-strength bolts) in shear	D
Pin plates/eyebars (high stress on net section)	E
Terminations of longitudinal fillet welds	E
Attachments normal to flanges or plates, no significant load	
$L < 51mm$	C
$51mm \leq L \leq 101mm$	D
$101mm < L$	E
Edge Distance $< 10mm$ for all lengths	E
Attachment plates or flange thickness $> 25mm$ for all lengths	E'
Transverse stiffeners (like short attachments or cross-bracing or diaphragms)	C
Load-carrying attachments with two stress ranges (distortion included)	E
Attachments with rounded ends and fillet or groove welds ground smooth	
Transition radius $> 50mm$	D
Transition radius $> 152mm$	C

This list is by no means complete; issues may arise regarding other common practices that cannot be identified from plans. These include: notches, misalignment and other geometric discontinuities, thermal cutting, weld joint design, residual stress, weld defects, intersecting welds, and inadequate weld access holes. Lastly, if drawings are inconsistent with actual conditions, it could prove difficult to identify where problems may occur. Space can be made available for new problematic details noted by DOTs, from routine inspections. This data can be used for future updating of databases.

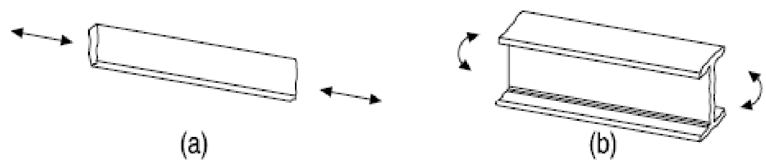
Table 2.2 AASHTO categories for plain material and mechanically fastened joints (AISC 2001)

Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 1 – PLAIN MATERIAL AWAY FROM ANY WELDING				
1.1 Base metal, except non-coated weathering steel, with rolled or cleaned surface. Flame-cut edges with surface roughness value of 1,000 μin. (25 μm) or less, but without re-entrant corners.	A	250 x 10^8	24 (165)	Away from all welds or structural connections
1.2 Non-coated weathering steel base metal with rolled or cleaned surface. Flame-cut edges with surface roughness value of 1,000 μin. (25 μm) or less, but without re-entrant corners.	B	120 x 10^8	16 (110)	Away from all welds or structural connections
1.3 Member with drilled or reamed holes. Member with re-entrant corners at copes, cuts, block-outs or other geometrical discontinuities made to requirements of Appendix K3.5, except weld access holes.	B	120 x 10^8	16 (110)	At any external edge or at hole perimeter
1.4 Rolled cross sections with weld access holes made to requirements of Section J1.6 and Appendix K3.5. Members with drilled or reamed holes containing bolts for attachment of light bracing where there is a small longitudinal component of brace force.	C	44 x 10^8	10 (69)	At re-entrant corner of weld access hole or at any small hole (may contain bolt for minor connections)
SECTION 2 – CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS				
2.1 Gross area of base metal in lap joints connected by high-strength bolts in joints satisfying all requirements for slip-critical connections.	B	120 x 10^8	16 (110)	Through gross section near hole
2.2 Base metal at net section of high-strength bolted joints, designed on the basis of bearing resistance, but fabricated and installed to all requirements for slip-critical connections.	B	120 x 10^8	16 (110)	In net section originating at side of hole
2.3 Base metal at the net section of other mechanically fastened joints except eye bars and pin plates.	D	22 x 10^8	7 (48)	In net section originating at side of hole
2.4 Base metal at net section of eyebar head or pin plate.	E	11 x 10^8	4.5 (31)	In net section originating at side of hole

Table 2.3 Diagrams for plain material and mechanically fastened joints (AISC 2001)

SECTION 1 - PLAIN MATERIAL AWAY FROM ANY WELDING

1.1 and 1.2



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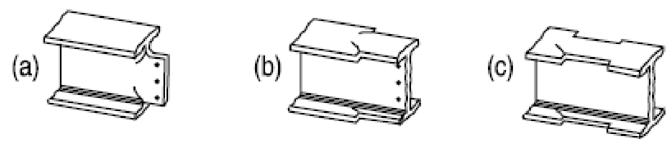

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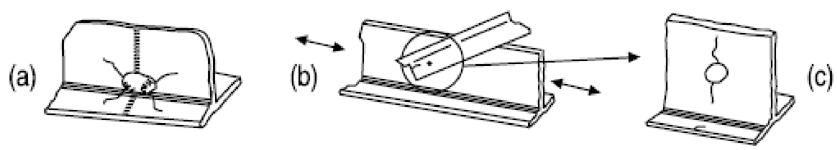
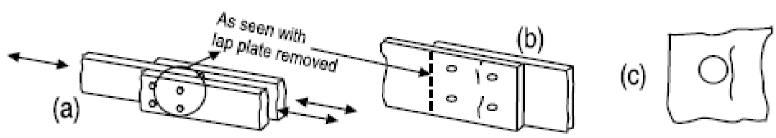


SECTION 2 - CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS

2.1



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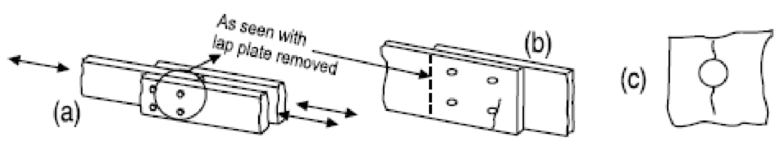

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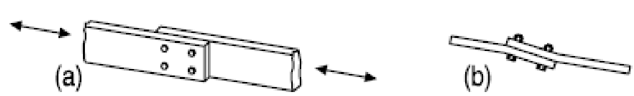
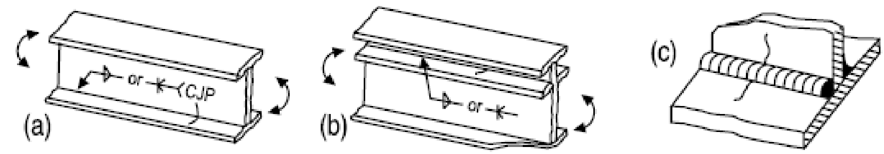
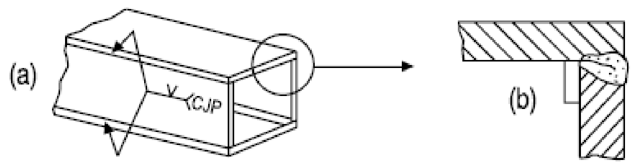


2.4

Table 2.4 AASHTO categories for welded joints in built-up members & longitudinal fillet welded ends (AISC 2001)

Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 3 – WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS				
3.1 Base metal and weld metal in members without attachments built-up of plates or shapes connected by continuous longitudinal complete-joint-penetration groove welds, back gouged and welded from second side, or by continuous fillet welds.	B	120×10^8	16 (110)	From surface or internal discontinuities in weld away from end of weld
3.2 Base metal and weld metal in members without attachments built-up of plates or shapes, connected by continuous longitudinal complete penetration groove welds with backing bars not removed, or by continuous partial-joint-penetration groove welds.	B′	61×10^8	12 (83)	From surface or internal discontinuities in weld, including weld attaching backing bars
3.3 Base metal and weld metal termination of longitudinal welds at weld access holes in connected built-up members.	D	22×10^8	7 (48)	From the weld termination into the web or flange
3.4 Base metal at ends of longitudinal intermittent fillet weld segments.	E	11×10^8	4.5 (31)	In connected material at start and stop locations of any weld deposit
3.5 Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends of coverplates wider than the flange with welds across the ends.				In flange at toe of end weld or in flange at termination of longitudinal weld or in edge of flange with wide coverplates
Flange thickness $\leq$ 0.8 in. (20 mm)	E	11×10^8	4.5 (31)	
Flange thickness > 0.8 in. (20 mm)	E′	3.9×10^8	2.6 (18)	
3.6 Base metal at ends of partial length welded coverplates wider than the flange without welds across the ends.	E′	3.9×10^8	2.6 (18)	In edge of flange at end of coverplate weld
SECTION 4 – LONGITUDINAL FILLET WELDED END CONNECTIONS				
4.1 Base metal at junction of axially loaded members with longitudinally welded end connections. Welds shall be on each side of the axis of the member to balance weld stresses.				Initiating from end of any weld termination extending into the base metal
$t \leq$ ½-in. (13 mm)	E	11×10^8	4.5 (31)	
$t >$ ½-in. (13 mm)	E′	3.9×10^8	2.6 (18)	

Table 2.1 Diagrams for welded joints in built-up members and longitudinal fillet welded ends (AISC 2001)

SECTION 3 - WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS
3.1 
3.2 
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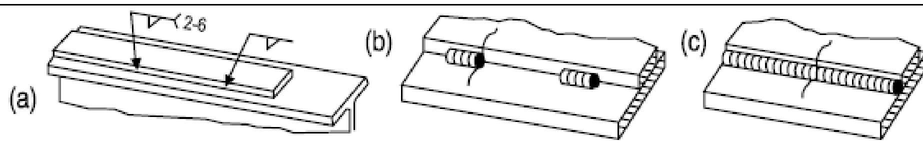
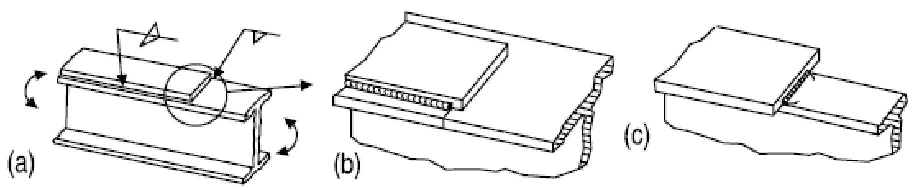
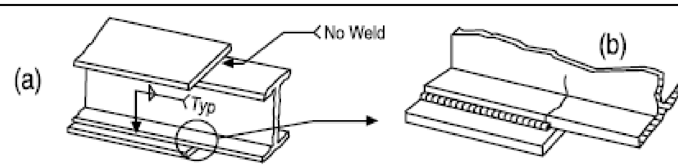
3.4 
3.5 
3.6 
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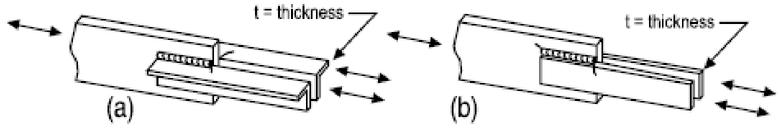
4.1 

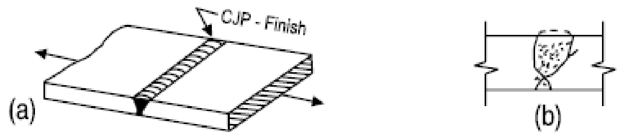
Table 2.2 AASHTO categories for welded joints transverse to direction of stress (AISC 2001)

Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS				
5.1 Base metal and weld metal in or adjacent to complete joint penetration groove welded splices in rolled or welded cross sections with welds ground essentially parallel to the direction of stress.	B	120×10^8	16 (110)	From internal discontinuities in filler metal or along the fusion boundary
5.2 Base metal and weld metal in or adjacent to complete joint penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in thickness or width made on a slope no greater than 8 to 20%.				From internal discontinuities in filler metal or along fusion boundary or at start of transition when $F_y \geq$ 90 ksi (620 MPa)
F_y < 90 ksi (620 MPa)	B	120×10^8	16 (110)	
$F_y \geq$ 90 ksi (620 MPa)	B′	61×10^8	12 (83)	
5.3 Base metal with F_y equal to or greater than 90 ksi (620 MPa) and weld metal in or adjacent to complete joint penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in width made on a radius of not less than 2 ft. (600 mm) with the point of tangency at the end of the groove weld.	B	120×10^8	16 (110)	From internal discontinuities in filler metal or discontinuities along the fusion boundary
5.4 Base metal and weld metal in or adjacent to the toe of complete joint penetration T or corner joints or splices, with or without transitions in thickness having slopes no greater than 8 to 20%, when weld reinforcement is not removed.	C	44×10^8	10 (69)	From surface discontinuity at toe of weld extending into base metal or along fusion boundary.
5.5 Base metal and weld metal at transverse end connections of tension-loaded plate elements using partial joint penetration butt or T or corner joints, with reinforcing or contouring fillets, F_{SR} shall be the smaller of the toe crack or root crack stress range.				Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at weld root subject to tension extending up and then out through weld
Crack initiating from weld toe:	C	44×10^8	10 (69)	
Crack initiating from weld root:	C′	Eqn. A-K3.3 or A-K3.3M	None provided	

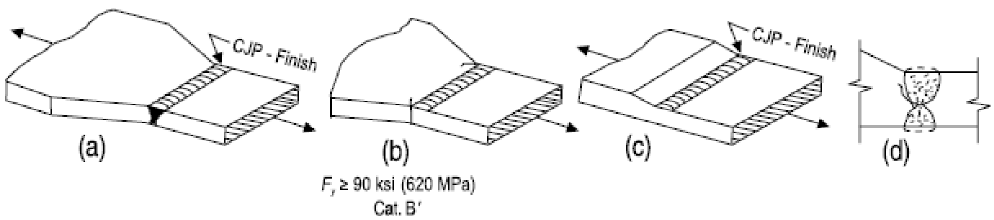
Table 2.3 Diagrams for welded joints transverse to direction of stress (AISC 2001)

SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS

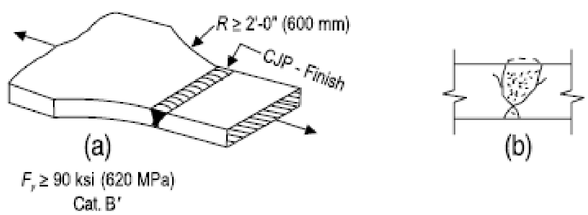
5.1



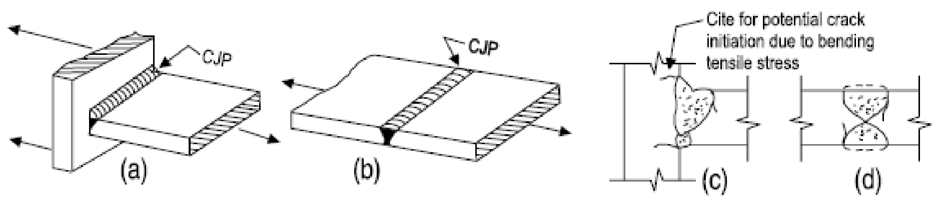
5.2



5.3



5.4



5.5

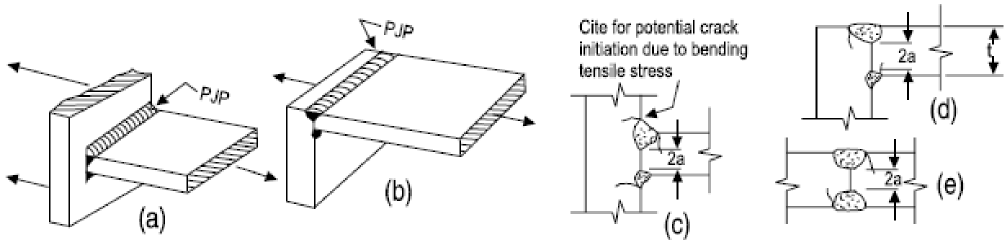


Table 2.4 AASHTO categories for welded joints transverse to direction of stress and base metal at welded transverse member connections (AISC 2001)

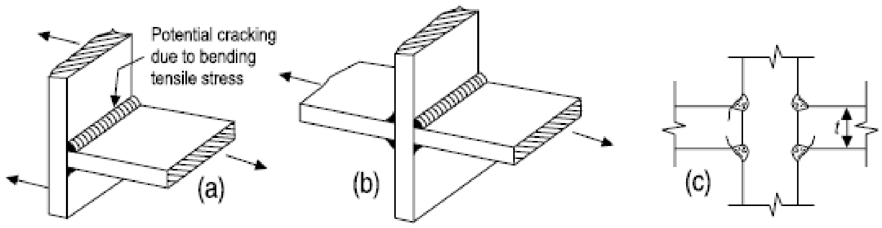
TABLE A-K3.1 (Cont'd)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)				
5.6 Base metal and filler metal at transverse end connections of tension-loaded plate elements using a pair of fillet welds on opposite sides of the plate. F_{SR} shall be the smaller of the toe crack or root crack stress range.				Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at weld root subject to tension extending up and then out through weld
Crack initiating from weld toe:	C	44 x 10^8	10 (69)	
Crack initiating from weld root:	C″	Eqn. A-K3.4 or A-K3.4M	None provided	
5.7 Base metal of tension loaded plate elements and on girders and rolled beam webs or flanges at toe of transverse fillet welds adjacent to welded transverse stiffeners.	C	44 x 10^8	10 (69)	From geometrical discontinuity at toe of fillet extending into base metal
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS				
6.1 Base metal at details attached by complete joint penetration groove welds subject to longitudinal loading only when the detail embodies a transition radius R with the weld termination ground smooth.				Near point of tangency of radius at edge of member
$R \geq$ 24 in. (600 mm)	B	120×10^8	16 (110)	
24 in. > $R \geq$ 6 in. (600 mm > $R \geq$ 150 mm)	C	44×10^8	10 (69)	
6 in. > $R \geq$ 2 in. (150 mm > $R \geq$ 50 mm)	D	22×10^8	7 (48)	
2 in. (50 mm) > R	E	11×10^8	4.5 (31)	

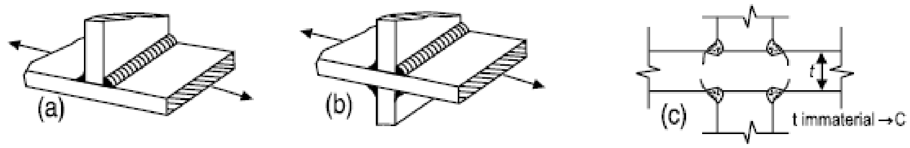
Table 2.5 Diagrams for welded joints transverse to direction of stress and base metal at welded transverse member connections (AISC 2001)

SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)

5.6



5.7



SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS

6.1

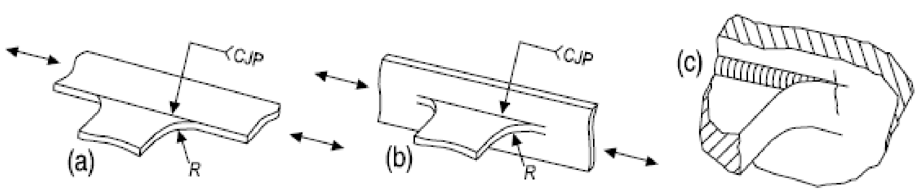


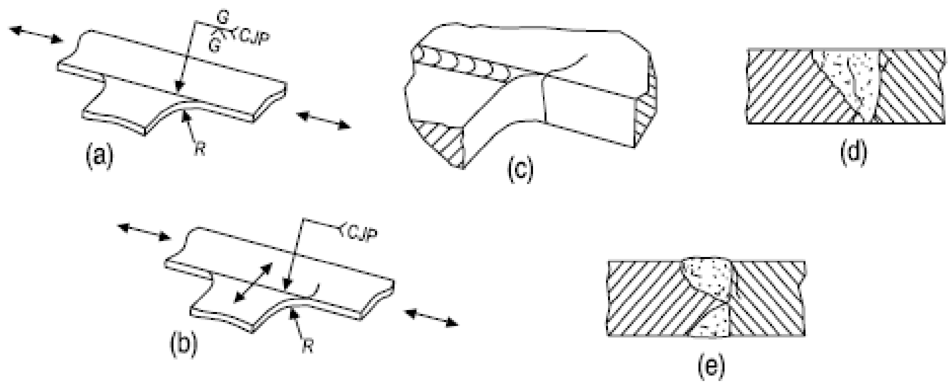
Table 2.6 AASHTO categories for base metal at welded transverse member connections (AISC 2001)

Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)				
6.2 Base metal at details of equal thickness attached by complete joint penetration groove welds subject to transverse loading with or without longitudinal loading when the detail embodies a transition radius R with the weld termination ground smooth:				
When weld reinforcement is removed:				
$R \geq$ 24 in. (600 mm)	B	120×10^8	16 (110)	Near points of tangency of radius or in the weld or at fusion boundary or member or attachment
24 in. > $R \geq$ 6 in. (600 mm > 150 mm)	C	44×10^8	10 (69)	
6 in. > $R \geq$ 2 in. (150 mm > $R \geq$ 50 mm)	D	22×10^8	7 (48)	
2 in. (50 mm) > R	E	11×10^8	4.5 (31)	
When weld reinforcement is not removed:				At toe of the weld either along edge of member or the attachment
$R \geq$ 24 in. (600 mm)	C	44×10^8	10 (69)	
24 in. > $R \geq$ 6 in. (600 mm > $R \geq$ 150 mm)	C	44×10^8	10 (69)	
6 in. > $R \geq$ 2 in. (150 mm > $R \geq$ 50 mm)	D	22×10^8	7 (48)	
2 in. (50 mm) > R	E	11×10^8	4.5 (31)	
6.3 Base metal at details of unequal thickness attached by complete joint penetration groove welds subject to transverse loading with or without longitudinal loading when the detail embodies a transition radius R with the weld termination ground smooth.				
When weld reinforcement is removed:				
R > 2 in. (50 mm)	D	22×10^8	7 (48)	At toe of weld along edge of thinner material
$R \leq$ 2 in. (50 mm)	E	11×10^8	4.5 (31)	In weld termination in small radius
When reinforcement is not removed:				At toe of weld along edge of thinner material
Any radius	E	11×10^8	4.5 (31)	

Table 2.7 Diagrams for base metal at welded transverse member connections (AISC 2001)

SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)

6.2



6.3

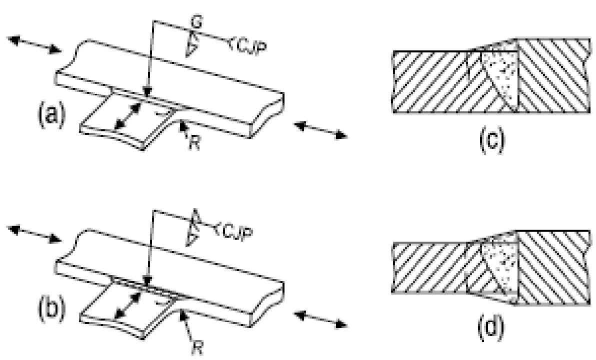
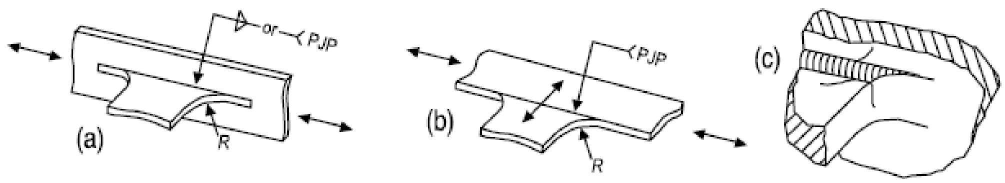


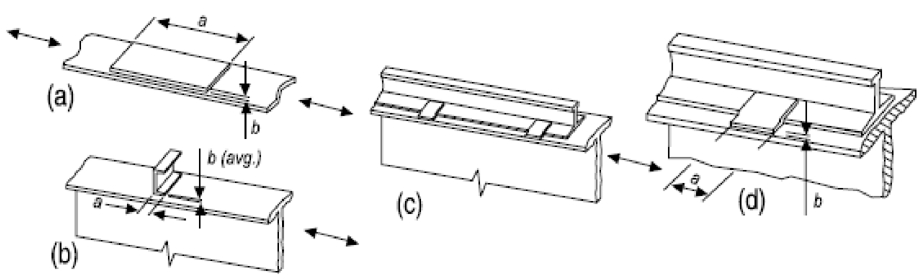
Table 2.8 AASHTO categories for base metal at welded transverse member connections and base metal at short attachments (AISC 2001)

Description	Stress Category	Constant C_f	Threshold F_{TH} Ksi (MPa)	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)				
6.4 Base metal subject to longitudinal stress at transverse members, with or without transverse stress, attached by fillet or partial penetration groove welds parallel to direction of stress when the detail embodies a transition radius, R, with weld termination ground smooth:				In weld termination or from the toe of the weld extending into member
$R > 2$ in. (50 mm)	D	22×10^8	7 (48)	
$R \leq 2$ in. (50 mm)	E	11×10^8	4.5 (31)	
SECTION 7 – BASE METAL AT SHORT ATTACHMENTS[1]				
7.1 Base metal subject to longitudinal loading at details attached by complete penetration groove welds parallel to direction of stress where the detail embodies a transition radius, R, less than 2 in. (50 mm), and with detail length in direction of stress, a, and attachment height normal to surface of member, b:				In the member at the end of the weld
a < 2 in. (50 mm)	C	44×10^8	10 (69)	
2 in. (50 mm) $\leq a \leq 12b$ or 4 in (100 mm)	D	22×10^8	7 (48)	
a > 12b or 4 in. (100 mm) when b is $\leq$ 1 in. (25 mm)	E	11×10^8	4.5 (31)	
a > 12b or 4 in. (100 mm) when b is > 1 in. (25 mm)	E′	3.9×10^8	2.6 (18)	
7.2 Base metal subject to longitudinal stress at details attached by fillet or partial joint penetration groove welds, with or without transverse load on detail, when the detail embodies a transition radius, R, with weld termination ground smooth:				In weld termination extending into member
R > 2 in. (50 mm)	D	22×10^8	7 (48)	
R $\leq$ 2 in. (50 mm)	E	11×10^8	4.5 (31)	

[1] "Attachment" as used herein, is defined as any steel detail welded to a member which, by its mere presence and independent of its loading, causes a discontinuity in the stress flow in the member and thus reduces the fatigue resistance.

Table 2.9 Diagrams for base metal at welded transverse member connections and base metal at short attachments (AISC 2001)

SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)
6.4 

SECTION 7 – BASE METAL AT SHORT ATTACHMENTS[1]
7.1 

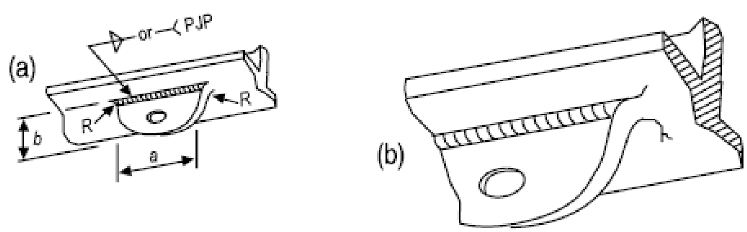
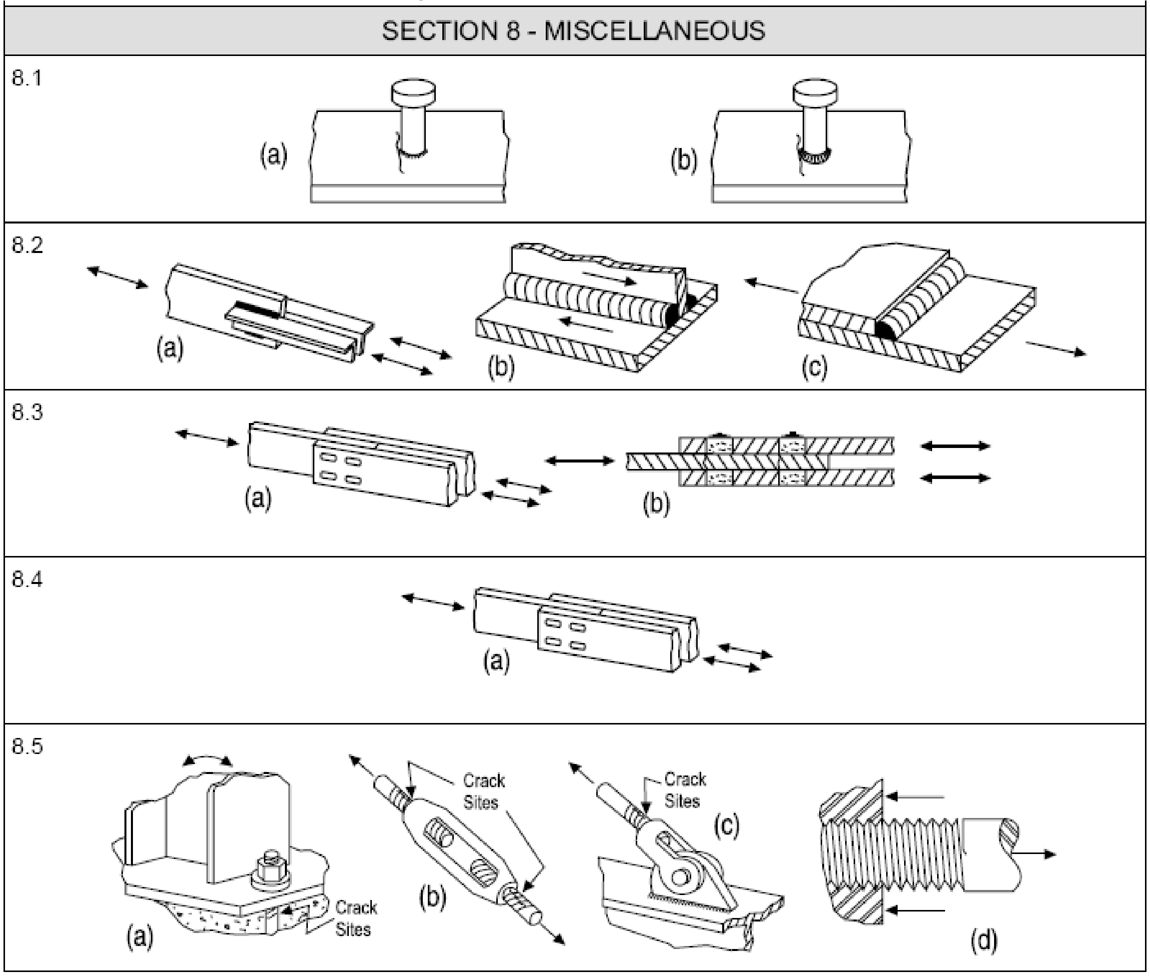
7.2 

Table 2.10 AASHTO categories for miscellaneous details (AISC 2001)

Description	Stress Category	Constant C_f	Threshold F_{TH} Ksi (MPa)	Potential Crack Initiation Point
SECTION 8 - MISCELLANEOUS				
8.1 Base metal at stud-type shear connectors attached by fillet or electric stud welding.	C	44×10^8	10 (69)	At toe of weld in base metal
8.2 Shear on throat of continuous or intermittent longitudinal or transverse fillet welds.	F	150×10^{10} (Eqn. A-K3.2 or A-K3.2M)	8 (55)	In throat of weld
8.3 Base metal at plug or slot welds.	E	11×10^8	4.5 (31)	At end of weld in base metal
8.4 Shear on plug or slot welds.	F	150×10^{10} (Eqn. A-K3.2 or A-K3.2M)	8 (55)	At faying surface
8.5 Not fully-tightened high-strength bolts, common bolts, threaded anchor rods and hanger rods with cut, ground or rolled threads. Stress range on tensile stress area due to live load plus prying action when applicable.	E′	3.9×10^8	7 (48)	At the root of the threads extending into the tensile stress area

Table 2.11 Diagrams for miscellaneous details (AISC 2001)



2.1.2 Distortional Fatigue Introduction

Distortional fatigue encompasses many distinct details, as seen in section 0. The premise is simple, rather than stress fluctuations developed directly from traffic, the stresses are generated by deformation, usually in the transverse direction. Researchers at the University of Minnesota examined the effects of one specific distortional detail: web gap rotation. "Up to 50% of all bridges with [the web gap] detail have fatigue cracks." (Fisher 1981)

The principle is as follows: as a vehicle travels over a bridge, loading may be concentrated on one of the girders, due to tire position. Or, if the girders are on skew supports, the distances from the location of the load to the supports will differ for adjacent girders. These loading situations cause differential deflection between adjacent girders. Braces or diaphragms connecting the girders rotate (Figure 2.3 & Figure 2.4), thus rotating the stiffeners they are connected to. The transverse stiffener and the girder web are stiff in comparison to the web alone when bent about their strong axis (Figure 2.5). On most bridges built before 1985, a gap, roughly two inches long, was left between the stiffener and the bottom girder flange to avoid welding to a tension flange. The web gap region absorbs the deformation and bends around its weak axis. Large stresses are generated in the thin web making them vulnerable to fatigue cracking (Figure 2.5).

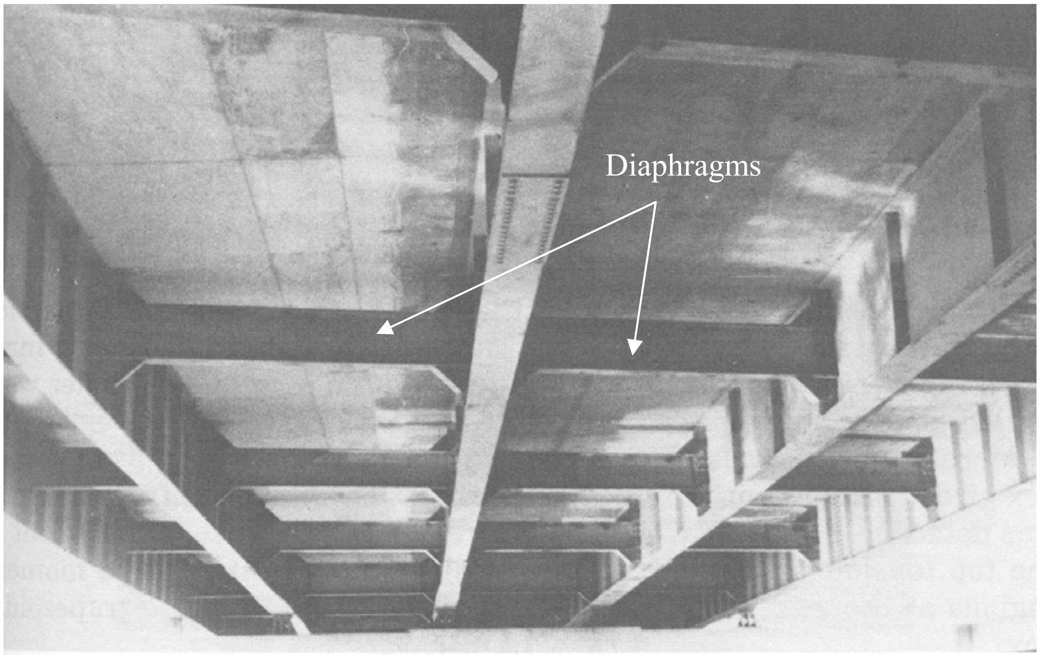


Figure 2.3 Diaphragms and transverse stiffeners on steel girder bridge (Fisher 1984)

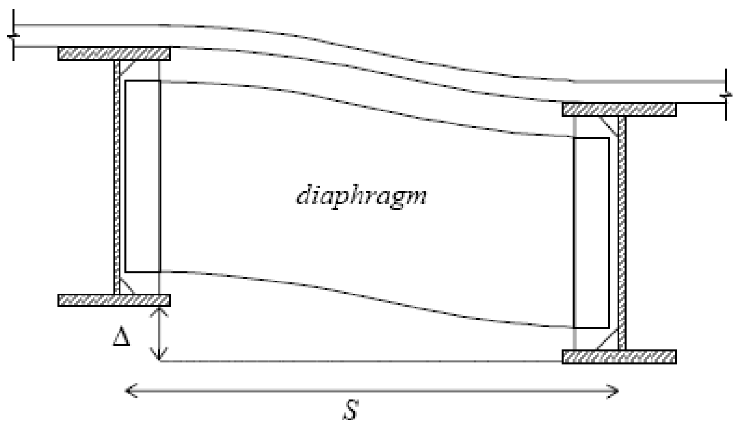


Figure 2.4 Differential girder displacement resulting in web rotations (Li, 2005)

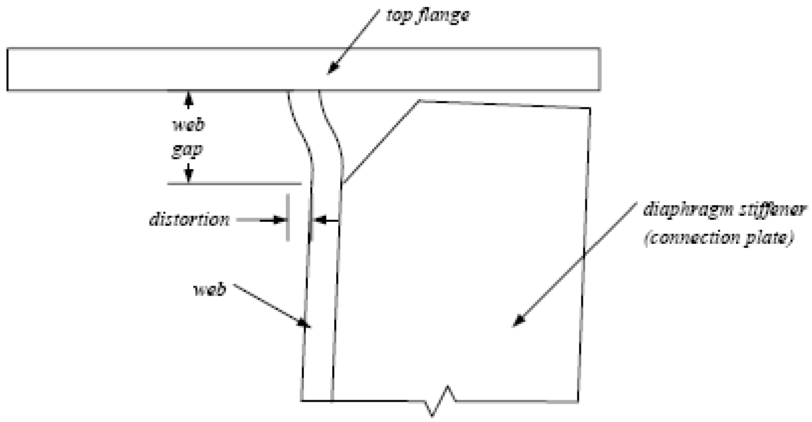


Figure 2.5 Relationship between variables for differential deflection and web gap distortion (Li, 2005)

After this problem was discovered, web gaps were eliminated from design by welding stiffeners directly to the top flange around 1985 (Jajich, 2000). Stresses can be approximated at the web gaps and those formulas are found in the stress estimation section, 0. Case studies for this and other details are found in section 0.

2.1.3 Case Studies

To determine which details may be susceptible to fatigue cracking, an in-depth investigation of past performance must be conducted. Throughout the United States cracks have affected a diverse set of details, which made transportation officials aware of fatigue vulnerability and led to exploration into their causes. Past examples are therefore a key mode for identifying problems that will reoccur in similar circumstances. This section outlines selected case studies of cracking; including a short description of the detail, cracking method, and an estimation of stress range and loading cycles before failure. For the most part, this section was derived from *Fatigue and Fracture in Steel Bridges* by Fisher (1984).

Fisher and his research team at Lehigh University have performed some of the most comprehensive work on fatigue and fracture to date. His book (Fisher 1984) provides much insight into the mechanisms of cracking and connects real examples with these mechanisms to derive numerical methods to explain crack growth. Many different factors contribute to cracking, including: traffic distributions, temperature conditions, welding flaws, and material properties.

2.1.3.1 Transverse Stiffener Web Gap

Both the I-90 Bridge over the Conrail yard in Cleveland and the I-480 Cuyahoga River Bridge near Independence, Ohio have approximately one inch spacing between transverse stiffeners and the bottom flange of girders. The I-90 Bridge has X-shape diaphragms and the I-480 Bridge has horizontal diaphragms with truss members connecting their midpoints to transverse stiffeners. Cracking occurred in both circumstances; below the transverse stiffeners as seen in Figure 2.6.

The cracks began either in the stiffener welds, directly below the stiffeners, or at the toe of the stiffener welds. (Fisher 1984)

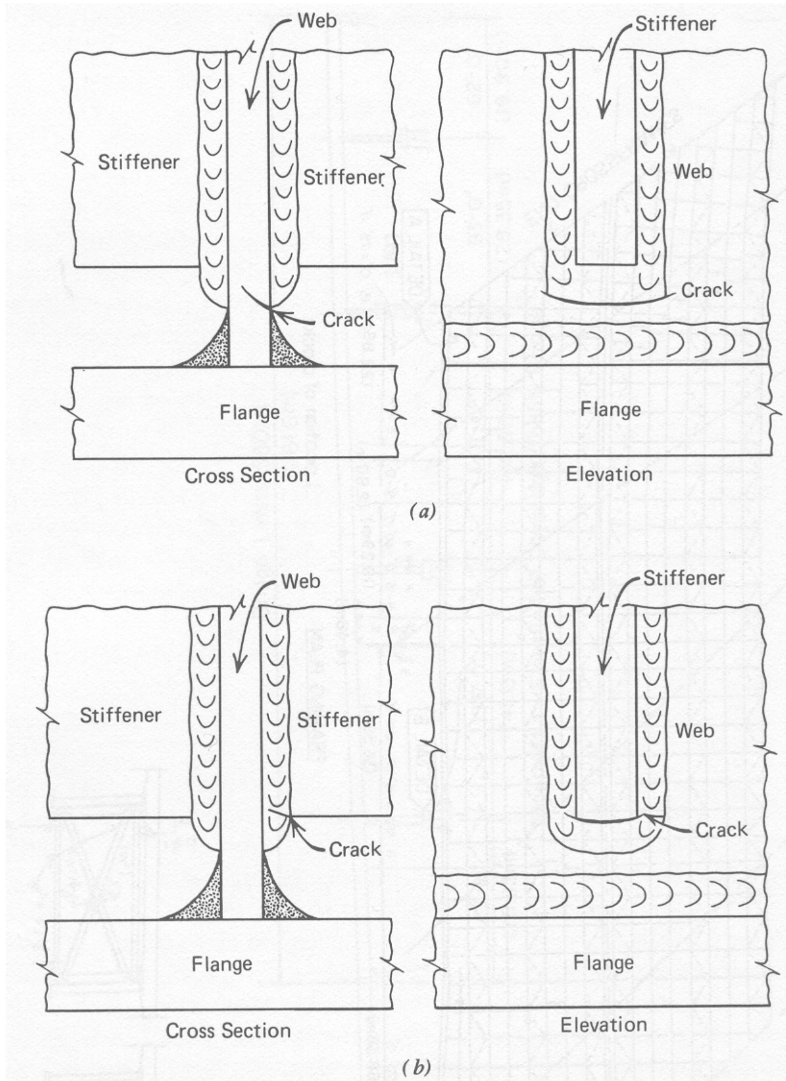


Figure 2.6 Transverse stiffener web gap cracks on the I-90 Bridge (Fisher 1984)

In the case of the I-90 Bridge, cracks were discovered during erection, and the cause was determined to be cyclic loading during transportation. The only members that were cracked were those that were carried by train, and the only details affected were near wooden supports during transportation. The members were presumed to have swayed during transportation, approximately 50,000 cycles, and the reversed bending stress range was nearly 46 ksi. It was discovered that the I-480 Bridge also cracked in transit, from approximately 88 ksi of stress during 7300 stress cycles. (Fisher 1984)

2.1.3.2 Insufficient Cope Radius

Cracking has occurred in many instances in coped members spanning between girders. An example of this is Canadian Pacific Railroad Bridge No. 51.5 near Ottawa, Ontario. The flame cut edge at the beam cope caused a stress concentration which initiated cracking at the cope corner (Figure 2.7). The traffic on this bridge was mostly coal trains, leading to an estimated effective stress range of 18 ksi. An estimated 1.3 million cycles happened before cracking was discovered. (Fisher 1984)

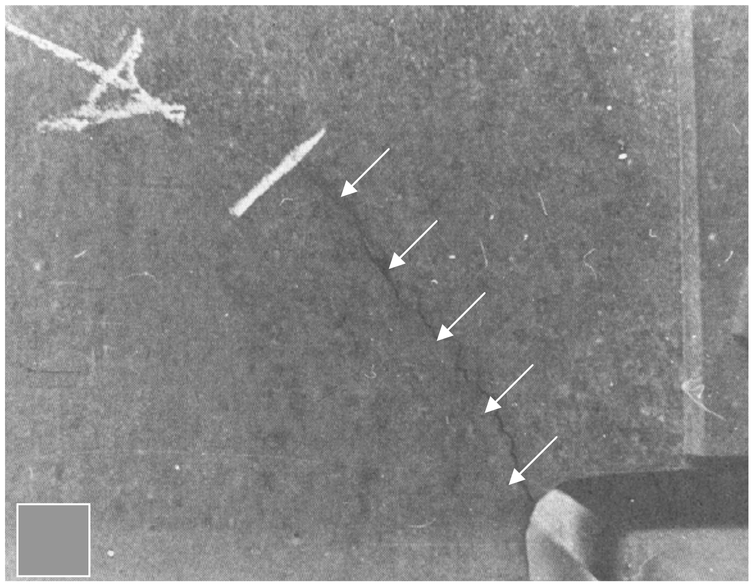


Figure 2.7 Crack originating from the coped stringer of Hwy 51.5 Bridge (Adapted from Fisher 1984)

In coping, since the flanges were removed from the beam, the bending resistance was reduced by 80%-90%. Approximately 1 million stress cycles would propagate a 0.015 in. edge crack to 4 in. Due to flame cutting, residual stresses cause initial cracking. (Fisher 1984)

2.1.3.3 Partial Length Cover Plate

The first example of this deficiency was discovered on Yellow Mill Pond Bridge in Connecticut. Cracking occurred at the ends of cover plates located on the bottom flange of girders (Figure 2.8). The cracks began at the toe of the weld and penetrated through the flange, severing it in two. It proceeded 7 in. up the web before termination (Figure 2.9). (Fisher 1984)

Figure 2.8 Typical cracking at weld toe of cover plate (Adapted from Fisher 1984)

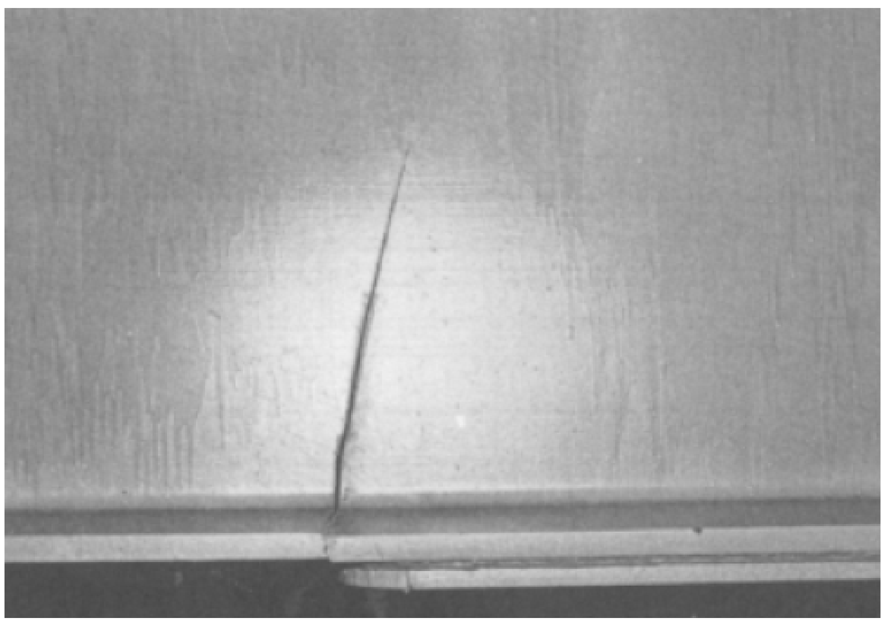


Figure 2.9 Crack at end of cover plate of Yellow Mill Pond Bridge propagating into web (Fisher 1984)

The cracking of the girder flange was completely fatigue related, and was due to the large stress concentration at the end of cover plate weld. Under extreme load condition the maximum stress intensity from tests ranged from 1.1 ksi to 1.98 ksi. The estimated number of cycles to grow the crack to a depth of 1 in. was 36 million. This type of cracking is the most common among bridges today. (Fisher 1984)

2.1.3.4 Shelf Plate Welded to Girder Web

The primary example of web connection plate fracture is the Lafayette Bridge over the Mississippi River in St. Paul. Cracking began from a poor groove weld between a horizontal connection plate and transverse stiffener. The crack was transferred through the connection plate into the web of the girder (Figure 2.10). From there it moved up the web as well as down, severing the bottom flange. The crack was discovered only 7.5 in. from the top flange. (Fisher 1984)

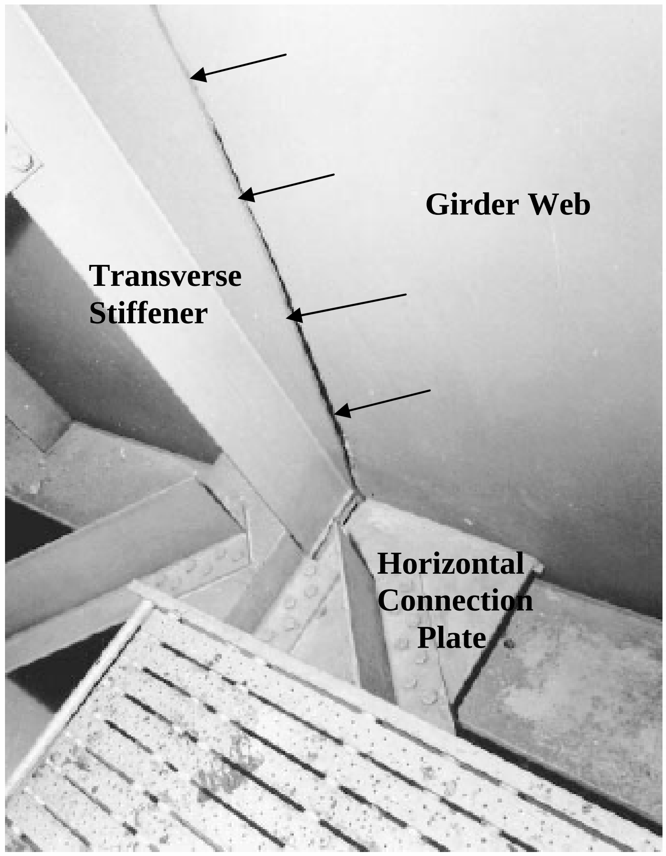


Figure 2.10 Cracked girder of Lafayette Street Bridge (Adapted from Fisher 1984)

The web connection plate issue lay with the occurrence of intersecting welds where the transverse stiffener met the lateral connection plate. Primarily, there was a defect in the weld between the stiffener and plate and since the weld was perpendicular to the maximum stress range, cracking occurred. This type of cracking was found on at least two other bridges with similar connections. An estimated 3.19 million cycles and a stress range of 4.68 ksi in the flange and 4.13 ksi in the gusset-web connection occurred before the crack was discovered. (Fisher 1984)

2.1.3.5 Stringer or Truss Floor-beam Bracket

For stringer to floor beam bracket distress, there is not a unique example. Cracking has occurred on at least three truss structures and in the stringers of two suspension bridges. Cracks are either located in the flange/web fillet or in the bracket itself (Figure 2.11). Cracking only occurred over the road expansion joints where stringers were discontinuous. (Fisher 1984)

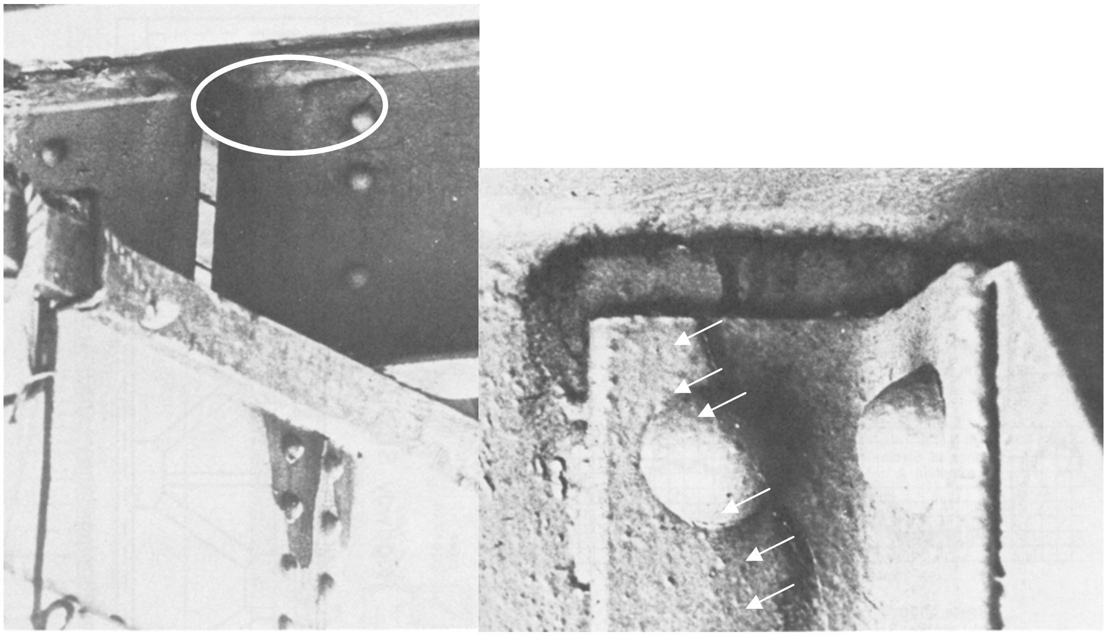


Figure 2.11 Stringer web cracks above bracket at roadway relief joint of the Walt Whitman Suspension Bridge (Adapted from Fisher 1984)

Movement at stringer ends led to out-of-plane bending of beams and connection angles. Approximately 19-35 ksi maximum stress range occurred over the web gap at relief joints. An estimate of 3.6 million trucks with three or more axles crossed the bridge before cracking was discovered. (Fisher 1984)

2.1.3.6 Welded Horizontal Stiffener

A transverse groove weld detail found at the center span of the Quinnipiac River Bridge in Connecticut resulted in a crack spreading through the bottom one-half of the girder web (Figure 2.12). This crack began due to insufficient welding between a horizontal stiffener and the girder web, which severed the stiffener before traveling into the web and bottom flange of the girder. Brittle fracture occurred through the majority of the web, but all other cracking was fatigue. It traveled 7in. up the web and cracked 65% of bottom flange. (Fisher 1984)

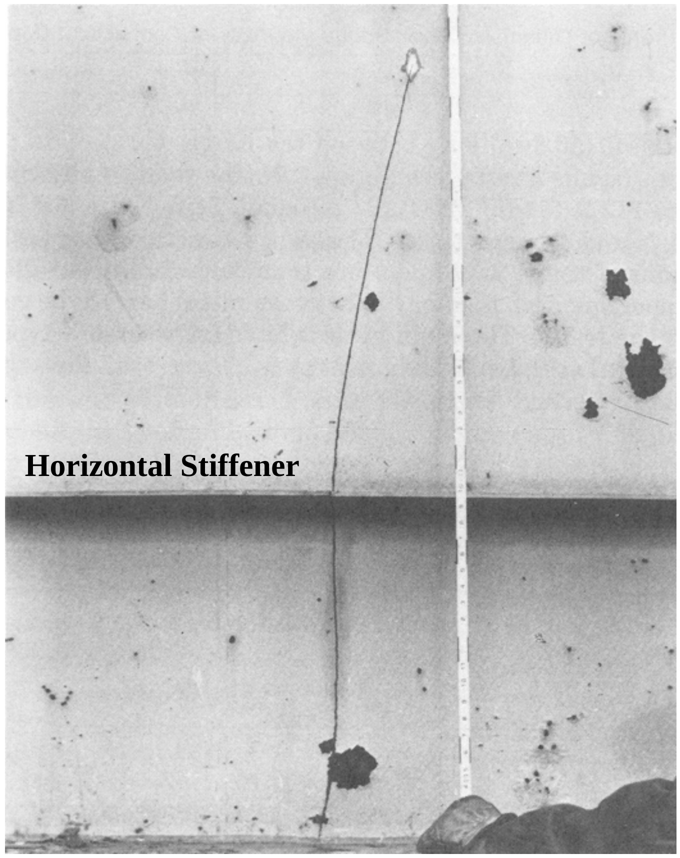


Figure 2.12 Crack in girder web of Quinnipiac River Bridge (Adapted from Fisher 1984)

A stress of 1.92 ksi and 1.5 million cycles per year were estimated for this bridge, but these numbers vary significantly depending upon the initial crack. Consequently, it would take about 2 years to crack the bottom flange and 15 to 20 years to propagate through the entire flange. Over the 9.5 years of service life before discovery, an estimated 14.5 million trucks passed at random over the bridge, which produced 30 million stress cycles. (Fisher 1984)

2.1.3.7 Haunch Insert

The first of three different types of transverse groove cracking details is at the end of haunch inserts. Cracking of the vertical butt-welds on the Aquasabon River Bridge, East of Thunder Bay is shown in Figure 2.13. The bottom flanges of the main girders were cut away to allow attachment of curved haunches at the piers. Because the joint was located at the point of moment reversal for dead load, the cracks fatigued heavily but no fracture occurred. (Fisher 1984)

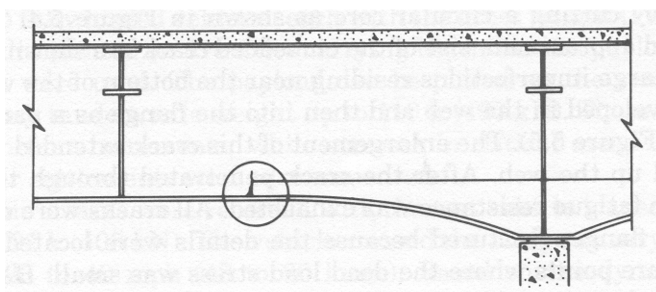


Figure 2.13 Cracked end of haunch insert on Aquasabon River Bridge (Fisher 1984)

Cracking was determined to have begun in the poor-quality transverse groove welds and propagated through the butt welds. The estimated effective stress range was 1.92 ksi at 1.5 million cycles per year. It would take 2 to 3 years to penetrate the bottom flange and 15 to 20 to severe it. (Fisher 1984)

2.1.3.8 Web Penetration

The Dan Ryan elevated train structure in Chicago contains many box girder bents along its length. A few of these experienced cracking at the point where the main girders penetrated through these box girders (Figure 2.14). The box girder was flame cut, and then welded to the main girder creating high stress in the welds adjacent to the stringer flange. This crack penetrated through the box girder webs and through its bottom flange. (Fisher 1984)

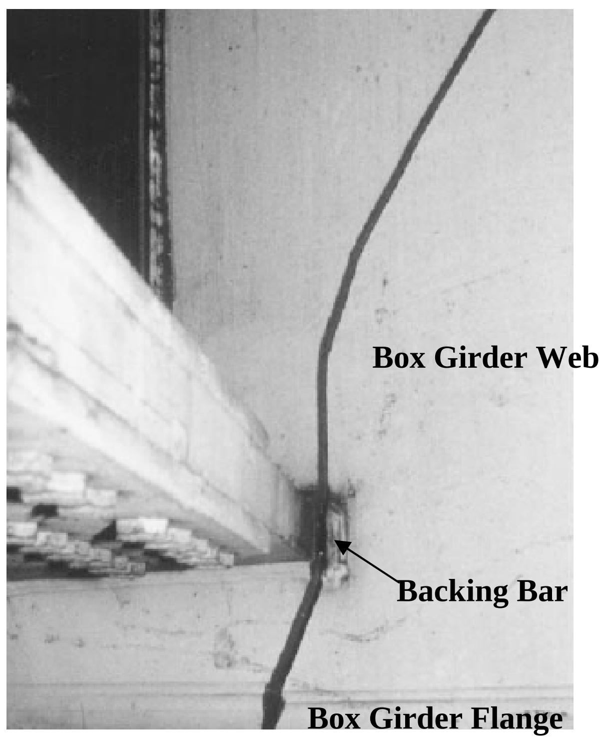


Figure 2.14 Crack through webs and flange of Dan Ryan Rapid Transit Structure (Adapted from Fisher 1984)

This cracking was induced by backing bars that were left around the penetration when it was groove welded. Examination of the weld found it to be poorly created, with large discontinuities. Train traffic produced an estimated 1.3 million cycles of 2.2 ksi to 3 ksi stress ranges, before fatigue cracks initiated. (Fisher 1984)

2.1.3.9 Cantilever Floor-beam Bracket

Cracking of cantilever floor-beam brackets such as those on the Allegheny River Bridge and the Lehigh River Bridge are common. Cracking occurred in tie plates that were sometimes used to connect the top flange of floor beams over main girders to another set of floor beams. Cracks began at tack welds used to attach the floor beams before they were riveted in place (Figure 2.15 & Figure 2.16). Due to deflections of the floor beams, a rotation occurred about the weak axis of the girders, causing bending at the girder interface. (Fisher 1984)

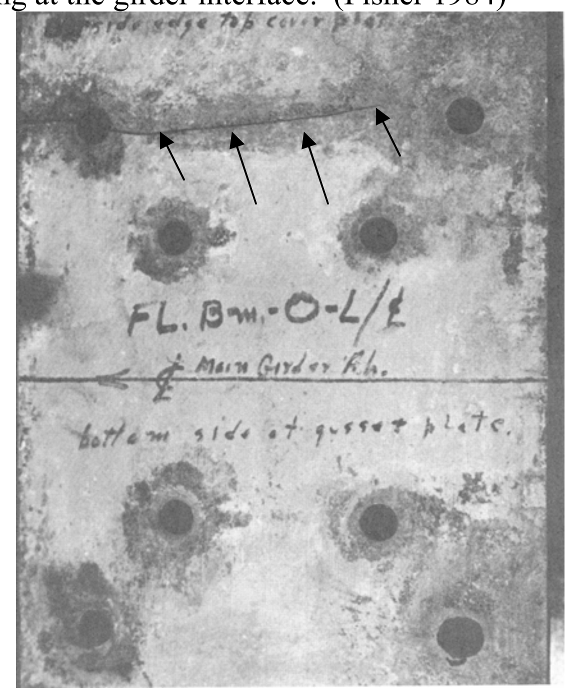


Figure 2.15 Crack through tie plate of Allegheny River Bridge (Adapted from Fisher 1984)

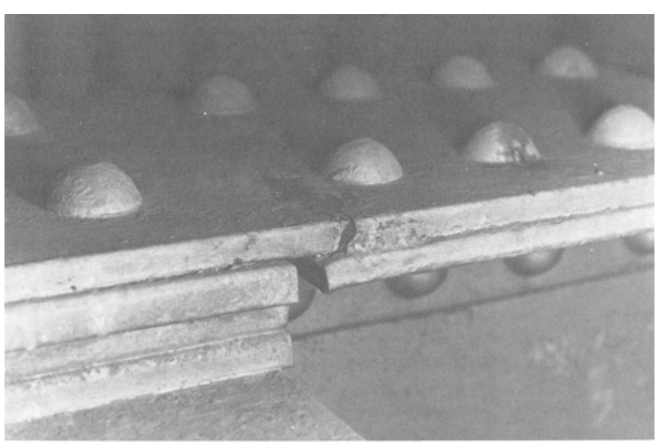


Figure 2.16 Crack originating at tack weld through tie plate of Lehigh River Bridge (Fisher 1984)

Cracking was substantial at the points in the span where largest deformations occurred. Calculations approximate that it would take 1.9 million cycles at 11.6 ksi or 14 million cycles at 6 ksi to begin cracking the floor beam brackets. Sometimes rivets cracked and stress was relieved, stopping crack growth. (Fisher 1984)

2.1.3.10 Tied Arch Floor-beam

The Prairie Du Chien Bridge between Wisconsin and Iowa contained floor beam to tied arch girder connections that were designed to carry the vertical load transferred from the beams and no bending moment. Namely, these were bolted/welded shear plate end connections. Cracks occurred along the weld between the beam flange and web and at the ends of the welds between the plate and girder (Figure 2.17). (Fisher 1984)

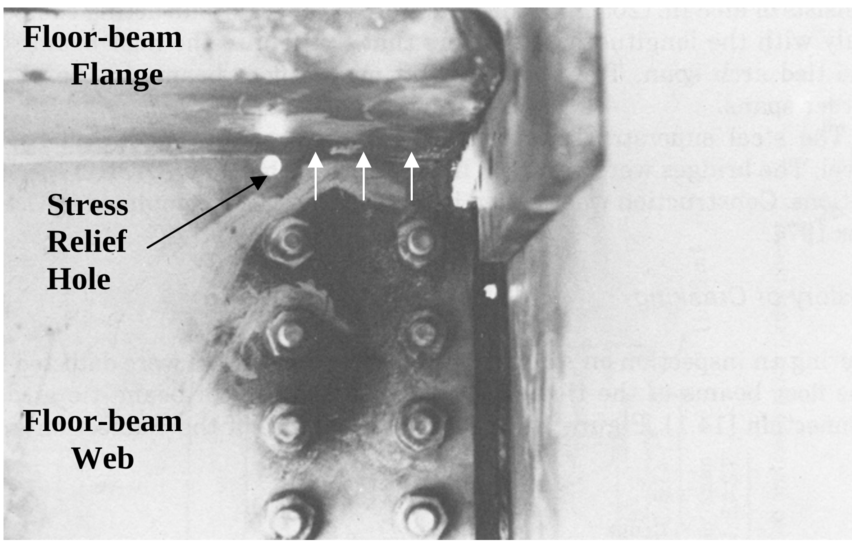


Figure 2.17 Cracking of floor-beam and floor-beam connection plate (Adapted from Fisher 1984)

The cause for cracking was movement of the joints due to unexpected rotation. None of the cracks appeared to be affected by temperature, which is unusual for many of the fatigue details. It was approximated that only 1 million stress cycles resulted in the majority of cracking. From calculations, this results in a stress differential of about 4.5 ksi. (Fisher 1984)

2.1.3.11 Continuous Longitudinal Weld: Box Girder Corner

The Gulf Outlet Bridge is a three-span truss bridge with a tied arch suspended span. Its box girders were welded longitudinally on the corners, connecting flanges to webs. Hydrogen-related cold cracking occurred in the welds at the time of fabrication (Figure 2.18). These cracks were undetectable to the naked eye; and only after grinding the surface were cracks detectable. Cracks occurred where extra weld thickness had been added manually to shop welds to increase strength in certain areas (Fisher 1984).

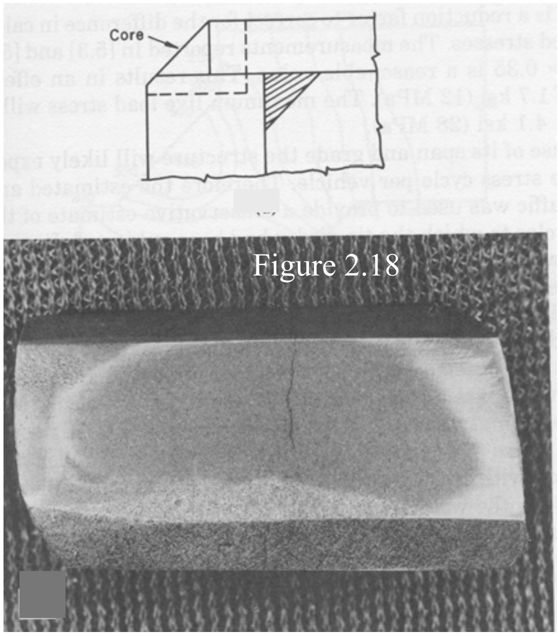


Figure 2.18 Core of cracked section of corner weld from Gulf Outlet Bridge box girder (Fisher 1984)

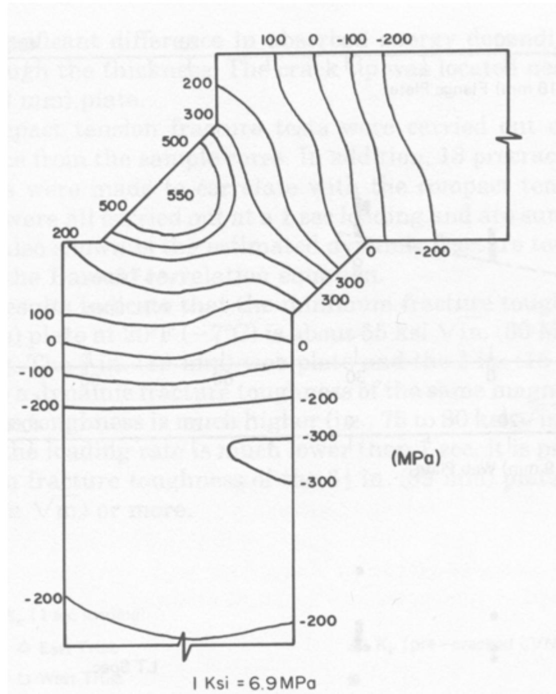


Figure 2.19 Stress concentration diagram for corner weld from residual heating stresses (Fisher 1984)

Crack growth occurred likely only in the largest cracks, with a growth stress threshold of 3.6 ksi. From calculations, if the effective range was 1.7 ksi, 4.8 million stress cycles would be necessary to propagate the crack a mere 0.005 in. Therefore, any significant cracking would require a larger stress range (Fisher 1984).

2.1.3.12 Cantilever: Lamellar Tear

Lamellar tearing is a failure mode that only occurs in highly restrained connections as in the case of the I-275 Bridge in Kenton County, Kentucky. This bridge has extended box girders that support one side of the road by cantilever (Figure 2.20). Cracks occurred at the top and bottom of the joint for this cantilever. The cracks occurred parallel to the grain of the steel and delaminated plies from each other (Figure 2.21) as the cantilever moment attempted to pull them apart (Fisher 1984).

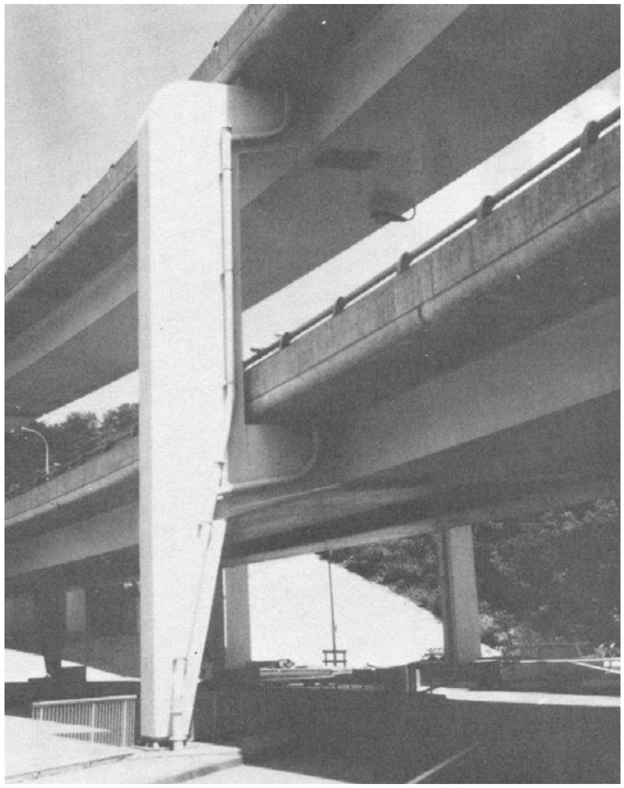


Figure 2.20 Rigid box girder frame of Ft. Duquesne Bridge (Fisher 1984)

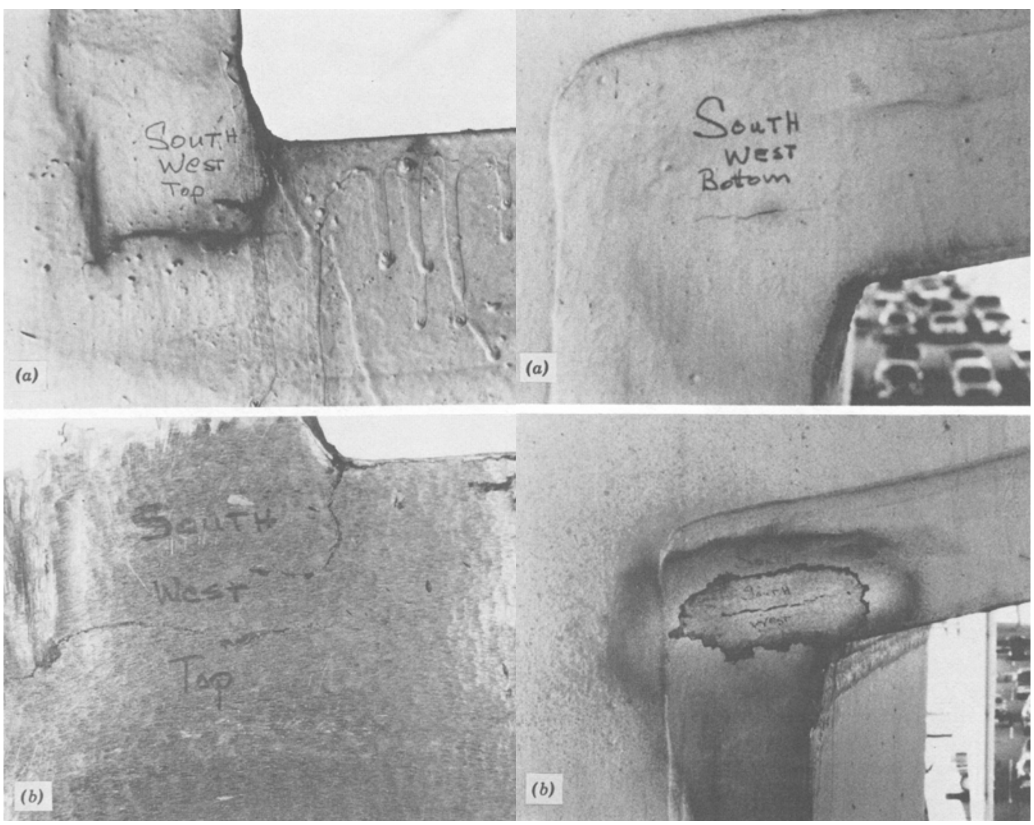


Figure 2.21 Lamellar cracks in girder tension flanges (left) and compression flanges (right) (Fisher 1984)

Due to the geometry of the structure, a very large moment was applied to the supporting arm. The girder flange was continuous through the column, creating a highly rigid connection. Lamellar tearing occurred in all cases before erection. The growth of the crack from the weld toe is what increased during the service life. If an initial crack of 1.8 in. is assumed, a stress range of 2 ksi yields an estimated 2.5 million cycles (Fisher 1984).

2.2 NCHRP Report 299 – Fatigue Life Evaluation Procedure

The following is a comprehensive procedure for estimation of the fatigue life of uncracked members subject to primary stresses. This method was conceived for the NCHRP in 1987 in response to the increased number of instances of cracking in steel bridges. The report consists of a multi-pronged method, in which traffic counts, vehicle weights, bending moments, and other factors, can be approximated. Furthermore, there are occasionally multiple different paths of approximation, each with a different level of accuracy. For the best estimation, detail dimensions can be measured and stresses can be gathered through instrumentation.

The procedure as it is shown below has a very low level of accuracy, since available traffic counts, details, and detail locations are not well known throughout Minnesota. Hence, although the procedure could be applied to the Minnesota Department of Transportation database, the usefulness and validity of the output is substantially diminished. In this case, the procedure is a reasonable starting point to establish which details will crack before others, rather than approximate lifespan.

For distortion-induced fatigue, moment range and member section can be found using the procedure described in Section 0. It outlines the stress estimation equations that were developed at the University of Minnesota (Jajich 2000).

When following the procedure, each aspect must be addressed, including: truck loading, total moment range, member section, reliability factor, remaining fatigue life factor, cycles per truck passage, lifetime average daily truck volume, and finally the remaining fatigue life. The structure of NCHRP 299 has been altered into a concise step-by-step procedure.

Truck Loading

Estimate fatigue truck geometry and weight by using either a standard AASHTO truck configuration for fatigue analyses (Figure 2.22), or another known configuration.

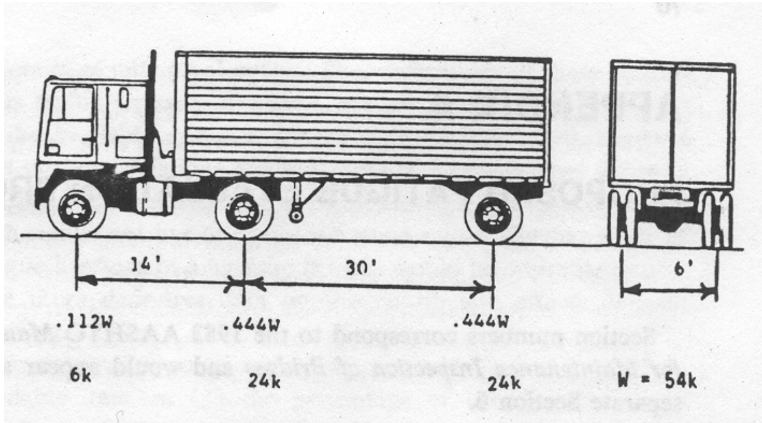


Figure 2.22 Standard fatigue truck geometry and loading (Moses et al., 1987)

Bunching occurs (traffic signal on/near bridge, steep hill on two-lane road)
Increase weight (*W*) 15%
Road is smooth (low impact loading)
Increase *W* 10%
Joint or pavement roughness could increase impact loading
Increase *W* 10% to 30%

Total moment range, ΔM

Transverse members – Place one fatigue truck at center of the traffic lane that produces the highest moment in the specific detail. The change in moment is the range.

Longitudinal members – Place one fatigue truck at locations that cause the maximum and minimum moments (or axial forces in truss members) at a specific detail. The difference is the range.

For straight longitudinal beams, girders, or stringers: $\Delta M_{new} = \Delta M \cdot DF$

DF = lateral distribution factor

Or use NCHRP Report 299 Appendix C method

Box-shaped members – divide into two I-shapes, each half of box and use following method for I-shapes

I-shaped members

2 members supporting deck – Assume deck is a simple beam with truck in center of outer lane.

More than 2 members

Interior members – $DF_i = \dfrac{S}{D} < \dfrac{S-3}{S}$

DF_i = interior distribution factor

S = girder spacing (ft)

D = interpolated from Table 2.16

Table 2.12 D factors for span length (Moses et al., 1987)

Span* (ft)	*D*
≤30	17
40	19
60	20
90	22
≥120	23

*For positive & negative bending regions in continuous spans, span length is distance between points of contraflexture under dead loads

Exterior members

Inner face of parapet or curb less than 1 foot outside centerline of exterior member OR lane shoulder width $> 4\,ft$: $DF_e = DF_i$

DF_e = exterior distribution factor

Otherwise

$P > 0.5$: $DF_e = 0.7 - 0.4P > DF_i$

$P \leq 0.5$: $DF_e = 0.9 - 0.8P > DF_i$

P = distance from exterior member centerline to outer lane centerline divided by girder spacing. (Negative if centerline is outside of exterior girder)

Member section

Bending members

$S_r = \Delta M / S_{member}$ where S_{member} = section modulus of member

Composite deck (with shear connectors)

Positive moment regions (dead load) – use composite section (article 10.38 of AASHTO Standard Specifications for Highway Bridges) and increase by 15%.

Negative moment regions (dead load) – use section including longitudinal rebar.

Noncomposite decks (without shear connectors)

No visual separation in positive bending regions – either use full composite section, or 30% increase over steel section

Visual separation or negative moment regions – steel section only

Truss members – $S_r = \dfrac{P}{A}$

Reliability factor, R_s

Remaining mean life – 50% probability that the remaining life will exceed this time.

$R_s = 1.0$

Remaining safe life – 97.7% probability for redundant members and 99.0% probability for nonredundant members that the remaining life will exceed this time.

$R_s = R_{s0} \cdot F_{s1} \cdot F_{s2} \cdot F_{s3}$

Redundant members – $R_{s0} = 1.35$

Nonredundant members $- R_{s0} = 1.75$

$F_{sn} = 1.0$ unless otherwise noted

Remaining fatigue life, Y_f

Infinite remaining life – if $R_s S_r < S_{SF}$ or $2R_s S_t < S_c$ fatigue life is infinite and no more calculations are needed

S_t = tension portion of stress range

S_c = compressive dead load stress

S_{FL} = limiting stress range from Table 2.17

Table 2.13 Table 10.3.1B of AASHTO Standard specifications for Highway Bridges

Detail Category	Detail Constant, *K*	Limiting Stress Range, S_{FL} (ksi)
A	68	8.8
B	33	5.9
B'	17	4.4
C	12	3.7*
D	6.0	2.6
E	2.9	1.6
E'	1.1	0.9
F	2.9	2.9

*Use 4.4 ksi for stiffeners

Cycles per truck passage, C

Longitudinal members

(a) Simple-span (L) girders

if $L \geq 40 ft$, then $C = 1.0$

if $L < 40 ft$, then $C = 1.8$

(b) Continuous span; within 10% of span on each side of interior support

if $L \geq 80 ft$, then $C = 1 + \frac{L-80}{400}$ where L is in feet

if $40 ft \leq L < 80 ft$, then $C = 1.0$

if $L < 40 ft$, then $C = 1.5$

(c) Continuous span girders elsewhere

if $L \geq 40 ft$, then $C = 1.0$

if $L < 40 ft$, then $C = 1.5$

(d) Cantilever girders (vibrations should be analyzed for increased stress cycles)

$C = 2.0$

(e) Trusses

$C = 1.0$

Transverse members (with spacing S, in ft)

if $S \geq 20\,ft$, then $C = 1.0$

if $S < 20\,ft$, then $C = 2.0$

<u>*Lifetime Average Daily Truck Volume*</u>

Present average daily truck volume in outer lane, $T = ADT \cdot F_T F_L$

ADT average daily traffic volume in both directions

F_T = fraction of trucks (excluding panel, pickup, & other 4 wheel/2 axle trucks)

If unknown, use 0.2 for rural interstates, 0.15 for rural highways and urban interstates and 0.1 for urban highways.

F_L = fraction of trucks in outer lane from Table 2.18

Table 2.14 F_L Values (Moses et al., 1987)

No. of Lanes	2-Way Traffic	1-Way Traffic
1	--	1.00
2	0.60	0.85
3	0.50	0.80
4	0.45	0.80
5	0.45	0.80
6 or more	0.40	0.80

Lifetime average daily trucks in outer lane, T_a is found using Figure 2.23.

T = present average daily truck volume in outer lane

a = present age of bridge

g = annual growth rate estimated by NCHRP Report 299 procedure

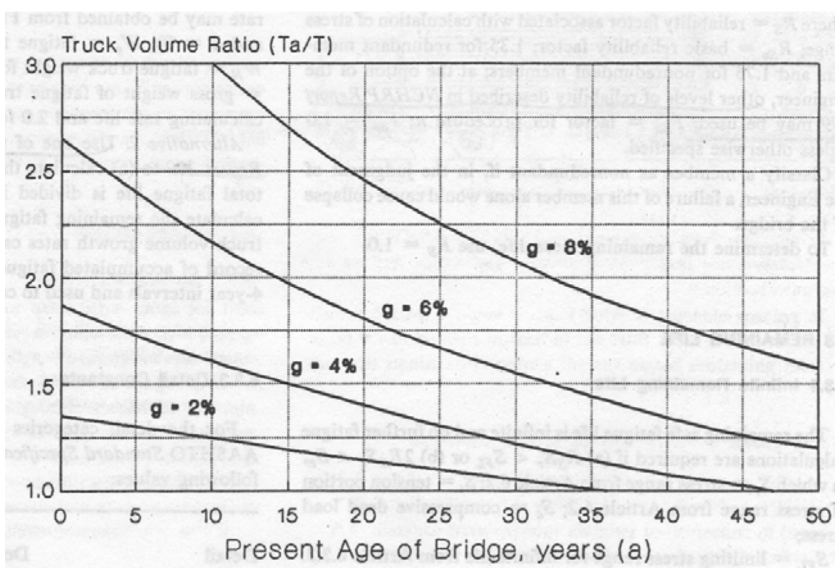


Figure 2.23 Growth rate graphs for determination of T_a value (Moses et al., 1987)

Finite remaining life
Single time period approximation method

$$Y_f = \frac{f \cdot K \times 10^6}{T_a C(R_s S_r)^3} - a$$

Y_f = remaining fatigue life in years
K = detail constant from Table 2.17
a = present bridge age (yr)
f = 1.0 for safe life, 2.0 for mean life

Double time period approximation method

$$Y_f = Y_N \left[1 - \frac{Y_p}{Y_1} \right]$$

$$Y_1 = \frac{f \cdot K \times 10^6}{T_p C(R_s S_r W_p / W)^3}$$

$$Y_N = \frac{f \cdot K \times 10^6}{T_N C(R_s S_r W_N / W)^3}$$

Y_p = present age in years
Y_1 = fatigue life in years from past volume
Y_N = fatigue life in years based on future volume
T_p = past average daily truck volume for outer lane
T_N = future average daily truck volume for outer lane
W_p = past weight
W_N = future weight

2.3 University of Minnesota Distortional Stress Estimation Procedure

Stresses caused by direct bending are well known. These can be replicated in a lab and translated to in-situ situations. The bending patterns and rotations caused by differential deflections, however, are more complicated. To gain an understanding of how stresses behave in diaphragms, it would be necessary to test numerous, multi-axial forces with many different lengths and positions. The solution instead is to attach strain gauges to actual bridges and augment strain data with finite element models.

Researchers at the University of Minnesota, under a Mn/DOT sponsored program, developed models of skewed bridges in an attempt to estimate the stresses in a certain web gap detail mentioned in section 0. During a five-year period, sustained research led to refinement of formulas and a more detailed classification of bridges.

In 2000, Jajich instrumented a skewed bridge that contained web gaps and beam diaphragms (Jajich et al. 2000). This single detail was examined at many different locations. By measuring displacements and stresses, an understanding of the stress concentration behavior was gained. After, a finite element analysis was conducted and calibrated with the readings. Finally, the life span of web gap fatigue was approximated.

From the data collected, it was discovered that the web gap stresses due to distortion were 2 to 4 times the stresses at the girder flange. From the finite element modeling, it was realized that the largest stresses at the toe of the fillet connecting the stiffener to the web, was double the value measured by strain gauges.

Traffic data over many months, as well as test runs with standard trucks enabled Jajich to estimate the number of loadings that occur within a given time frame. Combining these measurements, an estimate was made that the life of the web gap detail was between 45 and 75 years. Lastly, a general equation was formulated, Equation 2.1, estimating the relationship between geometric constraints and the peak stress.

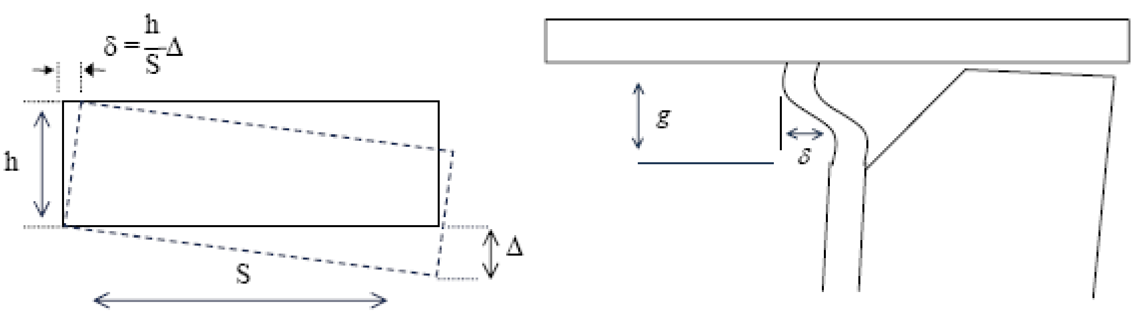


Figure 2.24 Distortional fatigue geometry (Jajich, 2000)

Equation 2.1

$$\sigma_{web-gap}^{\max} \approx \frac{3Et_w}{g^2}\delta = \left[\frac{3Et_w}{g^2}\right]\left[\frac{h\Delta^{girders}}{S}\right]$$

Berglund and Schultz (2002) altered the formula by Jajich to better compare with calibrated results (Equation 2.2). Berglund's most important contribution, however, was the development of a formula for estimating the differential displacement Δ, or $\Delta^{girders}$, between adjacent girders. He also provided an estimate of gap length when none was available in the bridge plans. Severtson et al. (2004) further modified this equation by neglecting lateral deflection of the web gap, δ. His study focused on the behavior of cross braces rather than rolled plate diaphragms. These braces resulted in significantly different results for stress approximation. Lastly, Li and Schultz (2005) tuned the equation with a more comprehensive finite element model and accounted for J-rail and sidewalk contributions. She also made changes to the estimation of differential deflection. In this section, a brief overview of the progression of the web gap stress equation is shown, as it could be incorporated into Mn/DOT's database.

Equation 2.2

$$\sigma_{wg} = 2E\left(\frac{t_w}{g}\right)\left(\frac{\Delta}{S}\right)$$

Refinement of the formula resulted in inclusion of the bridge skew angle as shown below.

Equation 2.3

$$\frac{\Delta}{S} = \frac{A_1 \cdot L^2 + A_2 \cdot L + A_3}{L}$$

Where values are taken from Table 2.19 for 20º, 40º, and 60º skews and interpolated for other angles. L is the span length.

Table 2.15 Skew constants (Berglund, 2002)

(L in meters)			
Deg.	A_1	A_2	A_3
20	-0.00001327	0.001486	-0.00864
40	-0.00001227	0.001522	-0.01034
60	-0.00001714	0.002185	-0.02328
(L in inches)			
Deg.	A_1	A_2	A_3
20	-3.3700E-07	0.001486	-0.3399
40	-3.1150E-07	0.001522	-0.4065
60	-4.3520E-07	0.002185	-0.9156

Since engineering plans and shop drawings sometimes do not show the distance between the bottom flange and the vertical stiffener, approximation for unknown web gap length is needed. t_w/g from Equation 2.4 can be used as a substitute in these cases.

Equation 2.4

$$\frac{t_w}{g} = K \cdot L + 0.4091$$

$K = -0.002858$ for L in meters
$K = -0.00007260$ for L in inches

Using reasoning similar to Jajich (2000), Severtson, Beukema and Schultz (2004) rederived the web gap stress equation by simplifying the slope-deflection formula for linear beam theory (Equation 2.5), and neglecting lateral deflection of the web gap, δ. This resulted in Equation 2.6 which accounted for the behavior of cross-brace diaphragms rather than rolled plate diaphragms. As is evident these cross braces give a much larger coefficient (3.5) than the one in Berglund's equation.

Equation 2.5
$$\sigma_{wg} = \frac{E \cdot t_w}{g}(2\theta_b + \theta_t + 3\frac{\delta}{g})$$

Equation 2.6
$$\sigma_{wg} = 3.5E\left(\frac{t_w}{g}\right)\left(\frac{\Delta}{S}\right)$$

Standard girder spacing for multi-girder bridges is 10.5 feet, for which Equation 2.6 was derived. Equation 2.7 is a cross-brace correction factor, which is multiplied by the Δ for bent-plate diaphragms to get Δ for cross-brace diaphragms of 10.5 ft. R_x is the ratio of deflection of cross-brace diaphragms to the deflection of bent-plate diaphragms.

Equation 2.7
$$R_x = 1 + B_1 \cdot L^2 + B_2 \cdot L$$

Table 2.16 Cross-brace constants for 10.5 ft girder spacing (Severtson and Beukema, 2004)

L	B_1	B_2
ft	-1.931E-05	5.432E-04
in	-1.341E-07	4.527E-05
m	-2.078E-04	1.782E-03

Li and Schultz (2005) further analyzed the web gap stress by examining the I94/I694 Bridge and the Plymouth Avenue Bridge. Her study focused on better estimation of differential deflection. She calibrated the equation to include new factors for different girder spacing and a relationship between sidewalks and J-rails on the Plymouth Avenue Bridge. These can then be applied to $\sigma_{wg} = C \cdot E(t_w/g)(\Delta/S)$, where $C = (2\theta_b + \theta_t)/(\Delta/S)$ is the web gap stress coefficient.

Table 2.17 Cross-brace constants* for 8 ft to 9.25 ft girder spacing (Li, 2005)

L	B_1	B_2
ft	-1.038E-05	3.232E-04
in	-7.209E-08	2.694E-05
m	-1.117E-04	1.060E-03

*Interpolate for spacing between 10.5 ft and 9.25 ft

Li's factor for the case when sidewalks are present instead of J-rails (Equation 2.8) is applied to Δ in the calculation of web gap stress. In most cases, R_d is less than unity, meaning sidewalks reduce distortional stresses by stiffening the bridge and reducing differential deflection.

Equation 2.8
$$R_d = 0.0013 \cdot L + 0.7378$$

For diaphragms and geometry similar to the I94/I694 Bridge, found by including correction factors, C = 2.25. For cross-braces and geometry similar to Plymouth Ave. Bridge, found by including correction factors and the sidewalk factor, C = 2.75.

If web gap lateral deformation is determined to be of importance, then defection the normalized web gap deflection, $\bar{\delta}$, should be included (Equation 2.9 & Equation 2.10).

Equation 2.9 $$\sigma_{wg} = C \cdot (1 + 3\bar{\delta}) \cdot E\left(\frac{t_w}{g}\right)\left(\frac{\Delta}{S}\right)$$

Equation 2.10 $$\bar{\delta} = \frac{\delta / g}{(\theta_t + 2\theta_b)}$$

By using the equation $\sigma_{wg} = C \cdot E\left(\frac{t_w}{g}\right)\left(\frac{\Delta}{S}\right)$, where C depends on a number of variables as described above, the distortional stress range in the web gaps can be approximated. The factors that affect this stress, such as presence of sidewalks and distance between girders, can be used to connect the Mn/DOT bridge database and the equations developed in the distortional fatigue research. These stresses can then be used in the procedure found in section 0 for approximation of bridge lifespan to create a preliminary model for classification of details.

Because the details in this study are classified in such a coarse way, small changes to the equations, such as sidewalks and J-rails, should not be considered. The resulting equation used for the current study is a combination of Equation 2.2, Equation 2.3, and Table 2.19, taking *C* to be 2.5. The reason for taking *C* equal to 2.5 is to represent an average between the *C* values for rolled plate diaphragms (2.0 from Jajich's study) and cross-brace diaphragms (3.5 form Severtson's study), as well as the differential deflection modifications from Li. Different constants could be used for the different types of diaphragms, but the coarse ranking is already limited to a lower level of accuracy. The resulting web gap stress formula is

Equation 2.11 $$\sigma_{wg} = 2.5E\left(\frac{t_w}{g}\right)\left(\frac{A_1 \cdot L^2 + A_2 \cdot L + A_3}{L}\right)$$

where E is the modulus of elasticity (29,000 ksi), t_w is girder web thickness, A_1, A_2, A_3 are factors for bridge skew (Table 2.19), g is web gap width, and L is bridge span length.

CHAPTER 3 – FRACTURE BACKGROUND

Brittle fracture is cause for concern in many of the details which exhibit fatigue problems. Few examples are present which show rapid fracture without initial fatigue. Therefore, to enumerate which details are prone to fracture, one must have a thorough understanding of which are fatigue-sensitive. It is also common to have both types of cracking, where a fatigue crack causes rapid fracture, then more fatigue, followed by fracture again.

Because fracture happens much faster than fatigue, taking only minutes compared to years, it is important to protect against catastrophic failure. Another problem with fracture is the inability to predict cracking due to a reduced number of warning signs. To best address these issues, a basic understanding of fracture mechanics and fracture histories can be used to note where possible hotspots are.

Factors contributing to fracture include: geometry, initial defects, temperature, Charpy V-notch toughness, and loading rate. Of these, temperature is almost impossible to predict, Charpy V-notch toughness may not be available, loading rate is difficult to quantify without measurement, and initial defects are unknown unless they are already recorded from inspections. This leaves geometry, which is shown to a certain degree, on bridge plans.

3.1 Standard Fracture Practice Reevaluation

Some important lessons have been learned from past experiences to help mitigate fracture and these have led to changes in standard practices. However, there are still many current issues that need to be addressed. A few code changes that have been made are the use of greater toughness welds, the removal of backing bars from bottom flanges, and sealing at top flanges. There are many areas that need improvement: The first is the rotary straightening of rolled shapes, which reduces toughness and ductility in the K-area. Welding of stiffeners in the K-area may continue to cause fabrication cracks. UT testing has proven to be unreliable and costly. Weld toughness is certified only once per year, hence it is significantly variable. This testing is also only performed in butt-welds on a flat plate; furthermore, a safety factor should exist between certification and Charpy-V Notch brittle fracture (Dexter 2003).

3.2 Welding Fracture Problems

In 1978 the United States Department of Transportation published “A Proposed Fracture Control Plan for New Bridges with Fracture Critical Members.” (FHWA 1978) This plan consisted of changes to welding codes, AASHTO Bridge Specifications, and general practice. By outlining changes that needed to be incorporated into current design, USDOT forecasted issues with former designs. Following are many of the changes to welding standards proposed by the document.

Some specific welds were outlined by the plan as being prone to fracture.

- Partial joint penetration (PJP) welds in fracture critical members (FCM) or loaded in tension orthogonally.
- Grinding groove-, butt-, and corner-welds flush on FCMs is necessary to reduce any stress concentration.
- Plug, slot, intermittent fillet, and groove welds are also stress-raisers.
- Holes or slots for another member to pass through should not be seal welded. Where used, the seal welds should not provide any rigidity.
- Fillet welds are permitted for shear only, not tension or compression normal to the weld.

Other changes include prohibiting types of details that stress welds in similar ways to those in the previous list.

- Lap joints in FCMs and Corner and T-joints that have some component of tension would all put fillet welds into tension.
- Butt-joints and groove welding from a single side without steel backing are considered problematic.
- Joints perpendicular to the applied stress with backing left in place or backing fillet welded outside the groove weld will cause stress concentration and possible fracture.
- Temporary tack-welding must be removed from bridges.
- Field welding of all FCMs is disallowed.
- Because of the way bolts are forgiving and welds are rigid, connections are no longer allowed to be designed to split the load between these two mechanisms.
- If stiffeners are used on only one side of the web, they must be welded to the compression flange.
- Diaphragms and hangers with flange or web connections perpendicular to applied stress require a complete joint penetration (CJP) welded T-joint.
- Cover plates have been restricted to eliminate the stress concentration at their ends. There are limited to one per flange, no more than 1.5 times the flange thickness. The plates should have square ends, be ground flush, and welded continuously across ends. In FCMs, when partial-length cover plates are used, the ends of the cover plates should be in compression.

Changes to procedures for designing fracture resistant details have assisted in managing failures. "Almost two decades of experience with [AASHTO Guide Specifications for Fracture Critical Non-Redundant Steel Bridge Members] have proved that they are successful in significantly reducing the number of fatigue cracks and brittle fractures." (Dexter, 1997)

Another detail that has an increased chance of cracking is the stud connector for composite deck slabs as seen in Figure 3.1. When shear connectors are used in negative moment regions, tension cracks may appear on the underside of the top girder flange (FHWA 1986).

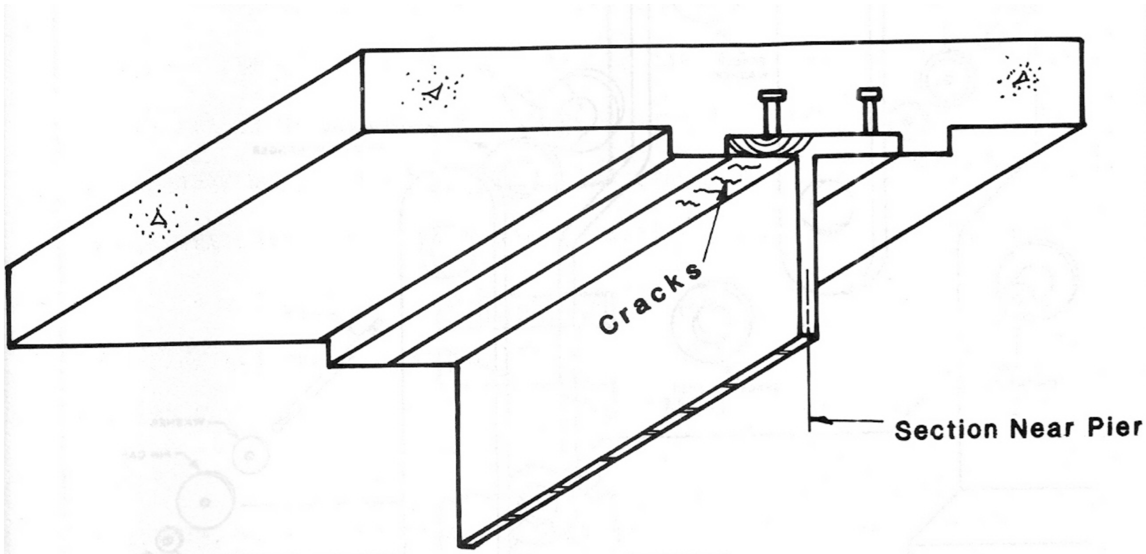


Figure 3.1 Composite section cracks from studs (FHWA 1986)

3.3 Case Studies

Besides recognizing that certain welds would increase stress concentration in various situations, the same details described in the fatigue section apply to fracture. Cover plates at Yellow Mill Pond Bridge showed only a small length of unstable fracture. Otherwise, the crack was generally completely fatigue. Web connection plates of the Lafayette Street Bridge exhibited brittle cracking after a fatigue crack propagated into the web from the gusset plate-stiffener weld. Besides a small section of fatigue in the girder web, the entire bottom flange and web were cleavage type fractures, probably occurring on a -22ºF day. (Fisher 1984)

Transverse groove weld cracking occurred in the Aquasabon River Bridge, Quinnipiac River Bridge, and U.S. 51, but varied significantly. The Aquasabon River Bridge cracked at the end of a horizontal stiffener, but did not show any fracture. Cracking of the Quinnipiac River Bridge began as a preconstruction groove weld crack, and fractured through the horizontal stiffener. At the same time, the crack fatigued into the web, where it fractured in a brittle manner through the web and into the bottom flange. Subsequently, it fatigued through the flange. Lastly, a groove-weld cracking of a cover plate on the U.S. 51 Bridge showed only fatigue cracking. (Fisher 1984)

Web penetrations of the Dan Ryan Rapid Transit structure show that cracking was largely due to fracture, with all cracks initiating from fatigue. Continuous longitudinal welds along the corners of the box sections on the Gulf Outlet Bridge occurred most likely during welding and exhibited little fatigue and no unstable fracture growth. (Fisher 1984)

Of the cracked details described in the fatigue section that were distortion induced, none showed signs of fracture surfaces or unstable crack growth propagation. All of the cracks were discovered before they had an opportunity to propagate into areas where rapid unzipping was likely. Attention must still be paid of to these details; if left unattended fatigue could easily lead to uncontrolled fracture. (Fisher 1984)

3.3.1 Plug-welded Misplaced Hole

The Illinois I-57 overpass at Farina had misplaced holes in two beam webs that were plug welded and redrilled. The holes were for bolted diagram connections. Brittle cracks propagated from these holes, cracking through the bottom flange and along the web of the girder. The longest of these cracks was fifteen feet along the girder web. (Fisher 1984)

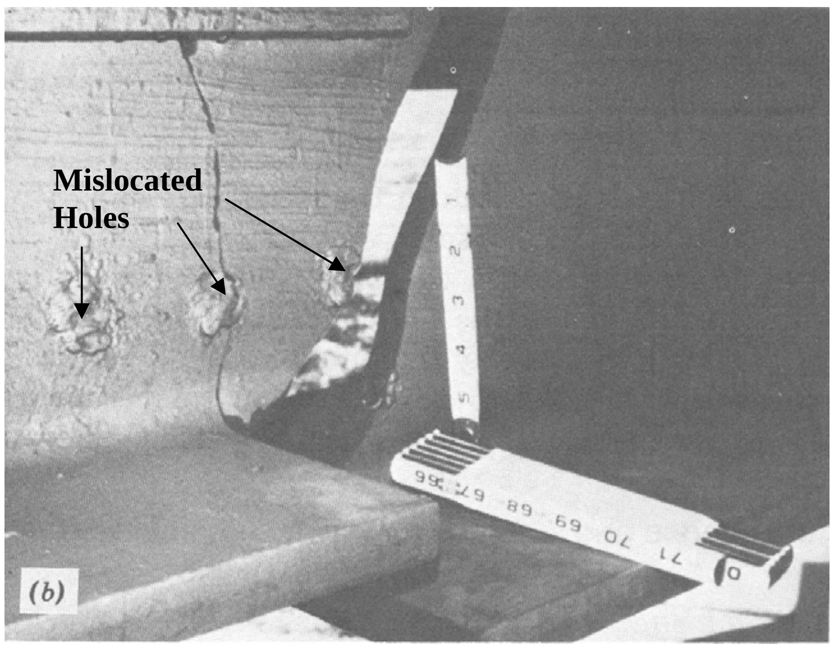


Figure 3.2 Cracking at mislocated bolt holes of Highway 28 Bridge over I-57 (Fisher 1984)

At best guess, brittle fracture occurred during extremely cold temperatures. The bridge had no heavy traffic loading and only light farm equipment and small vehicles. Stress range was estimated at only about 3 ksi with a maximum at 3.46 ksi from strain gauge readings, far below AASHTO allowable stress for these members. The estimated number of cycles was 130,000 to 260,000. Other tests were conducted to check ASTM-A36 specifications and Charpy V-notch toughness; the beam was well within limits. The only conclusion was that "fatigue sharpened the natural cracks, and brittle fracture resulted from the presence of plug-welded holes." This detail, therefore is considered almost completely fracture susceptible. (Fisher 1984)

3.3.2 Eyebar and Pin Plate

Eyebars and pin plates usually involve heat treatment and forge welding, which create a weakness that eases cracking. The details are fracture critical since cracking is sudden and can lead to destruction of the entire bridge, as was seen by the Point Pleasant (Silver) Bridge. They are almost always replaced when found

The failure Point Pleasant Bridge is one of the most famous. It opened in 1926, and on December 15, 1967 all three spans collapsed and 46 lives were lost (Figure 3.3). The temperature was 30ºF. The failure is attributed to either overextension, or brittle fracture of corrosion cracks on one of the eyebars near the pinhole (Figure 3.4).

Figure 3.3 Collapse of Point Pleasant (Silver Bridge) (Fisher 1984)

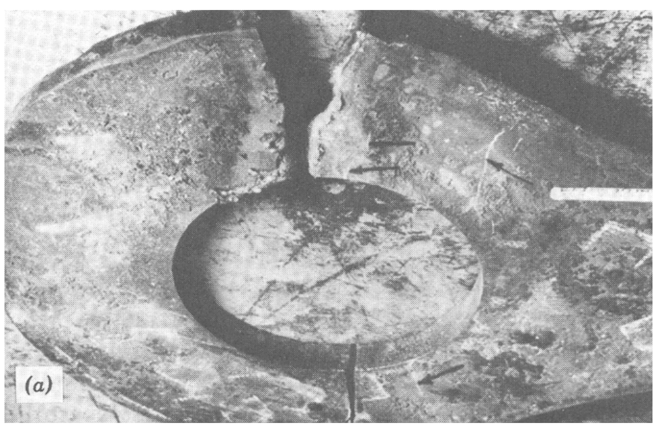


Figure 3.4 Eyebar fracture from Point Pleasant (Silver) Bridge failure (Fisher 1984)

The reason for the catastrophic collapse was the lack of redundancy in the structure, making the eyebars fracture critical. It was determined that the fatigue resistance of the setup was significant enough to prevent collapse. The prospect of traffic being more than allowed by design is still a possibility. Certainly though, corrosion and a high hardness were the main causes of cracking. This detail is certainly one that is volatile in its ability to crack catastrophically.

Another example of fracture resulting from corrosion issues is Illinois Route 157 over St. Clair Avenue. In this situation, water and salt were able to accumulate on the pin hanger expansion joint, causing the joint to freeze. The combination of thermal stresses, because the joint was frozen, and maximum truck traffic caused the girders to exceed their yield stress. There were fractures in multiple girders, causing a drop of one-half to three-quarters of an inch.

CHAPTER 4 – VARIATIONS OF COMMON DETAILS

In general, details about cracking flagged elements are not well known. However, a few failures are common enough to separate them in to gross categories. To do this, additional information must be collected from published sources that have investigated the crack probability, or the number of cracks per number of details of a certain type. There is simply not enough history to allow for separation of detail variations.

Of the details flagged from this study, the most cracks occur in: web gaps, copes, partial length welded cover plates, shelf plates welded to girder webs, and welded horizontal longitudinal stiffeners. These five details may indeed be common enough to separate into different sub-categories; nevertheless, lack of past research makes it difficult to do so. Furthermore, sub-classification of these details may be finer than the ranking system used and any fine-tuning may be lost in the gross ranking.

Although the recording of variations among similar details may seem futile, the information collected in the database will enable future classification to a finer degree. At a minimum, this process will allow details to be classified according to their correct AASHTO *S-N* category, which may prove useful if stresses can be calculated in the future. The other benefit to sorting elements is that the database will display the worst one, which will allow inspectors to be aware of the most severe case.

In the following sections, equations and classifications are presented based upon either past code restrictions or research-backed formulas.

4.1 Transverse Stiffener Web Gap

Research at the University of Minnesota has allowed for comprehensive formulas for estimating stresses. Regrettably, the information needed for the analysis of a bridge includes items which are not always available (e.g., actual length of the web gap, *g*) in order to use the most precise of the formulas. Simplifications to the formula by Berglund allow for a somewhat less precise estimation of stress for an HS-20 truck:

$$\sigma_{WebGap} = 2.5E\left(\frac{t_w}{g}\right)\cdot\frac{A_1L^2 + A_2L + A_3}{L}$$ where A_1, A_2, A_3 are from Table 2.19, which relates to the skew of the bridge. Additionally, $\left(t_w/g\right)$ can be approximated as (0.4091 - 0.0000726*L*), where *L* is span length in inches, if t_w and *g* are unknown.

The factors for cross braces and sidewalks are considered too precise for this study. Using a general truck count given in the database (ADTT), the length of the spans, the thickness of the web, the web gap distance, and bridge skew, the formula may be used to estimate the number of cycles before cracking. Conservatively, “anything bigger than a pickup is considered a truck,” (Pierce, July 2006) as determined by Figure 4.1. Since no other detail is able to give an accurate stress level, it will not be calculated for this one; therefore, traffic levels are not used in the current study.

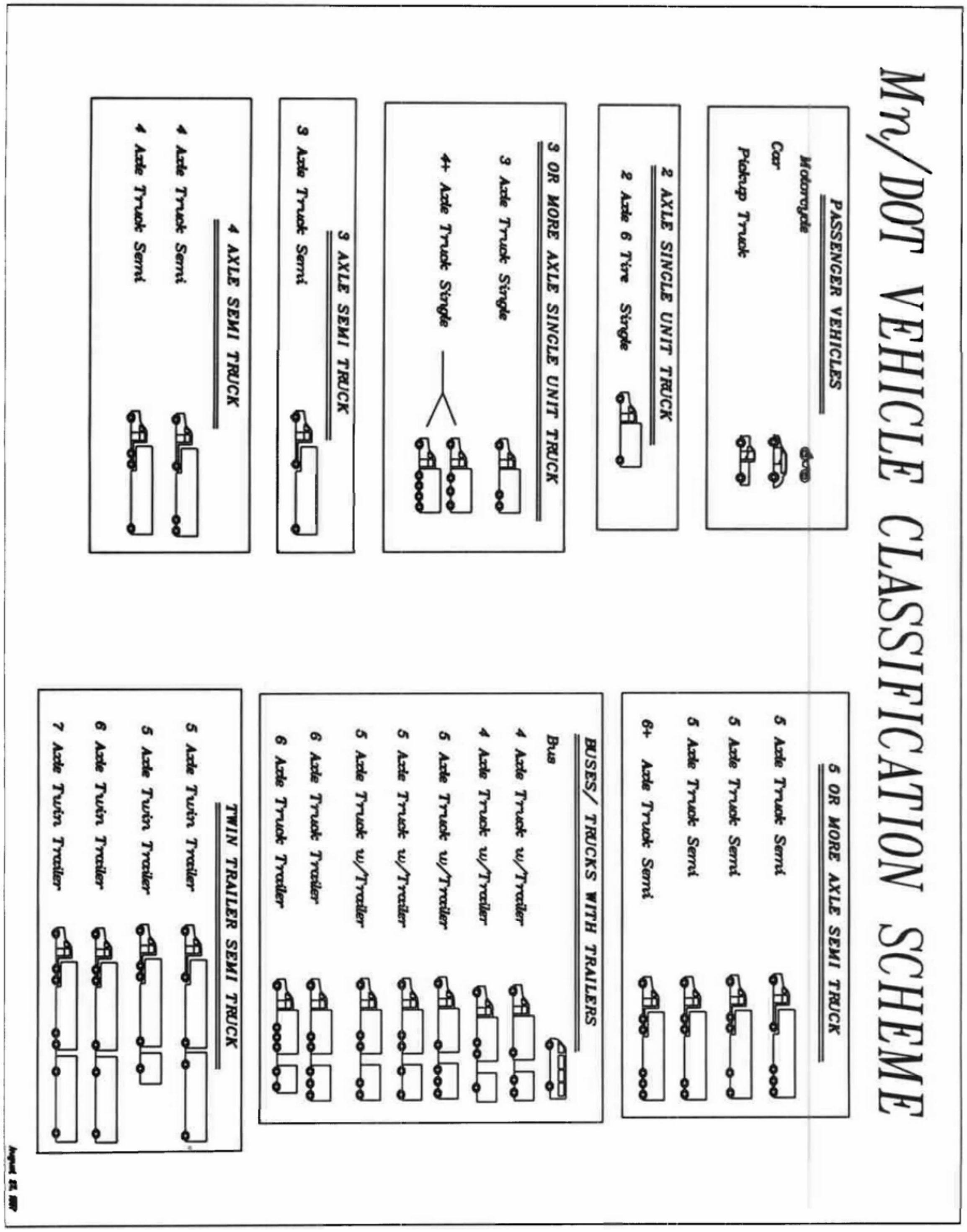


Figure 4.1 Mn/DOT vehicle classification scheme (Pierce, July 2006)

4.2 Insufficient Cope Radius

The AISC Manual of Steel Construction in 1986 recommended a cope radius of 0.5 inches (12.7mm) for all purposes (including buildings) for coped beams. Coped flanges made by flame cutting cause high residual tensile stresses. Gouges, overcuts, and small radii make the area highly susceptible to fatigue.

The current Mn/DOT procedure requires a re-entrant radius of 1 inch (Minnesota Department of Highways 1964) and a radius of 2 inches for copes used at hinged joints (Dahlberg, July 2006)

One study showed that the theoretical stress concentration factor, *SCF*, as a function of cope radius, *R*, could be determined by Equation 4.1 (Yam, 1990).

Equation 0.1 $$\log(SCF) = 0.937 - 0.285\log(R)$$

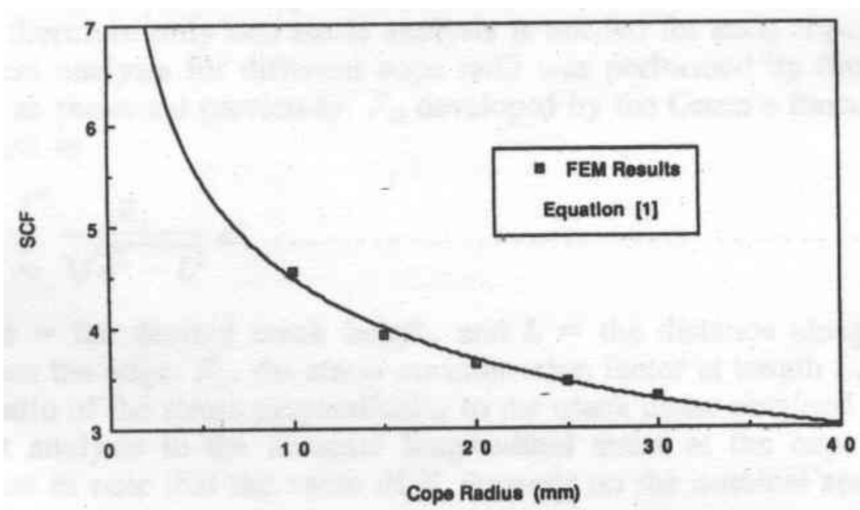


Figure 4.2 Stress concentration factors for various cope radii (Yam, 1990)

To relate this method to the *S-N* fatigue categories, simply multiply the nominal stress range by the *SCF* and use this stress on the category B curve (Yam, 1990). As seen by Figure 4.2, once the cope radius is less than one inch (25mm) the stress concentration factor grows at a significant rate. For this reason, separating copes with smaller radii is important. Simply distinguishing between filleted and square-cut corners is insufficient. Furthermore, recording the cope with the smallest radius may have a significant effect on ranking of the bridge.

4.3 Partial Length Cover Plate

Much of the classification of cover plates comes from information gathered by researchers at Lehigh University in the mid 1970s. The first publication of any classification was in Fisher's *Detection and Repair of Fatigue Damage in Welded Highway Bridges* (Fisher, 1981), where the thickness of cover plates came into effect. From his research, Fisher determined that for plates narrower than the flange or wider than the flange with welds across the ends and the thickness of the flange was greater than 0.8 inches, the stress category was E. Otherwise it was E'.

This finding was translated to the AASHTO code in 1983, in section 10.13.2: "Partial length welded cover plates shall not be used on flanges more than 0.8 inches thick for non-redundant load path structures subjected to repetitive loadings that produce tension or reversal of stress in the member." No distinction is made in the AASHTO code for tapering of either thickness or width of cover plates.

The location of welded cover plate termination could also be an important factor in cracking. In the past, the AASHTO code required that plates be terminated a certain distance past where they were required. No AASHTO code change provided a limit for termination of welded cover plates near the inflection point. There is also no evidence that Mn/DOT actually terminated any plates deliberately at inflection points and no conclusions can be drawn that cracking was less severe. For further information, see section 0 on Mn/DOT history.

4.4 Shelf Plate Welded to Girder Web

Shelf plates are considered an attachment in the direction of the applied stress and when they are longer than 4 inches AASHTO stress category E should be used (Fisher, 1981). Cracks initiate most often at intersecting welds where lateral plates are attached to girder and transverse stiffeners. The intersecting welds in the corner allow for cracks to propagate into the girder web, leading to brittle fracture of the girder (Fisher, 1981 & 1984).

If the shelf plate is groove welded to the girder web then the 1989 AASHTO Code classifies the plate depending on its length, thickness, and transition radius:

For lengths greater than $12t_p$ or 4": if $t_p < 1"$ then E; else if $t_p \geq 1"$ then E'

For transitioned ends with welds ground smooth, and radius R:

If $R \geq 24"$, then B; else if $24" > R \geq 6"$, then C;
else if $6" > R \geq 2"$, then D; else if $2" > R \geq 0"$ then E

If there is a transition, without the ends ground smooth, then category E applies.

A slightly different version of this was used in the 1983 AASHTO specification as can be seen in the timeline chapter of this report.

The termination point for stiffener welds can be seen in Figure 4.3, which shows the standard stiffener detail Mn/DOT currently uses. This clip would apply at the shelf plate detail as well, as noted by Dave Dahlberg (2006) and any stiffener corner clip size less than that shown in the diagrams is ranked as higher danger.

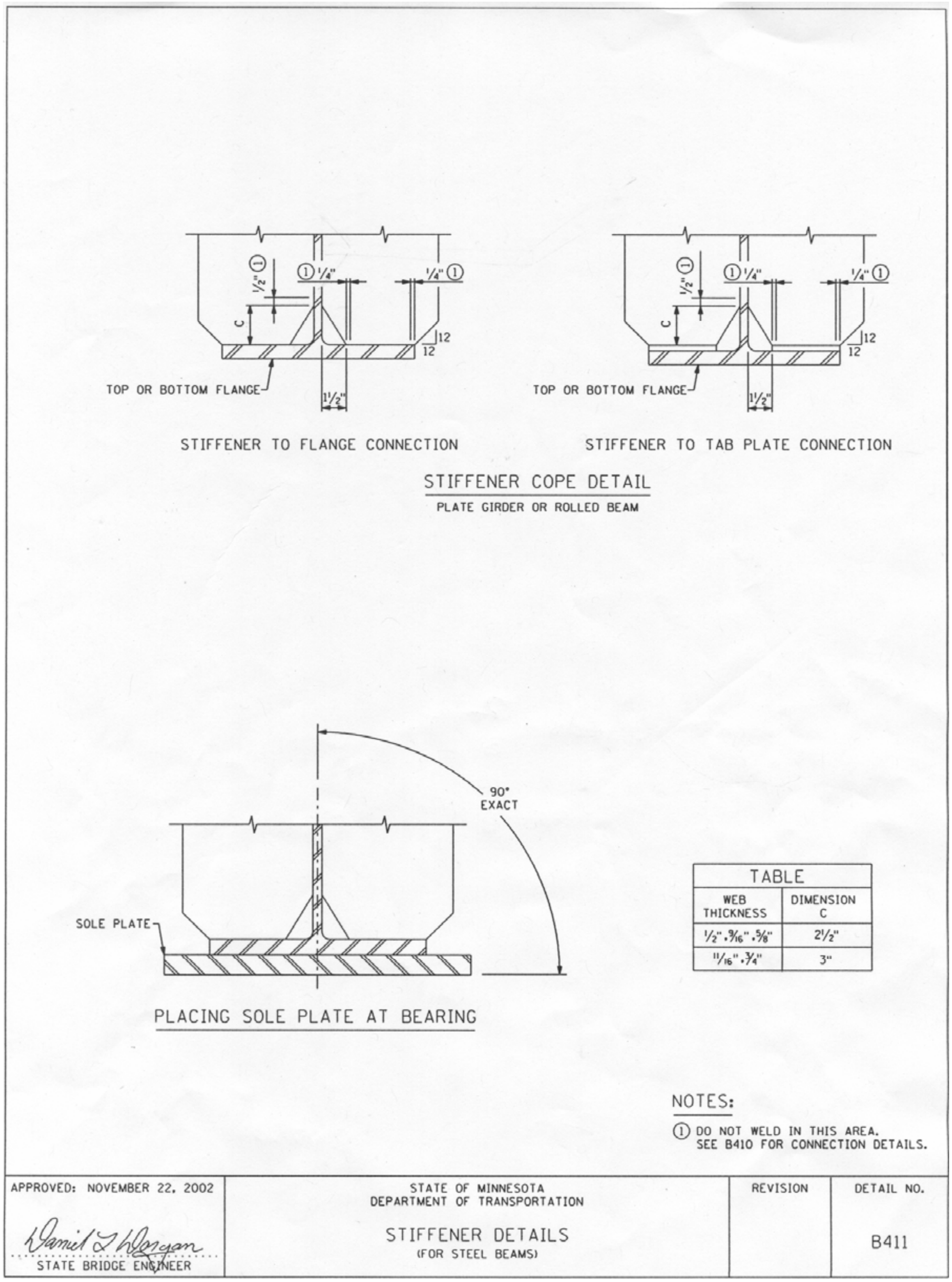


Figure 4.3 Standard weld termination distances for stiffeners (Mn/DOT, 2002)

4.5 Welded Horizontal Stiffener

According to Fisher, the critical point for longitudinal stiffeners is at the end of the longitudinal weld; he assigns that location as fatigue category E (Fisher, 1981). Any intermittent welding of longitudinal stiffeners should be considered more susceptible than continuously welded ones.

Groove welds made in the 1940s and 1950s rarely had adequate nondestructive testing, since secondary elements were not paid the respect they should have. ". . . connected parts were thought to have only architectural significance." Commonly, longitudinal stiffeners were added in tension regions to create an unbroken line. Horizontal stiffeners in tension regions are of special concern (Fisher, 1981).

Other problem areas include welds connecting the ends of horizontal stiffeners when they are joined and when A514 steel was used, due to its difficulty in welding (Fisher, 1981).

The NCHRP Report 206, published in 1979 classified the ends of horizontal stiffeners as E and their middle as B. The AASHTO 1989 code adopted the following classification for fillet welds on horizontal stiffeners:

For no transition, if $t_p < 1''$, then E; else if $t_p \geq 1''$, then E'

For transitioned ends with welds ground smooth, and radius R:

If $R \geq 2''$, then D; else if $2'' > R \geq 0''$, then E

If there is a transition without the ends ground smooth, then category E applies.

CHAPTER 5 – DEPARTMENT OF TRANSPORTATION HISTORY

5.1 General

Besides ranking bridges in terms of fracture danger, ranking in relation to probability of initial cracking is helpful. To achieve this, information from many state DOTs, especially Mn/DOT, pertaining to the number and type of cracks they experience must be collected. Then, by compiling the cracking histories the details which are most susceptible to fatigue can be determined.

This collection of data is not without problems, however; a coordinated effort by DOTs to compile their information is nearly nonexistent. Instead, code changes and research are evaluated on a case-by-case basis when individual DOTs feel addressing a detail is imperative. To better understand how a larger collection of bridges is performing, cooperation among various groups should be increased.

The background for Minnesota's history is derived from in-person interviews conducted in 2006 by the first author with staff from the Mn/DOT Office of Bridges and Structures, including David Dahlberg, Pete Wilson, and James Pierce, during which the specifics of details found to cause fatigue and fracture cracking were discussed. The cooperation of other DOTs sparked by this report involves a survey intended to gain an understanding of the general *order* of susceptible details. The term "order" refers to a numbering of details listed in the survey (Section 0) by the DOTs. The order is a numbering from 1 to 11 which arranges details in order of frequency of cracking, with 1 representing the most frequent, and 11 the least. Not all numbers must be used, and only the details that have had cracks are numbered.

5.2 Mn/DOT Detail History

<u>Transverse stiffener web gap</u>

Before 1985, gaps were left between transverse stiffeners and girder flanges in an attempt to avoid welding to the tension flange and transferring torsion inform the lateral bracing systems. Geometric factors affecting stress levels are identified for this detail in section 0. Web gaps in Minnesota usually varied between tight fit and two inches, with fewer problems occurring on the bottom flange than the top (Wilson, May 2006). Many small cracks have even been found in the base metal of girders containing these details. Small plates parallel to girder flanges were sometimes added as a retrofit, filling the gap; however, these were commonly welded to stiffeners and only tack welded to the girder flange. They too developed cracks as seen in Figure 5.1 and 5.2.

Figure 5.1 Tack welded web gap filler plate - Bridge #55803 (Wilson, October 2006)

Figure 5.2 Tack welded filler plate - Bridge #55804 (Wilson, October 2006)

Insufficient cope radius

Cracking of copes has occurred in Minnesota and is usually the effect of a design flaw. Problems occur when the cope radius is too small or is flame-cut. There are quite a few overcut copes as well, which mostly occur on floor beams. Problems are common where top and bottom flanges are cut from the beams so that they can be fit to connection plates (Wilson, May 2006). After 1964, radius size and flame cutting requirements were adopted by the Standard Specifications for Highway Construction, with a current requirement of one inch radius (Mn/DOT, 2005).

Partial length cover plate

Welded cover plates have been used by Minnesota for many years to increase local flange thickness. They usually terminate in "Area A," the negative moment region for dead loading. One case was found where a crack propagated entirely through the top flange after initiating from a cover plate. Although this was the only case found where cracking was in the base metal, a handful of cracks have been found in cover plate welds that would have eventually lead to similar outcomes.

Dave Dahlberg examined a sample set of continuous steel bridges on the trunk highway system between 1950 and 1989 and found that only one case from twenty-six examined had cover plates ending at the inflection points. "On that bridge, the cover plates ran to the field splice, which is the approximate location of the inflection points. For the rest of the bridges, it looks like they stopped the cover plates as specified by the code." (Dahlberg, August 2006)

Further investigation on another set of five Minnesota bridges with construction dates ranging from 1960's to 1981 revealed that only one bridge, Mn/DOT #02803, had plate terminations close to inflection points. In addition, the bridge from 1981 did not have terminations near inflection points. From this, it can be deduced that coverplates were not terminated at inflection points intentionally until at least after common use ended in the early 1980's. Minnesota bridges known to have examples of cover plates include: #9779, #9780, #19843, #82801, #02803, and #27015.

One factor that could possibly affect the cracking of the cover plate detail is the geometry of the cover plate tip. In Minnesota, tapering is usually done by decreasing the width of the cover plate near the end, as opposed to thickness which is common among other DOTs (Peterson, May 2006). The overall cover plate and flange thicknesses are certainly possible factors affecting cracking, congruent with the AASHTO Code.

Shelf plate welded to girder web

Most of these welded details have already been examined and are flagged in the Mn/DOT database and no bolted shelf plates have been found. Commonly, shelf plates intersect vertical stiffeners which create intersecting welds, considered to be stress-raisers. To prohibit fatigue issues the shelf plate should be welded to the stiffener with copes around the stiffener welds. When cracking has occurred, it was usually attributed to a bad cope detail (Wilson, May 2006). Some factors that may increase susceptibility of this detail are longer plates, intersecting welds, and stiffener to plate welds terminating too close to the girder web, in violation of the current detail standard, Figure 4.3 (Peterson, May 2006). Bridge # 9320 has an example of this detail.

Stringer or truss floor-beam bracket

Floor beams connected with riveted angles are very common and have developed cracks (Peterson, May 2006). There has only been one case of complete failure of the angle, probably due to excessive truck traffic. These are typically on fracture critical bridges and frequently fail due to corrosion. Floor beam to floor beam connection plates are not as much a concern as floor beam to truss connections. The only cracks from this kind of detail were due to lateral forces not linked to fatigue (Wilson, May 2006). Both of these details are most likely affected by average daily traffic (ADT) (Peterson, May 2006).

Welded horizontal stiffener

Horizontal stiffeners are common in large bridges and bridges with haunch plates. A couple of cracks have been discovered where intersecting welds occur. Minnesota has yet to see crack propagation into the base metal (Wilson, May 2006). There is not a written policy in Minnesota, but in general, if a horizontal stiffener ends in a tensile zone a minimum radius of six inches with fillet welds would be used (Category D). If the engineer feels fatigue is a concern in the area of stiffener termination, a groove weld might be used with a larger radius to change the AASHTO *S-N* category to C or B. For older designs, it is thought that a similar design process was used (Dahlberg, May 2006). Factors affecting cracking in Minnesota may include: intersecting stiffeners, either with or without intersecting welds, and field welding (Peterson, May 2006). Bridge # 9800 has an example of a welded horizontal stiffener.

Haunch insert

Very few haunch inserts are present in the Minnesota bridge inventory. Where they do occur, cracking may be present at their ends. An example of this was the cracking that occurred on the Lexington Avenue Bridge (35E over Mississippi River) (Wilson, May 2006). However, this cracking is most likely a result of distortional stresses and not in-plane stresses (See Figure 5.3 and Figure 5.4).

Figure 5.3 Haunch insert cracking - Lexington Avenue Bridge (Wilson, October 2006)

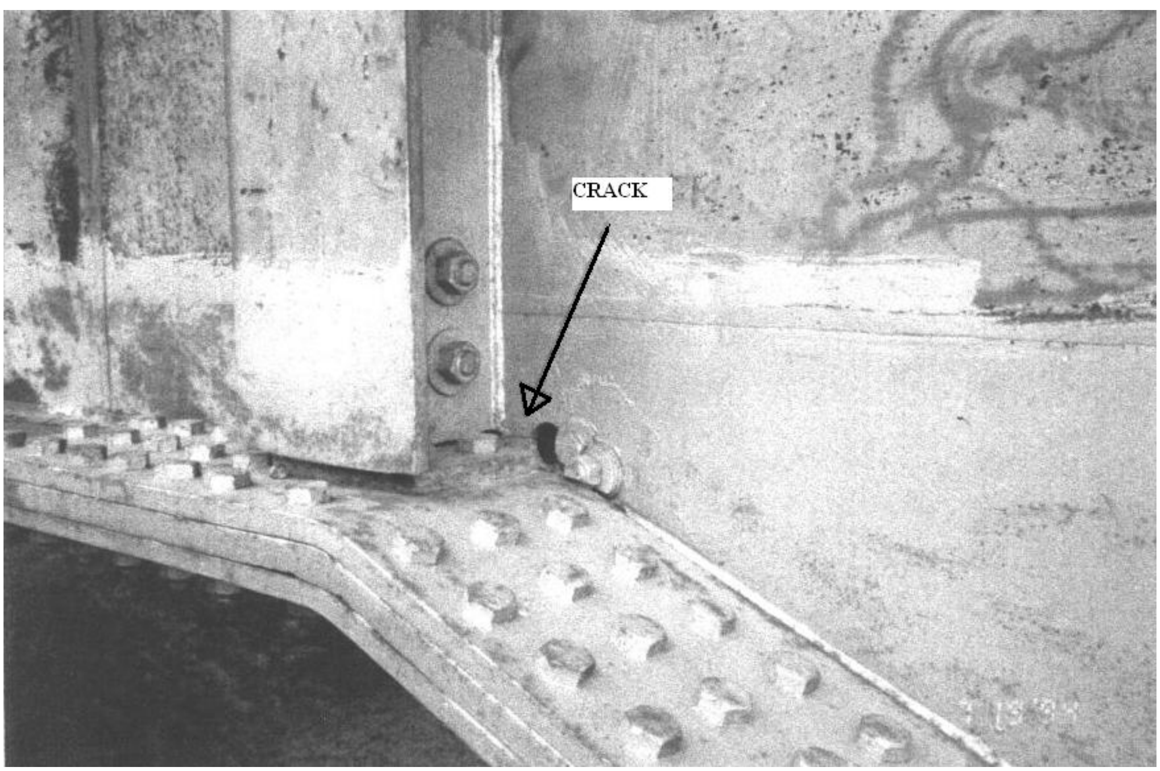


Figure 5.4 Haunch insert cracking (close-up) - Lexington Avenue Bridge (Wilson, October 2006)

Web penetration

Web penetrations in Minnesota bridges commonly occur in pier caps, where box girders intersect piers. Cracks have been found on both the inside and outside of box girders, at the welds connecting the web penetration (Figure 5.5 and Figure 5.6). Most of these cracks originate where backing bars are still in place.

Figure 5.5 Cracking at sealed web penetration - Bridge #69831 (Wilson, October 2006)

Figure 5.6 Cracking at coped web penetration - Bridge #69831 (Wilson, October 2006)

<u>Plug–welded misplaced hole</u>
Many Minnesota bridges contain plug welds in various locations, but they are difficult to identify on bridge plans. The best approach to quantify them is to record instances when they have been discovered by inspection. Although there have been noted cases of plug welds with flaws in them, there have been no cracks associated with this detail in Minnesota (Wilson, May 2006).

<u>Field-welded splice</u>
Cracks have been found in field welded splices of floor beams as seen in Figure 5.7. Occasionally, when a small bridge was widened, local bridge owners (e.g., municipalities and counties) welded the floor beams on-site with little care. Plans are unlikely to show these poorly-welded splices. About 50-60 bridges have this detail, the task is to identify those bridges having this detail.

Figure 5.7 Field welded splice – Bridge #90856 (Wilson, October 2006)

Eyebar and pin plate

About twenty Minnesota bridges contain eyebars or pin plates. They are all truss bridges generally built between 1880 and 1920 (Wilson, May 2006). A newer version of a pin and hanger connection was continued to be used until the early 1980's and cracks have appeared in these, usually due to corrosion.

Lateral bracing to girder bottom flange

Only one known case in Minnesota exists where braces are connected to the girder bottom flange instead of the web and no cracking has been discovered yet (Wilson, May 2006).

Cantilever floor-beam bracket

Only a handful of bridges in the Minnesota database have cantilever floor beam brackets. A couple of examples include the University of Minnesota Washington Avenue Bridge pedestrian walkway and the Wakota Bridge (#9360). Usually the detail is on a fracture critical bridge with a more intensive inspection schedule.

Backing bar

Backing bars were sometimes left in place until the 1978 Standards for Highway Construction Mn/DOT forced them to be eliminated. A fair number of cases have occurred with cracking initiating at backing bars; examples include shelf plates and pier cap interiors (Wilson, May 2006).

Intermittent weld
Bridges with intermittent welds are rare and have never shown any signs of cracking. The only known location of this type of weld in Minnesota is on bridge #6566 (Wilson, May 2006).

Tack weld
Tack welds are cause for many of the crack initiations classified under other details. Although they are rarely seen on bridge plans, they can be added to the database as they are found in the field. The degree of importance, due to the increased probability of cracking, must be considered in classifying this detail. For example, tack welds on tension members have a higher likelihood of cracking than on most other members.

Continuous longitudinal weld: box girder corner
Other than at pier caps, few Minnesota bridges have box girders with a possibility for corner cracking (Wilson, May 2006). Minnesota Bridges with welded box girders include: #27788, #27789, and #27791.

Cantilever: lamellar tear
There are no steel cantilevers in the Minnesota database believed to be able to produce lamellar tearing (Wilson, May 2006).

5.3 National Department of Transportation Survey

A survey was performed in this study in an effort to expand the scope of bridges from which cracks are enumerated and to identify problems that are not found in published sources. The information procured was used to identify additional details not contained in the survey as well as to estimate frequency of cracking. This survey was sent to bridge engineers from all fifty state DOTs, four bridge and toll authorities, the federal transportation authorities, and the Canadian provinces. The goal of the survey was to receive responses from ten DOTs and combine their replies for a more comprehensive background of problematic details. The results were better than expected, with sixteen responses and a wealth of new details that could be studied thoroughly in future research.

5.3.1 Survey Document

The following is the document which was sent to all 63 authorities. The first version of the survey was sent as an attachment with pictures included from those in section 0. However, the attachment size was too large and was rejected by 75% of the recipients, so the following in-email document was sent:

This email contains no attachments or pictures, to reduce the file size for those unable to open the previous survey. If you were able to open the attachment in the previous email, delete this message. Thank you to all of the DOTs who have already submitted their input into this study. Results should be turned in preferably by the end of July.

<u>PLEASE FORWARD THIS MESSAGE TO THE INTENDED PARTY</u>

Attn: Director of Inspections, Department of Transportation

Regarding: Steel Fatigue Details Research

Adam Lindberg
M.S. Bridge Fatigue Researcher
University of Minnesota
Civil Engineering Department

Research is being performed at the University of Minnesota with the sponsorship of the Minnesota Department of Transportation (Mn/DOT) to assist in enumerating and ranking fatigue-susceptible details that may affect the performance of steel bridges. The goal of this research is to rank details with a history of cracking, so as to alert inspectors and facilitate budget estimations. By collecting a comprehensive list of details prone to fracture, a more precise evaluation will result, thus safety is not compromised for the sake of economy.

If you are willing to assist the collection of these fatigue-susceptible steel bridge details, please fill out the following survey. Your time is very much appreciated. If you would prefer to provide information in a different format, please contact me at 763-607-6760 or lind0990@umn.edu (preferred).

Thank you,

Adam Lindberg

<u>Part 1</u>: Below are details that are known to have caused cracking. Please rank them by the order of occurrence experienced in your state or area, where 1 is the most common, up to the least common. If you have never experienced cracking (failure) of a certain detail, use 0.

<u>Part 2</u>: This section is more important than the first. Please provide *any* other details that have led to (premature) fracture and which are not included in this list. Please provide any such details even if they are no longer allowed by code or have been eliminated from your bridge inventory. Indicate the approximate number of cases of fracture as well as the factors that you believe affected the failure of this detail (Examples: tapering of ends, position on bridge, weld geometry). Do not include corrosion failures.

BEGIN NUMBERING NOW
(Non-picture version – for any further explanations, please contact Mr. Lindberg at lind0990@umn.edu)

_____*Transverse Stiffener Web Gaps* – Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to *both* flanges of the girders; instead a gap was left anywhere from 0” (bearing) to 2” or more.

Cracking can occur in any number of elements in this area, including the welds, the girder flange, the girder web, or the plate.

_____*Cover plated beams and flange gussets* –Cover plates are plates attached to the underside of girder flanges to increase the moment of inertia of girders locally. Cracks usually form at the ends of the cover plates, either in the cover plate, the weld, or the girder flange.

_____*Stringer to floor beam (truss) brackets* – Cracking occurs in the angles connecting floor beams to other elements. Cracks can occur anywhere within the angle connector, especially around bolts or rivets.

_____*Cantilever floor-beam brackets* – These plates are laid horizontally and usually bolted or riveted to the girders. They protrude out to the sides of the bridge and are connected to the top of short beams that extend out from the bridge to increase bridge width. Cracking can occur around tack welds, bolts, rivets, or across the plate parallel to girders.

_____*Web Connection Plates* – Horizontal plates welded to the girder web used for diaphragms or other attachments. Cracks usually occur when the plate intersects a transverse stiffener.

_____*Transverse Groove Welds* – Groove welds on girder webs. Usually these occur at the end of horizontal stiffeners, or welds connecting long sections of horizontal stiffeners together, or the end of haunch inserts (The bottom flange is cut out of the girder and replaced with a groove-welded, higher-depth section to increase moment of inertia around supports).

_____*Web penetrations* – When a member passes through the web of another. Cracking is more common when backing bars are left in place. These details are common in pier caps or box girders.

_____*Coped members* – Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually is caused by too small of a radius or no radius.

_____*Tied arch floor beams* – Floor beams can exhibit separation of beam web and flange due to rotation of the beams under distortional fatigue.

_____*Continuous longitudinal welds* – Commonly, these long welds connect plates or other shapes along their length, to form some sort of built-up-section. Cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds.

_____*Lamellar Tearing* – Separation of layers of metal *within* a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

Thank you very much for your assistance. Your input will help advance the overall understanding of fatigue susceptible details.

-Adam Lindberg

5.3.2 Survey Results

All who responded to the survey provided frequency results through their ordering of the given details and a few supplied extra details for Part 2 of the survey. Appendix C contains the actual response emails. Table 5.1 shows the results of those who responded, with the numbers inverted; i.e. if only numbers 1-5 were used, 1 was changed to 5, 2 to 4, 3 remained 3, 4 to 2, and 5 to 1. The reason for not using the preferred order in the survey request is the ease for the responding authorities to order details in a sequence from the most common to the least common, rather than the other way around. By inverting the order such that the highest numbers represent the most common failures makes the combination of survey results more straightforward.

To obtain an overall ordering, the "reversed" numbers found in Table 5.1 are added in each column to create a sum at the bottom of the table. These sums form the overall order of the details, but the values are meaningful only in relative sense. No additional point system is enacted to try to denote the incidence of individual details. In this way, details are ensured their order by scoring one more point than the next-most-frequent detail.

The symbol assigned to each detail corresponds to one of the 11 detail descriptions that were described in the survey letter:

α = transverse stiffener web gap
β = insufficient cope radius
γ = partial length cover plate
δ = shelf plate welded to girder web
ε = stringer to floor-beam truss bracket
ζ = transverse groove weld (welded horizontal stiffener & haunch insert)
η = web penetration
θ = cantilever floor-beam bracket
κ = continuous longitudinal weld: box girder corner
λ = tied arch floor-beam
μ = cantilever: lamellar tear

Table 5.1 Survey Results Table*

Responding Authorities	Detail Symbol**										
	α	β	γ	δ	ε	ζ	η	θ	κ	λ	μ
Arkansas	7	5	3	4	6	0	1	2	0	0	0
California	5	2	3	4	0	0	0	0	1	0	0
Delaware	2	3	4	0	0	1	0	0	0	0	0
Georgia	3	2	0	0	1	0	0	0	0	0	0
Illinois	5	6	3	2	4	0	1	0	0	0	0
Indiana**	6	5	4	3	1	2	0	0	0	0	0
Minnesota	9	6	8	5	7	4	3	2	0	1	0
Mississippi	4	3	0	2	1	0	0	0	0	0	0
Missouri	8	4	2	5	7	6	3	0	0	0	1
Montana	5	1	9	8	7	4	0	3	6	0	2
New Jersey	2	1	0	0	0	3	0	0	0	0	0
Tennessee	4	1	2	5	0	0	0	3	0	0	0
Texas	4	2	0	0	1	0	3	0	0	0	0
Washington	4	7	1	6	2	0	0	0	5	3	0
Wyoming	5	4	3	0	2	0	0	0	0	0	1
Army Corps-New England	0	0	1	0	0	0	0	0	0	0	0
TOTALS	**73**	**52**	**43**	**44**	**39**	**20**	**11**	**10**	**12**	**4**	**4**

*Numbers represent ordering by individual authorities, with largest number representing most frequent cracking
**implied by use of words, not numbers (i.e. few, some, many)

5.3.3 Additional Survey Comments

In response to Part 2 of the survey, which asks for any additional details not mentioned, quite a few states added comments. These ranged from more thorough descriptions of the ordered details to additional details. These, for the most part were not added to the classification table, Table 7.1, because they were either variations of those already included or almost no cases exist in Minnesota. Below are all of the responses in quotations.

Georgia had no further details to add. They did, however, supply the "approximate number of known bridges with the type of cracks over the last twenty years is as follows: Transverse stiffener web gaps – ten, coped members – two, stringer to floor beams – one."

Indiana supplied very thorough answers to each of the questions and did not provide extra details for considering adding to this report.

Transverse stiffener web gaps: “INDOT has had +/- 15 bridges, over the last 20-years, (mostly in the early 1990’s), that developed cracks similar to detail (a). Generally these cracks developed when the Stiffener had an X-Bracing attached, and the X-bracings were staggered due to skew, thus causing out-of plane bending cracking to develop. On advanced cracking, we would also get a horizontal crack along the toe of the weld between the web and the flange.”

Cover plates: “The great majority of the welded coverplates on bridges in Indiana are tapered. We have only had a few small cracks in the welds at the toes of the terminal ends of the welds. Purdue University conducted quite a bit of research on these coverplates for us in the 1990’s, and convinced us that the welds on these tapered coverplates will grow very slowly, thus giving our inspectors plenty of time to see and find them. They also developed a bolted retro-fit that we are using extensively on INDOT Bridges.”

Stringer to floor beam (truss) brackets: “We have had very few of these types of cracks on INDOT bridges. Most occurred in the 1980’s or earlier. These types of cracks are still found on some of our Local Bridges.”

Cantilever floor beam brackets: “We have not had this type of problem on any of our INDOT Bridges, and I have not heard of any on our Local Bridges.”

Web Connection Plates: “INDOT has had four of these types of cracks over the last 23-years, (1983{I-70}, 1985 {I-65}, 1994 {I-64}, and May 2006 {I-70}). I believe that the 1983 crack was 48" long, the 1985 crack was 21" long, the 1994 crack was 70" long, and the 2006 crack was 9.5" long. The first three cracks began around the Web/Transverse Stiffener/Connection Plate intersecting weld area. The 2006 crack began around the end of the weld of the Horizontal Connection Plate – away from the stiffener. All of these locations had X-Bracings and Lateral Bracings attached.”

Transverse groove welds: “INDOT has had three bridges with multiple cracks in Horizontal Web Stiffeners, all in the late 1990’s. All of these cracks developed in poor quality splice welds in the stiffener plates, and NOT in the welds to the girder webs.”

Web penetrations: “INDOT has not had this type of cracking, and we only have a few bridges with this type of detail.”

Coped members: “INDOT has had many of these types of cracks, but mainly in the 1980’s and 1990’s. We have not had much of this lately, on INDOT Bridges, but are probably still having this problem on our Local Bridges. Most of our cracking resulted from significant section loss to the web, above or below the connection plate.”

Tied arch floor beams: “INDOT has not had this type of cracking on our Tied Arch bridges. We have had a number of crack indications in our Tie-Chords on one bridge, mainly due to welds flaws and shrinkage during fabrication. Only a few of these have resulted in actual cracks.”

Continuous longitudinal welds: “INDOT has not had this type of cracking on the “Box Members” that make up the Tie-Chords of our Tied Arch bridges. We have had a number of crack indications in these welds, mainly due to welds flaws and shrinkage during fabrication, but no actual cracks.”

Lamellar tearing: “NONE -- INDOT does not have many of these types of details.”

“***Nevada***’s experience with fatigue damage has related mostly to out-of-plane bending due to perpendicular cross-frames in skewed bridges or interior “Z-bracing” in tub girders.” There are only a couple of steel box girder bridges in Minnesota, thus this detail would be too rare to classify (Peterson, September 2006). “Horizontal wind-bracing has been the next most prevalent cause of fatigue cracking, particularly where the horizontal connection plate has been welded particularly close to vertical web bracing and/or cross-frame connection plates. The remaining causes ranked [in the table] have had minimal occurrences and could virtually be ignored as significant causes of fatigue fractures.”

Illinois added a couple of comments that are covered already under longitudinal stiffeners, “Another detail which has experienced cracking is where the fillet weld connecting a longitudinal stiffener to girder web terminates at or near the fillet weld connecting a transverse (vertical) stiffener to the web.”

“In recent years, since the failure of the Hoan Bridge, this is thought to have been a brittle fracture that is the result of tri-axial constraint rather than the result of fatigue. However, I understand that there may still be some discussion as to whether this brittle fracture may occur, at least in some cases, when there is a pre-existing flaw in the weld or web material or when there is a very small fatigue crack present. I know this survey relates to fatigue, but for these reasons I mention this crack type here.” This information may be useful for future research on a more specific project.

“We have had webs crack where web connection plates, typically for lateral wind bracing connections, terminate at or are notched around transverse (vertical) web stiffeners. I have ranked this type of connection as #5 above. The cases we have experienced we believe to be brittle fracture similar to that of the Hoan Bridge rather than fatigue related; however, for the reasons stated above in our Part 2 response I have included it in our response.” This type of cracking is included in the web connection plate detail of the survey.

Washington supplied a lengthy list of details that it felt were not included:

A. “Modular expansion joints. We have experienced a number of failures in the center bar to support bar connection welds of some of these units. In the cases I have seen, the welds create fixity between these members and the relatively high flexibility of the system then causes significant connection moments to develop until the weld cracks and the rotational restraint goes away. Occasionally, failures in center bar groove weld splices have also occurred." Mn/DOT is aware of this issue and it is outside the scope of this project. If necessary, these details can be tracked already with Pontis element numbers (Peterson, September 2006).

B. "Open-metal grid decks. The welds connecting the intersecting bars of several designs have failed often. Weld quality in these usually secondary connections is sometimes an issue. But, it is thought that failure is largely due to the overall flexibility of the units coupled with significant impact loading leading to relatively high fatigue stress ranges in the under-designed and often poorly constructed welds." Although there are a couple examples of filled grid decks, their number does not justify tracking them (Peterson, September 2006).

C. "Fracture of component (channel) in a built-up riveted truss tension chord. (One occurrence.) Fracture most likely initiated at a punched rivet hole. Other cases were found before members fully fractured." This type of detail is rare, and not worth the trouble.

D. "Secondary truss members. Welded gusset plate connections are subjected to low stress, high cycle vibration. Such members have very little damping." Gary Peterson added a comment about this specific detail, saying, "I assume this would be a detail similar to having wind or X bracing angles welded to a horizontal shelf plate that is bolted or welded to a beam web. I don't think we have as much interest in secondary member connections, but we are interested in the shelf plate to web connection" which is included in a section of this report (Peterson, September 2006).

E. "Toe of welds where web stiffeners are welded to box girder bottom flanges. The relatively thin plates may be flexing out-of-plane under traffic." As mentioned before, there are very few box girder bridges.

F. "Riveted stringer-to-floorbeam brackets. We have several bridges where rivet heads have sheared off." This detail is beyond the scope of this research (Peterson, September 2006).

Montana

"We have had some very serious cracks occur in welded plate girders, which initiated at the intersection of or near intersection of fillet welds connecting longitudinal stiffeners and transverse stiffeners. The cracks were sudden and explosive starting near the weld intersections and heading up to very near the top flange and down and through a good portion (two-thirds) of the bottom flange." This detail is already covered under the longitudinal stiffener section of this report. "See a report entitled "Evaluation and Retrofit of Highway Bridges to Prevent Constraint-Induced Fracture From Web Attachments" by William J. Wright , Turner-Fairbanks Highway Research Center, and John W. Fisher, Robert Conner, Lehigh University ATLSS Center, to get a good description of what we feel happened."

“***Wyoming*** has experienced fatigue failures with two details not included in the survey. The attached structural drawings include these details.”

1) “The first is the bracket detail at columns E and E' as shown in the middle of the lower half of drawing RG870-D. This bracket supports a floor beam above a steel arch. It is attached to the steel arch by an angle section with a single row of bolts. Tension in the bracket due to movement of the floor beam created a prying action on the angle-to-arch connection which ultimately caused the angle to fracture through the single line of bolts.”

There are probably none of these details in Minnesota; if there is a similar circumstance, it is rare (Peterson, September 2006).

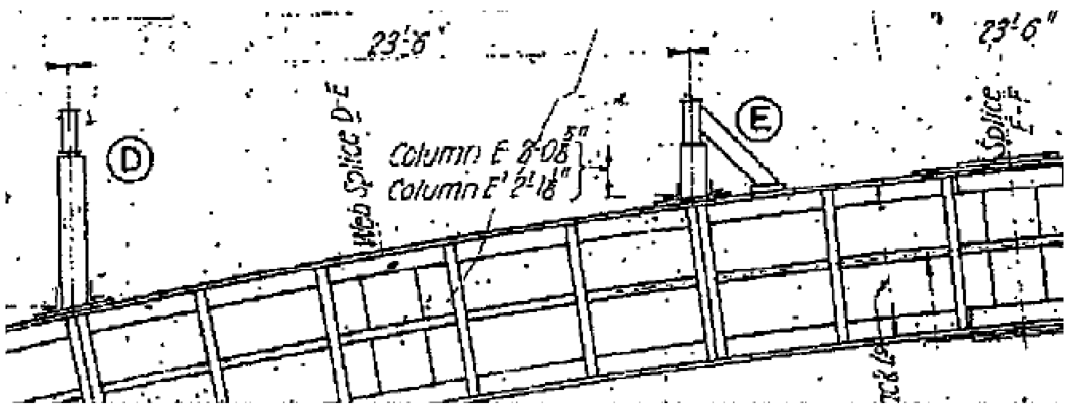


Figure 5.8 Wyoming DOT bracket detail – RG870-D (Fredrick, 2006)

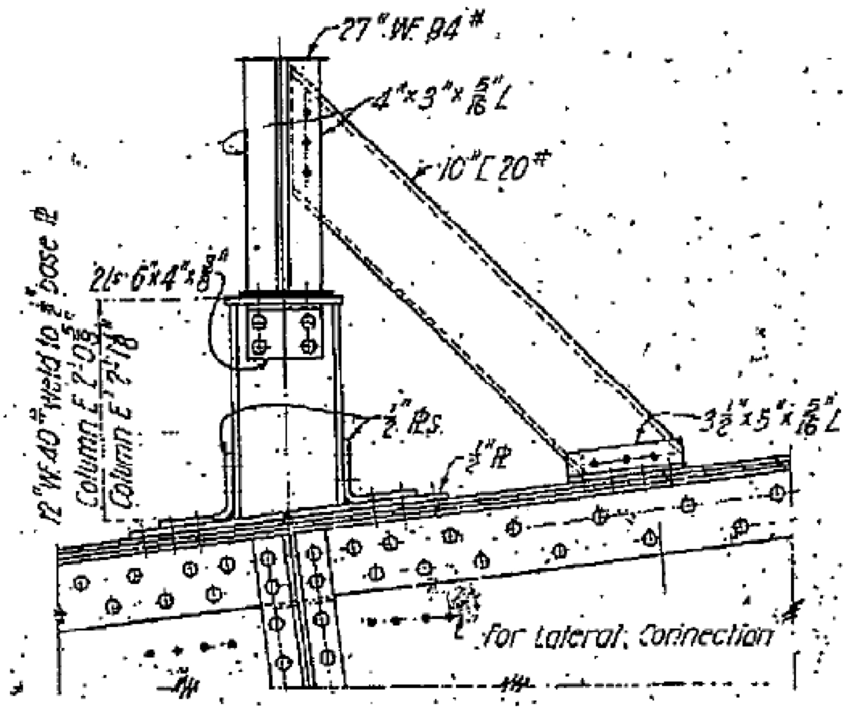


Figure 5.9 Wyoming DOT bracket detail member sizes – RG870-D (Fredrick, 2006)

2) “The second is a dog-bone-shaped hanger as pictured in the suspended span details of Drawing Number 2156. After some years in service, two of these hangers fractured across the width of the member where the round portion transitions to the straight sided shaft. It is believed that the geometrical transition of this member was abrupt enough to cause a stress concentration sufficient to fail the hanger. These hangers were replaced with units having straight sides for full length thereby eliminating the stress riser. The new hangers performed as required until the bridge was replaced many years later.”

These types of details are already classified under the eyebar and pin plate section.

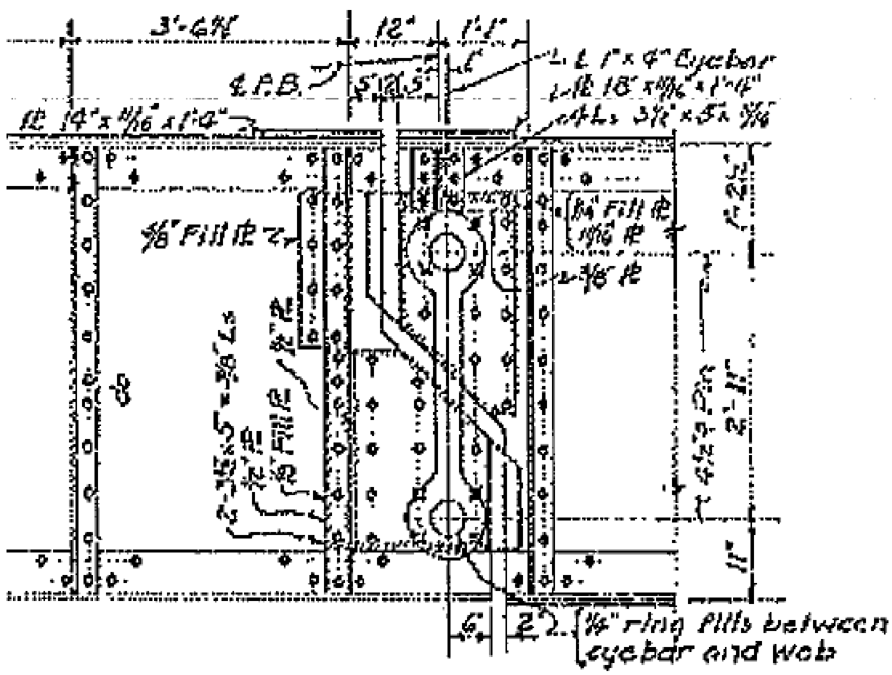


Figure 5.10 Wyoming DOT eyebar detail – 2156 (Fredrick, 2006)

CHAPTER 6 – CODE CHANGE TIMELINE

For the ease of compilation of susceptible details in the inventory, it is helpful to discern which bridges may have problems. Besides simply restricting the subset of bridges to those on the trunk highway system for traffic volume reasons, bridge construction date can further narrow the group. By determining dates that certain details were eliminated or restricted by design code, the compiler can ascertain which bridges to examine.

Over the years, the "Standard Specification for Highway Bridges" (1981) code was developed and refined by the American Association of State Highway and Transportation Officials (AASHTO). New editions of the code are developed anywhere from every three to six years. Interim specifications are updated between these, on an almost annual basis. A fatigue section of the code contains pertinent provisions. Investigation of this section as time progressed shows development as new problems were discovered.

6.1 General Code Timeline

The pertinent editions of the AASHTO standard specification that were examined for fatigue developments were: 1969 (10th Ed.), 1973 (11th Ed.), 1977 (12th Ed.), 1983 (13th Ed.), 1989 (14th Ed.), 1992 (15th Ed.), 1996 (16th Ed.), 2002 (17th Ed.). Interim specifications are created too often to track minor changes; furthermore, using only full edition specifications gives a good enough approximation for determining which bridges to examine. Only the interim specifications from 1974 and 1985 were scrutinized to better pinpoint the time of important changes.

The fatigue code in the AASHTO specification (Fatigue Code) for the most part had only slight modifications year to year, with the exception of two years of large modification. The largest change came in 1974, with the creation of the A-E fatigue stress categories and identification of fatigue life for new fatigue *S-N* curves, similar to those in Figure 2.1. Before then, the allowable fatigue stress, F_r was found by a less specific formula and different stress curves. The number of stress cycles for bridges were assumed at 2,000,000, 500,000, or 100,000 depending on bridge location. Each of these three cyclic levels, with separate levels of loading, had its own curve. Although the 1974 interim specification was the first to outline the new format, not until 1977 was the first, comprehensive, non-interim specification released. With the development of A-E categories, fatigue calculations were simplified and details were more specifically categorized.

The second big change took place in the 1989 AASHTO Specifications. That specification adopted many of the *S-N* categories and reclassified details to better match results that researchers were obtaining. The specific changes are outlined later in this chapter.

Other codes are important for tracking changes, especially those from which bridge design was actually guided. The first of these is the American Welding Society's (AWS) Structural Welding Committee code originating in 1963. In 1988, AWS and AASHTO combined to form the "Bridge Welding Code" which outlines specific weld requirements and other detail welding information. Also, The Minnesota Department of Transportation developed its own procedures in 1929 and published them as the "Minnesota Standard Specifications for Construction," which is updated anywhere from 4 to 11 years, plus interim specifications.

Because many bridges are designed or under construction while new versions of the code are released, there is a certain time necessary for actual bridges to reflect the changes. Dave Dahlberg of the Mn/DOT Bridge Office states, "You can assume about a 2 year lag time between any specification change and when the changes actually showed up in bridges being constructed (Dahlberg, May 2006)."

Only after 1977 were clear, modern classifications for fatigue evaluation used. Therefore, bridges built before 1979 (1977 code + 2 year lag) should be examined for all flagged details in this study.

6.2 Specific Code Timeline

Each of the following sections breaks down the precise changes to the wording for the AASHTO Code's fatigue sections. Figure 6.1 is taken from the 1977 Fatigue Code, expressing the entire fatigue provisions. Later sections describe changes made to these pages.

1.7.2—REPETITIVE LOADING AND TOUGHNESS CONSIDERATIONS

(A) Allowable Fatigue Stress

Members and fasteners subject to repeated variations or reversals of stress shall be designed so that the maximum stress does not exceed the basic allowable stresses given in Articles 1.7.1 and 1.7.41(B) and that the actual range of stress does not exceed the allowable fatigue stress range given in Table 1.7.2A1 for the appropriate type and location of material shown in Table 1.7.2A2 and illustrated in Figure 1.7.2.

Main load carrying components subjected to tensile stresses which may be considered nonredundant load path members—that is where failure of a single element could cause collapse—shall be designed for the allowable stress ranges indicated in Table 1.7.2A1 for Nonredundant Load Path Structures.

The range of stress is defined as the algebraic difference between the maximum stress and the minimum stress. Tension stress is considered to have the opposite algebraic sign from compression stress.

In Table 1.7.2A2 "T" signifies range in tensile stress only: "Rev." signifies a range of stress involving both tension and compression during a stress cycle.

TABLE 1.7.2A1
REDUNDANT LOAD PATH STRUCTURES (1)

Category See Table 1.7.2A2	Allowable Range of Stress, F_{sr} (ksi) (MPa)			
	For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For over 2,000,000 Cycles
A	60 (413.69)	36 (248.21)	24 (165.47)	24 (165.47)
B	45 (310.26)	27.5 (189.60)	18 (124.10)	16 (110.31)
C	32 (220.63)	19 (131.00)	13 (89.63)	10, 12* (68.95), (82.74)*
D	27 (186.16)	16 (110.31)	10 (68.95)	7 (48.26)
E	21 (144.79)	12.5 (86.18)	8 (55.15)	5 (34.47)
F	15 (103.42)	12 (82.74)	9 (62.05)	8 (55.15)

*For transverse stiffener welds on girder webs or flanges.

Figure 6.1 1977 AASHTO fatigue code (AASHTO, 1977)

NON REDUNDANT LOAD PATH STRUCTURES (2)

Category See Table 1.7.2A2	Allowable Range of Stress F_{sr} (ksi) (MPa)			
	For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	For over 2,000,000 Cycles
A	36 (248.21)	24 (165.47)	24 (165.47)	24 (165.47)
B	27.5 (189.60)	18 (124.10)	16 (110.31)	16 (110.31)
C	19 (131.00)	13 (89.63)	10, (68.95) 12*(82.74)	9, (62.05) 11*(75.84)
D	16 (110.31)	10 (68.95)	7 (48.26)	5 (34.47)
E	12.5 (86.18)	8 (55.15)	5 (34.47)	2.5 (17.24)
F	12 (82.74)	9 (62.05)	8 (55.15)	7 (48.26)

*For transverse stiffener welds on girder webs or flanges.

(1) Structure types with multi-load paths where a single fracture in a member cannot lead to the collapse. For example, a simply supported single span multi-beam bridge or a multi-element eye bar truss member has redundant load paths.

(2) Structure types with a single load path where a single fracture can lead to a catastrophic collapse. For example, flange and web plates in one or two girder bridges, main one-element truss members, hanger plates, caps at single or two column bents have non-redundant load paths.

TABLE 1.7.2A2

General Condition	Situation	Kind of Stress	Stress Category (See Table 1.7.2A1)	Illustrative Example (See Fig. 1.7.2)
Plain Material	Base metal with rolled or cleaned surfaces. Flame cut edges with ASA smoothness of 1000 or less	T or Rev.	A	1,2
Built-up Members	Base metal and weld metal in members without attachments, built-up of plates, or shapes connected by continuous full or partial penetration groove welds or by continuous fillet welds parallel to the direction of applied stress	T or Rev.	B	3,4,5,7
	Calculated flexural stress at toe of transverse stiffener welds on girder webs or flanges	T or Rev.	C	6

Figure 6.1 (Continued)

TABLE 1.7.2A2 (Continued)

General Condition	Situation	Kind of Stress	Stress Category (See Table 1.7.2A1)	Illustrative Example (See Fig. 1.7.2)
	Base metal at end of partial length welded cover plates having square or tapered ends, with or without welds across the ends	T or Rev.	E	7
Groove Welds	Base metal and weld metal at full penetration groove welded splices of rolled and welded sections having similar profiles when welds are ground flush and weld soundness established by nondestructive inspection.	T or Rev.	B	8,10,14
	Base metal and weld metal in or adjacent to full penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2 1/2, with grinding in the direction of applied stress, and weld soundness established by nondestructive inspection	T or Rev.	B	11,12
	Base metal and weld metal in or adjacent to full penetration groove welded splices, with or without transitions having slopes no greater than 1 to 2 1/2 when reinforcement is not removed and weld soundness is established by nondestructive inspection	T or Rev.	C	8,10,11,12,14
	Base metal at details attached by groove welds subject to longitudinal loading when the detail length, L, parallel to the line of stress is between 2 in. (50.8 mm) and 12 times the plate thicknesses, but less than 4 in. (101.6 mm)	T or Rev.	D	13

Figure 6.1 (Continued)

TABLE 1.7.2A2 (Continued)

General Condition	Situation	Kind of Stress	Stress Category (See Table 1.7.2A1)	Illustrative Example (See Fig. 1.7.2)
	Base metal at details attached by groove welds subject to longitudinal loading when the detail length, L, is greater than 12 times the plate thickness or greater than 4 in. (101.6 mm) long	T or Rev.	E	13
	Base metal at details attached by groove welds subjected to transverse and/or longitudinal loading regardless of detail length when weld soundness transverse to the direction of stress is established by nondestructive inspection			
	(a) When provided with transition radius equal to or greater than 24 in. (.610 m) and weld end ground smooth	T or R	B	14
	(b) When provided with transition radius less than 24 in. (.610 m) but not less than 6 in. (.152 m) and weld end ground smooth	T or R	C	14
	(c) When provided with transition radius less than 6 in. (.152 m) but not less than 2 in. (.051 m) and weld end ground smooth	T or R	D	14
	(d) When provided with transition radius between 0 in. and 2 in. (0 and .051 m)	T or R	E	14

Figure 6.1 (Continued)

TABLE 1.7.2A2 (Continued)

General Condition	Situation	Kind of Stress	Stress Category (See Table 1.7.2A1)	Illustrative Example (See Fig. 1.7.2)
Fillet Welded Connections	Base metal at intermittent fillet welds	T or Rev.	E	
	Base metal adjacent to fillet welded attachments with length, L, in direction of stress less than 2 in. (50.8 mm) and stud-type shear connectors	T or Rev.	C	13,15,16,17
	Base metal at details attached by fillet welds with detail length, L, in direction of stress between 2 in. (50.8 mm) and 12 times the plate thickness but less than 4 in. (101.6 mm)	T or Rev.	D	13,15,16
	Base metal at attachment—details with detail length, L, in direction of stress (length of fillet weld) greater than 12 times the plate thickness or greater than 4 in. (101.6 mm)	T or Rev.	E	7,9,13,16
	Base metal at details attached by fillet welds regardless of length in direction of stress (shear stress on the throat of fillet welds governed by stress category F)			
	(a) When provided with transition radius equal to or greater than 24 in. (.610 m) and weld end ground smooth	T or R	B	14
	(b) When provided with transition radius less than 24 in. (.610 m) but not less than 6 in. (.152 m) and weld end ground smooth	T or R	C	14

Figure 6.1 (Continued)

TABLE 1.7.2A2 (Continued)

General Condition	Situation	Kind of Stress	Stress Category (See Table 1.7.2A1)	Illustrative Example (See Fig. 1.7.2)
	(c) When provided with transition radius less than 6 in. (.152 m) but not less than 2 in. (.051 m) and weld end ground smooth	T or R	D	14
	(d) When provided with transition radius between 0 in. and 2 in. (0 and .051 m)	T or R	E	14
Mechanically Fastened Connections	Base metal at gross section of high-strength bolted slip resistant connections, except axially loaded joints which induce out-of-plane bending in connected material	T or Rev.	B	18
	Base metal at net section of high-strength bolted bearing type connections	T or Rev.	B	18
	Base metal at net section of riveted connections	T or Rev.	D	18
Fillet Welds	Shear stress on throat of fillet welds	Shear	F	9

(B) Load Cyles

The number of cycles of maximum stress range to be considered in the design shall be selected from Table 1.7.2B unless traffic and loadometer surveys or other considerations indicate otherwise.

Allowable fatigue stresses shall apply to those Group Loadings that include live load or wind load.

The number of cycles of stress range to be considered for wind loads in combination with dead loads, except for structures where other considerations indicate a substantially different number of cycles, shall be 100,000 cycles.

Figure 6.1 (Continued)

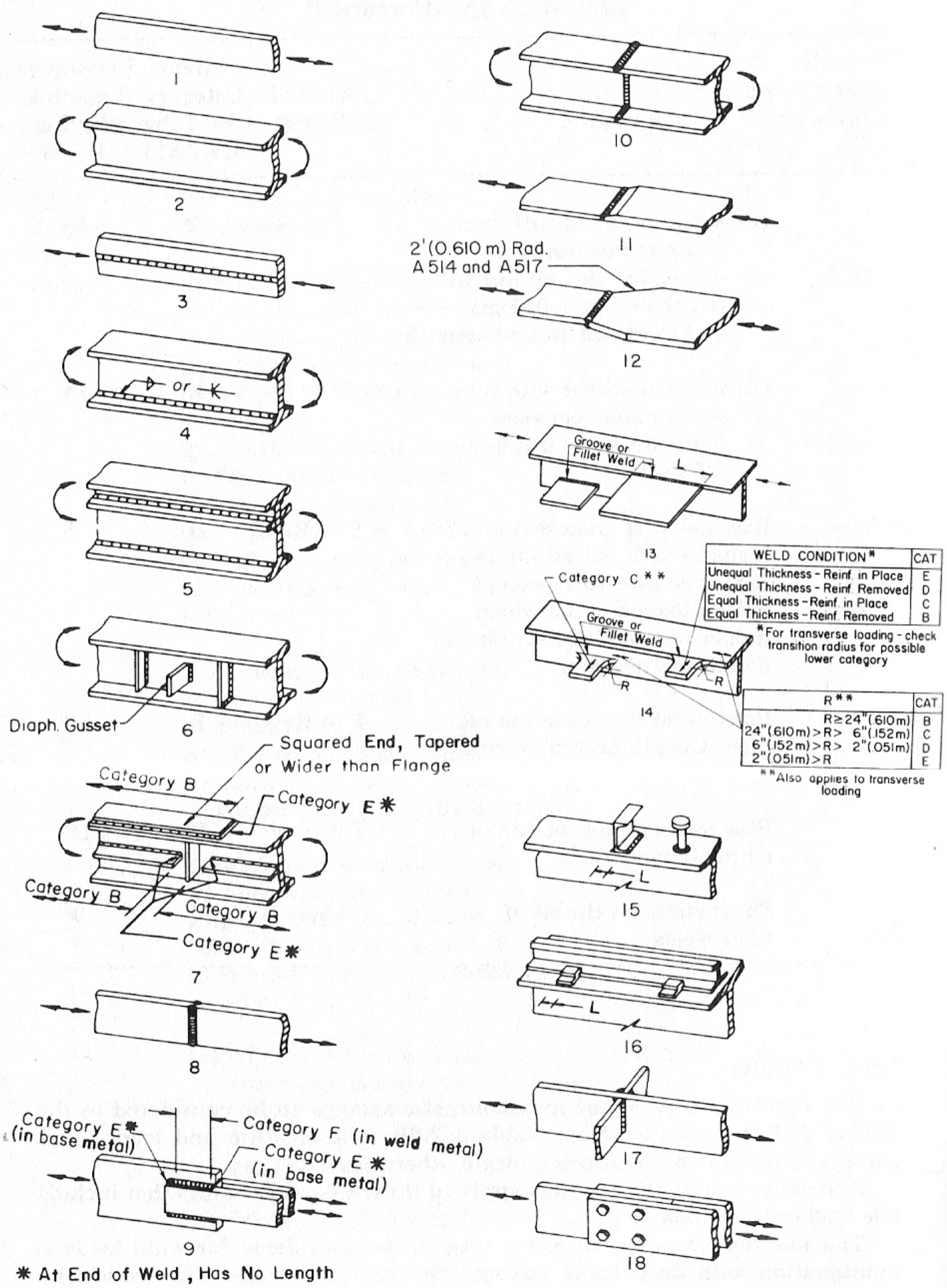


FIG. 1.7.2—Illustrative Examples

Figure 6.1 (Continued)

1.7.2 DESIGN 151

TABLE 1.7.2B—Stress Cycles

Main (Longitudinal) Load Carrying Members

Type of Road	Case	ADTT*	Truck Loading	Lane Loading†
Freeways, Expressways, Major Highways and Streets	I	2500 or more	2,000,000**	500,000
	II	less than 2500	500,000	100,000
Other Highways and Streets not included in Case I or II	III		100,000	100,000

Transverse Members and Details Subjected to Wheel Loads

Type of Road	Case	ADTT*	Truck Loading
Freeways, Expressways, Major Highways and Streets	I	2500 or more	over 2,000,000
	II	less than 2500	2,000,000
Other Highways and Streets	III	. . .	500,000

*Average Daily Truck Traffic (one direction).
†Longitudinal members should also be checked for truck loading.
**Members shall also be investigated for "over 2 million" stress cycles produced by placing a single truck on the bridge distributed to the girders as designated in Article 1.3.1(B) for one traffic lane loading.

Figure 6.1 (Continued)

6.2.1 Fatigue Code Changes 1977 to 1983

In 1983 category E' was added to describe a certain classification of coverplates which were slightly more susceptible to fatigue than category E. The following table and notes describe that change.

Allowable Fatigue Stress – Redundant Load Path Structures

Cycles	100,000	500,000	2,000,000	>2,000,000
E'	16 ksi	9.4 ksi	5.8 ksi	2.6 ksi

The note, "Partial length welded cover plates shall not be used on flanges more than 0.8 inches thick for nonredundant load path structures." was also added to the table.
The detail classification table (DCT) added to the cover plate section:

- (a) Flange thickness < 0.8 in = E
- (b) Flange thickness > 0.8 in = E'

Other changes that were made to the DCT include:

- Parts (a) through (d) for fillet weld transitions were condensed to only: (a) when provided with transition radius equal to or greater than 2 inches and weld ground smooth = D, or (b) less than 2 inches = E.
- A table note was added, stating, "Gusset plates attached to girder flanges with only transverse fillet welds is not recommended."

6.2.2 Fatigue Code Changes 1983 to 1989

In 1989 many changes were made to the AASHTO Bridge Code due to new research that was conducted mostly by Fisher (1984). This led to creation of a sub-classification of the B category, B' which is slightly more susceptible to fatigue. This new category was used to identify B details with backing bars left on and also more brittle material.

Allowable Fatigue Stress – Redundant Load Path Structures

Cycles	100,000	500,000	2,000,000	>2,000,000
B'	39 ksi	23 ksi	14.5 ksi	12 ksi

Allowable Fatigue Stress – Nonredundant Load Path Structures

Cycles	100,000	500,000	2,000,000	>2,000,000
B'	31 ksi	18 ksi	11 ksi	11 ksi
E'	12 ksi	7 ksi	4 ksi	1.3 ksi

Changes to the DCT include:

- The built-up members section distinguished between backing bars left on = B' and backing bars removed = B.
- The width of cover plates becomes important with, "Base metal at ends of partial length welded coverplates wider than the flange without welds across the ends" = E'
- An addition was made to the Groove Welded Connection section: "Base metal and weld metal in or adjacent to full penetration groove weld splices with 2 ft. radius transitions in width, when welds are ground flush with grinding in the direction of applied stress and weld soundness established by nondestructive inspection." = B.
- If A514/A517 base metal has a transition width or thickness, B' is used.

New in the DCT was a separation of the groove weld section into: groove welded connections, groove welded attachments (longitudinally loaded), and groove welded attachments (transversely loaded).

Longitudinally loaded groove welds:

- Added, "Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is less than 2 in." = C.
- E' is designated for detail thickness greater or equal to 1.0 in. for the longest of the detail lengths.

Transversely loaded groove welds:

- If reinforcement is not removed, the B category given for the 24 in. transition radius is not an option.
- If the plate thicknesses are unequal, both B and C categories are not applicable
- If reinforcement is not removed and thicknesses are unequal, only E applies.

Fillet welded connections:
Like the groove welds, fillet welds were separated into the same three categories.

Longitudinally loaded fillet welds:

- E' is designated if $t_{detail} \geq 1.0$ in. for the group with the longest of the detail length.

Transversely loaded fillet welds:
For welds perpendicular to the direction of stress:

- (a) Detail thickness ≤ 0.5 in. = C
- (b) Detail thickness > 0.5 in. = (Note*)

Note* Allowable fatigue stress range on throat of fillet welds transversely loaded in a function of the effective throat and plate thickness. $S_r = S_r^C \cdot \left(\frac{0.06 - 0.79\, H/t_p}{1.1 t_p^{1/6}} \right)$

Where S_r^C is equal to the allowable stress range for Category C.

Table notes:

- Transversely loaded partial penetration groove welds are prohibited.
- Gusset plates attached to girder flange surfaces with only transverse fillet welds are prohibited.

To amend the issue with distortional fatigue of the web gap, Section 10.20.1 on diaphragms and cross frames added the sentence, “Vertical connection plates such as transverse stiffeners which connect diaphragms or cross frames to the beam or girder shall be rigidly connected to both top and bottom flanges.”

6.2.3 Fatigue Code Changes 1989 to 1992

In 1992 the following paragraph was added: “For unpainted weathering steel, A709, all grades, the values of allowable fatigue stress range, Table 10.3.1A, as modified by footnote d, are valid only when the design and details are in accordance with the FHWA *Technical Advisory on Uncoated Weathering Steel in Structures*, dated October 3, 1989.” The stress range tables included alternate strengths for category A:

Allowable Fatigue Stress – Redundant Load Path Structures

Cycles	100,000	500,000	2,000,000	>2,000,000
A	(49) ksi	(29) ksi	(18) ksi	(16) ksi

Allowable Fatigue Stress – Nonredundant Load Path Structures

Cycles	100,000	500,000	2,000,000	>2,000,000
A	(39) ksi	(23) ksi	(16) ksi	(16) ksi

Groove welded connections:

- A514/A517 base metal set for the B' stress curve was changed to AASHTO M270 Grades 100/100W (ASTM A709).

Up until the 1992 code, re-entrant corners were designed with a ¾ inch filleted radius before cutting. In the 1992 code, no provision restricted cope radius.

6.2.4 Fatigue Code Changes 1992 to 1996

Built-up members:

- Base metal at ends of partial length welded coverplates with high-strength bolted slip-critical end connections. = B.

Eyebar or Pin Plates section (added):

- Base metal at the net section of eyebar head, or pin plate T E
- Base metal in the shank of eyebars, or through the gross section of pin plates with:
  (a) rolled or smoothly ground surfaces T A
  (b) flame-cut edges T B

6.2.5 Fatigue Code Changes 1996 to 2002

Composite action was addressed in 2002: "For members with shear connectors provided throughout their entire length that also satisfy the provisions of Article 10.38.4.3, the range of stress may be computed using the composite section assuming the concrete deck to be fully effective for both positive and negative moment."

A limit of the fatigue live load to HS 20 loading was added, so as to minimize error with fatigue curves that are not calibrated for larger loading. No changes were made to the fatigue classification table.

6.3 Changes to the "Minnesota Standard Specifications for Highway Construction"

The "Minnesota Standard Specifications for Highway Construction, 1964" published by the Minnesota Department of Highways, provides specific guidelines for construction procedures. It generally lags years behind the AASHTO code and sometimes even lag behind bridge design practice. Currently, this Department of Transportation "code" is much more consistent and up-to-date. This section outlines a few slight changes that are of interest to this study.

The 1964 specifications included a clause for the size and shape of re-entrant corners: "All interior and re-entrant corners shall be filleted whether shown in the Plans or not. A minimum radius of one inch shall be used unless a shorter radius is indicated in the Plans. Where fillets are made by the "flame cutting" method, all evidence of notching and edge hardening shall be removed. . . . . Fillets less than one inch in radius shall be formed by drilling the fillet corner and sheeting or "flame cutting" the balance of the re-entrant." (Minnesota Department of Highways, 1964) Since this wordage was adopted, no changes have been made to the specifications.

Removal of backing bars was left vague in the timeline of the AASHTO code. Changing the Curve from B to B' may have discouraged some use of remaining backing bars, but the B' category may have still been considered acceptable in many states. In Minnesota, practice actually preceded the AASHTO Code in this circumstance, as seen in the 1978 construction code:

"Butt welds made with the use of backing bars shall have the weld metal thoroughly fused with the backing when such backing is led in place to become part of the structure. Such backing is made as one continuous strip when possible. Where more than a single length is required such lengths shall be joined by completer penetration butt welds before fitting into place. Those welds shall be examined by radiographic testing prior to their use and shall be free of all defects . . . . Backing strips on weldments to be galvanized shall be seal welded. Backing strips to be removed shall be chipped or air-arc gouged without damage to the base material or the weld. Excess strip material remaining shall be completely removed by grinding. All surface roughness shall be ground smooth; all minor defects shall be faired-in; and all necessary welded repairs shall be ground flush with the base material."

6.4 Multiple Code Quick-reference Timeline

By combining all of the code changes from the current chapter, a general timeline can be formulated to efficiently identify what changes were made each year. The timeline is Table 6.1, and references various sources:

AASHTO – AASHTO Specifications for Highway Bridges
AASHTO Interim – Interim specifications for Highway Bridges
Dave Dahlberg – Information gathered from Dave Dahlberg at the Mn/DOT
Bridge Office through interviews
Minnesota Construction Code – Minnesota Standard Specifications for Highway
Construction

Table 6.1 Quick-Reference Timeline

1964	1" radius required for re-entrant corners. (Minnesota Construction Code)
1974	Outlining of new A-F classification system. Vague list of details with their classification letter. Diagrams showing fatigue details are included. (AASHTO Interim)
1977	More complete A-F classification with more specific details listed. This code developed the basic fatigue calculation procedure used today. (AASHTO)
1978	Backing bars must be removed. (Minnesota Construction Code)
1983	Category E' for cover plates on redundant flanges thicker than 0.8". Banned on thick flanges when nonredundant. Categories B and C for transitioned fillet welds were eliminated. Gusset to flange with transverse fillets not recommended. (AASHTO)
1985	Welds required between diaphragm connection plates and girder flanges. (AASHTO Interim)
1986	Cover plates stopped being used by Mn/DOT. (Dave Dahlberg - Mn/DOT Bridge Office)
1989	Cover plates wider than flange without welds across ends - E'. B' added for backing bars left on. Groove welded splices with large transitions - B. A514/A517 metal used with transitions - B'. Groove and fillet welded connections organized by direction of stress. Longitudinal groove welds < 2" - C. Categories with longest groove or fillet welded detail length & thicker than 1" - E'. B eliminated when reinforcement is left for transverse loaded groove welds. For unequal plate thicknesses B & C were eliminated. Unequal & reinforcement left - B, C & D were eliminated. Transverse fillets & plate thickness> 0.5" stress is a function of thickness. Transverse loaded partial penetration groove welds are prohibited. Filleted gusset to flange is prohibited. (AASHTO)
1992	An alternate stress category A was added for unpainted weathering steel. Base metal corresponding to B' was changed to AASHTO M270 Grades 100/100W. Re-entrant corners with fillets <3/4" were eliminated. (AASHTO)
1996	Welded cover plates with slip-critical end connections given B classification. Added: net section of eye bar head and pin plates = E, metal in smooth eye bar shanks = A, flame-cut shanks = B. (AASHTO)
2002	Composite action of deck can be considered in calculations. A limit of HS20 truck must be used for stress calculations. (AASHTO)

CHAPTER 7 – DETAIL CLASSIFICATION

By using the information gathered in Chapters 2 – 6, a finalized classification scheme can be developed. This classification involves the fatigue and fracture backgrounds presented in 0 and 0 by both discovering which details should be flagged and providing the basis for risk of catastrophic failure. 0 provides further insight into the effect of geometric variations on risk level and frequency of cracking. 0 uses historical data to organize details based on their rate of occurrence. Lastly, 0 illustrates the time in which cracking occurred; in doing so, it restricts the number of bridges which need to be considered for classification.

7.1 Ranking Number

Ranking is the last piece required to complete the classification. The ranks of details are developed in the current chapter and the method is described herein. The details described previously will be ranked in order from 1 to 4, with 1 representing the least potential for performance problems or failure, and 4 representing the most. Details ranked 0 are the recommended design, showing no previous fatigue of fracture issues. The gross categorization levels are as follows:

0 – No issues, recommended use
1 – Low consequence of cracking, low frequency of occurrence
2 – Low consequence of cracking, high frequency of occurrence
3 – Medium consequence of cracking
4 – High consequence of cracking

The reason for ranking by these categories is the application for which the numbers will be used. Details ranked with a 4 will be closely watched to guard against unpredictable failures. Those in rank 3 or higher may be replaced when bridge retrofit is performed. Rank 2 or higher details can be avoided in new construction and will be monitored for propagation and inspected during routine inspections. The least problematic, rank 1, has been known to create cracking with little effect on the structure. These details are noted for quick reference if more serious problems occur for a specific detail type. It also allows bridges that have numerous types of rank 1 details to be compared to ones that have fewer.

Rank numbers are assigned qualitatively, with no numerical frequency of occurrence, or fatigue cycles before failure. However, the ranking is built upon a plethora of sources, providing a many-faceted background for its categorization.

For a detail to earn the most severe of the rankings an unpredictable nature must have been exhibited in past instances. Most of these details are contained in the fracture section of this report, but a few fatigue examples have created cracks large enough to significantly impair the strength of the bridges. When large cracks are found it usually implies rapid growth, since the previous inspection likely noted few or no flaws. Besides the rapidity of cracking, detail placement plays a role. Although recording specific placement of every detail is too laborious for this study, future work could be done to pinpoint location effects. All in all, historical examples create the basis for identifying rank 4 details.

Rank 3, or medium danger is the most ambiguous of the groups and its scope defines the limits for ranks 4 and 2. The group acts as more of a transition region between details with serious problems and those which would likely not lead to catastrophic failure, regardless of time. In almost every circumstance, cracks are discovered and repaired before they have the opportunity to cause serious problems. Thus, a judgement must be made as to the likelihood of dangerous failure.

Like the most severe details, those with a ranking of 3 are classified based on history. If cracking was discovered in a detail that "may" have propagated over time to be dangerous to a structure, it is given rank 3. As seen in the classification table (Table 7.1), rank 3 is commonly the most severe rank in details which are further broken down based on geometry. The worst combination of geometry yields higher ranking than details combining all problematic features.

Details that cause minor cracks are more common than severe cracks; therefore, separating the minor ones by frequency of occurrence is appropriate. Ranking levels 2 and 1 represent more and less frequent cracking, respectively. Again a line must be drawn between the two ranks; it separates common and independent identification of problems. 0, containing the Minnesota Department of Transportation history and a survey of other DOTs, describes an ordering of details that occur on a common basis. Details that many DOTs recognized are considered "common" and given a rank of 2.

Details ranked 2 comprise the majority of geometrically sub-classified details because their high frequency allowed for further classification. One of the most complicated separations between frequent and rare cracking was the web gap. Using Equation 2.11, the web gap stress was computed for various combinations of web thickness, web gap length, span length, and skew. The separation was drawn as seen in Table 7.1, to accommodate any bridges with "more susceptible than average" geometries into rank 2. More susceptible geometries include a high web thickness or skew angle, or a small gap or span length. It turned out, using an arbitrary coefficient of 2.5 as explained in Section 0, the separation between rank 1 and 2 occurred between stresses of 12.9 and 14.5 ksi. A list of calculations can be seen in APPENDIX B.

Rank 1 depicts details that are neither dangerous enough, nor common enough to be classified in other categories. The reason for classifying them at all is that they do exhibit cracking. In many instances these details will crack in combination with secondary effects, such as corrosion or initial flaws. Also, since the separations between this rank and either 2 or 3 is somewhat ambiguous, it is important to still identify rank 1 details rather then ignore them. Furthermore, future circumstances could lead to the changing of rank numbers.

7.2 Classification Table

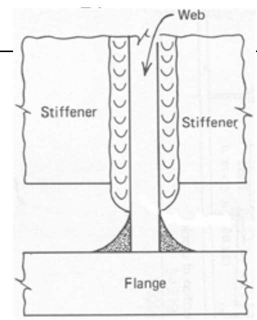
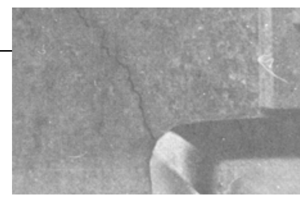
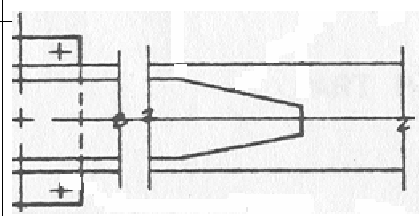
The following classification table is a list of details for application into the Minnesota Department of Transportation database. Each picture included in the table is a general picture of the detail and different geometries occur. The date of prohibition is the date in which either a code or the standard method for Mn/DOT construction was altered to exclude the specific detail. Years with an "r" designation represent restrictions, which may have discouraged a detail, noticed a problem with a detail, or changed the AASHTO fatigue category to decrease use. For more specific timeline information, including what restrictions were added, see 0.

The following designations represent the source from which the change took place:
AASHTO – American Association of State Highway and Transportation Officials: *Specifications for Highway Bridges*
MN – Minnesota Department of Highways: *Standard Specifications for Highway Construction*
Fisher – Research by John Fisher, see Section 0 for details
P – Pierce, James: Mn/DOT specific practices email correspondence
W – Pete Wilson (last known case in a Minnesota bridge)
r – Use restricted due to realization of problem, change in S-N category, or recommended procedure; not complete prohibition of detail.
xxxx – Still in use, no code restrictions

Definitions for variables used in the classification table are as follows:
t_w = thickness of girder web
L = span length
skew = angle of bridge relative to supports
Mn/DOT Std = Minnesota stiffener welding standard (see Figure 4.3)
L_p = length of welded side of plate
t_p = thickness of plate
R = transition radius
PJP = partial joint penetration weld

To find a rank number from the table, read each geometric constraint for a certain detail, starting with the top and working down. The first category to satisfy all geometry is the correct one. Constraints separated by commas must all be satisfied.

Table 7.1 Overall Classification Table

Detail Name	Description		Date of code prohibition	Geometric Partitions	Rank #
Web Gap on Diaphragm Transverse Stiffener (excluding intermittent stiffener)	Gap usually from tight fit to 2" between vertical stiffener and bottom or top flange of girder where no welded or bolted connection exists		1989	top flange, *tw*>0.50", web gap<2", *L*<80'	2
			1989	top flange, *tw*>0.25", web gap<1", *L*<80'	2
			1989	top flange, *tw*>0.50", web gap<2", *L*<160', *skew* > 40º	2
			1989	top flange, *tw*>0.25", web gap<1", *L*<160', *skew* > 40º	2
			1989	bottom flange (positive moment region)	1
			1989	Otherwise	1
Insufficient Cope Radius	Re-entrant cope radius is nonexistent, too small, or not ground smooth.		1964 MN	Over-cut or square-cut	2
			1964 MN	Transition radius < 1"	1
			xxxx	Transition radius ≥ 1"	0
Partial Length Cover Plate	Plates welded to top or bottom girder flange to increase moment capacity locally.		1983r AASHTO	Flange thickness > 0.8"	3
			1986 MN	No end taper, wider than flange, no weld across end	3
			1986 MN	No end taper, other cases	2
			1986 MN	End taper	1

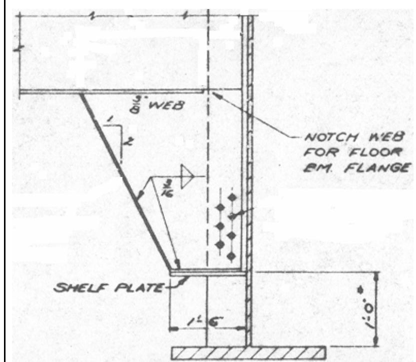
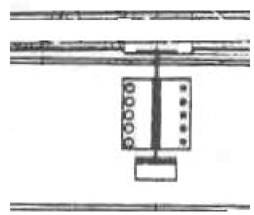
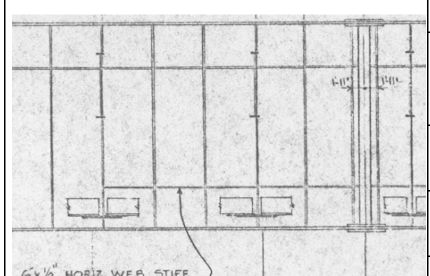
<table>
<tr><th>Detail Name</th><th>Description</th><th>Picture</th><th>Date of code prohibition</th><th>Geometric Partitions</th><th>Rank #</th></tr>
<tr><td rowspan="5">Shelf Plate Welded to Girder Web</td><td rowspan="5">Horizontal plate welded to inside face of girder web used for floor beam or lateral bracing connections. Intersecting plate is either coped around or welded to shelf plate.</td><td rowspan="5">
</td><td>2002 MN</td><td>Intersecting plate, cope and weld not compliant with Mn/DOT Std</td><td>4</td></tr>
<tr><td>1989r AASHTO</td><td>Intersecting plate, cope and weld compliant with Mn/DOT Std, (Lp>12tp or 4") or R<6"</td><td>3</td></tr>
<tr><td>xxxx</td><td>Intersecting plate, cope and weld compliant with Mn/DOT Std, (Lp≤12tp & 4") and R≥6"</td><td>2</td></tr>
<tr><td>1989r AASHTO</td><td>No intersecting plate, (Lp>12tp or 4") or R<6"</td><td>2</td></tr>
<tr><td>xxxx</td><td>No intersecting plate, (Lp≤12tp and 4") and R≥6"</td><td>1</td></tr>
<tr><td rowspan="2">Stringer or Truss Floor-beam Bracket</td><td rowspan="2">Angle bracket connecting stringers to floor beams or floor beam to truss.</td><td rowspan="2"></td><td>xxxx</td><td>Floor beam to truss</td><td>3</td></tr>
<tr><td>xxxx</td><td>Stringer to floor beam</td><td>1</td></tr>
<tr><td rowspan="5">Welded Horizontal Stiffener</td><td rowspan="5">Horizontal plates welded continuously to girder webs.</td><td rowspan="5">
</td><td>1960r Fisher</td><td>Continuation through tension zone</td><td>4</td></tr>
<tr><td>1960r Fisher</td><td>Multiple plates welded end-to-end or vertical stiffener interrupts horizontal stiffener</td><td>3</td></tr>
<tr><td>1989r AASHTO</td><td>R=0, tp>1"</td><td>3</td></tr>
<tr><td>1989r AASHTO</td><td>R<2"</td><td>2</td></tr>
<tr><td>xxxx</td><td>R≥2"</td><td>1</td></tr>
</table>

Table 7.1 (Continued)

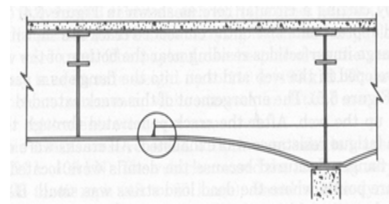
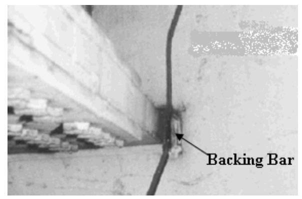
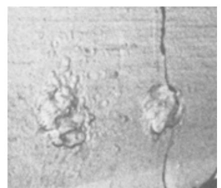
Detail Name	Description	Picture	Date of code prohibition	Geometric Partitions	Rank #
Haunch Insert	Bottom girder flange and part of web removed and section added to increase girder depth locally. Welded.		1989 AASHTO	Transverse *PJP* groove welds	3
			xxxx	No transverse *PJP* groove welds	2
Web Penetration	A hole cut through the web of member to allow another plate or member to pass through.		xxxx	Hole cut in box girder except at pier cap	3
			xxxx	Intersecting girders at pier cap	2
			xxxx	Hole cut in open girder	1
Plug-Welded Misplaced Hole	Misplaced holes filled by plug welding, regardless of location.		xxxx		4
Field-Welded Splice	Splice between girders, beams or girder to beam which was done by local bridge owners.		xxxx		4

Table 7.1 (Continued)

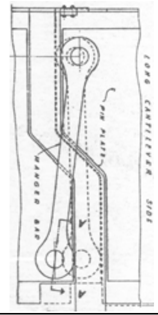
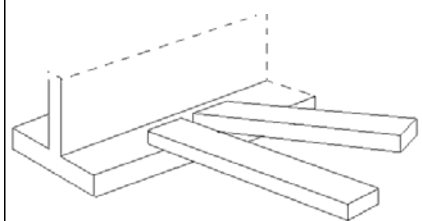
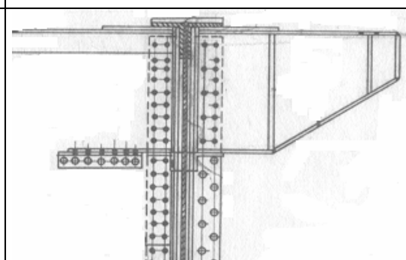
Detail Name	Description	Picture	Date of code prohibition	Geometric Partitions	Rank #
Pin and Eyebar Truss and Pin and Hanger Assembly	Eyebars or pin plates used as tension members.		1989 P		4
Lateral Bracing to Girder Bottom Flange	Lateral bracing such as cross-braces or diaphragms welded to the bottom flange of girders and not the web.		1983r 1989 AASHTO		3
Cantilever Floor-beam Bracket	Plate lain horizontally over beam top flange cantilevered to side. Bolted or welded to top flange of each member.		1986 W		3

Table 7.1 (Continued)

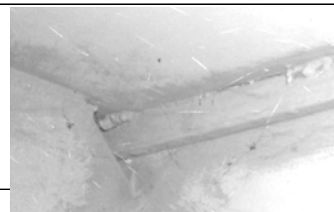
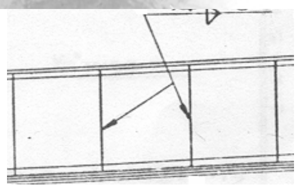
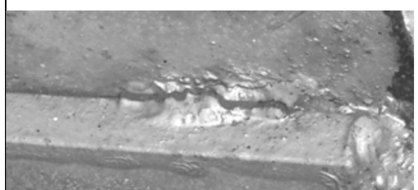
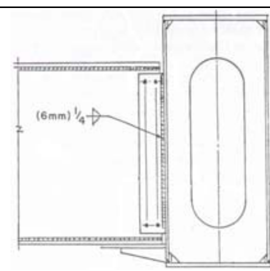
Detail Name	Description	Picture	Date of code prohibition	Geometric Partitions	Rank #
Backing Bar	Bars or plates left on bridges used for supporting back of welds, not ground smooth.		1978 MN		3
Intermittent Weld	Welding along a section such as vertical stiffener that is not continuous.		xxxx		3
Tack Weld	Bolts, plates or any other members that were tack-welded into place without removing welds afterwards.		1970 P		2
Tied Arch Floor-beam	Floor-beam shear connection to tied arch girder web		xxxx		1
A514 Steel (T1 steel)	Low fracture toughness steel		1989r AASHTO		3

Table 7.1 (Continued)

CHAPTER 8 – RANKING PROGRAM

8.1 Overview

To utilize the classification table from 0 a program was developed using Microsoft EXCEL, which incorporates the various geometric properties to determine appropriate rank numbers. This program also computes a composite rank number for the bridge, which enables bridge owners to list bridges which may be considered for repair or retrofit. The output from the program shows all of the information that was entered. This allows anyone examining the database to find the most severe detail of each type immediately by examining the data string for each bridge. Ultimately, the program is a tool which applies research to literally and directly accomplish the bridge ranking.

The composite (bridge) rank number is a sum of the rank numbers for the most severe of each detail type. The maximum possible bridge rank number is 54, representing the worst detail for each of the 18 types present in one bridge. The lowest possible bridge rank number is 0, for a bridge having none of the 18 types of susceptible details. The majority of bridges are expected to rank below 12, with new bridges having lower rank numbers.

For ease of use, a blank copy of the program interface can be printed and used as a questionnaire in which geometric constraints from each detail can be written. This allows anyone examining bridge plans or performing a field inspection to provide data that is easily entered into the ranking program.

8.2 Program Operation

Simple macros are developed using "If" and "Else" statements in Microsoft EXCEL to properly categorize each detail. "N/A" output is explained in the following paragraph. The input variables, i.e. 1A, 1E, etc. are shown in Figure 8.1, the input interface.

If 1=Y:

If 1A=bot …………………………………………………………………… 1
Else If 1E>0.5 And 1D<2 And 1C<80 ……………………………………...... 2
Else If 1E>0.25 And 1D<1 And 1C<80 ………………………………. 2
Else If 1E>0.5 And 1D<2 And 1C<160 And 1B>40 ……….... 2
Else If 1E>0.25 And 1D<1 And 1C<160 And 1B>40 ... 2
Else ……………………………………………….. 1

If 2=Y:

If 2A=N …………………………………………………………………... 2
Else If 2B<1…………………………………………………………. 1
Else …………………………………………………………………….. 0

If 3=Y:

If 3A>0.8 ………………………………………………………………… 3
Else If 3C=Y ……………………………………………………………... 1

Else If 3D>3B And 3E=N …… 3
Else …… 2

If 4=Y:

If 4B=Y If 4A=N …… 4
Else If 4C>12*4D …… 3
Else If 4C>4 …… 3
Else If 4E<6 …… 3
Else …… 2
Else If 4C>12*4D …… 2
Else If 4C>4 …… 2
Else If 4E<6 …… 2
Else …… 1

If 5=Y:

If 5B=Y …… 3
Else If 5A=Y …… 1
Else …… N/A

If 6=Y:

If 6A=Y …… 4
Else If 6B=Y …… 3
Else If 6D<0.1 And 6C>1 …… 3
Else If 6D<2 …… 2
Else …… 1

If 7=Y:

If 7A=Y …… 3
Else …… 2

If 8=Y:

If 8A=Y …… 3
Else If 8C=Y …… 2
Else If 8B=Y …… 1
Else …… N/A

IF 9=Y: …… 4
IF 10=Y: …… 4
IF 11=Y: …… 4
IF 12=Y: …… 3
IF 13=Y: …… 3
IF 14=Y: …… 3
IF 15=Y: …… 3
IF 16=Y: …… 2
IF 17=Y: …… 1
IF 18=Y: …… 3

BRIDGE NUMBER ___________

DETAIL NAME	INPUT	INPUT TYPE	WORST CASE	RANK NO.
Transverse Stiffener Web Gap	1	(Y/blank)	Y	0
Gap flange	1A	(top/bot)	top	
Bridge span skew	1B	(deg)	↑	
Span length containing web gap	1C	(ft)	↓	
Web gap height	1D	(in)	↓	
Girder web thickness	1E	(in)	↑	
Insufficient Cope Radius	2	(Y/blank)	Y	0
Is there a cope radius	2A	(Y/N)	N	
Cope radius (0 if none)	2B	(in)	↓	
Partial Length Cover Plate	3	(Y/blank)	Y	0
Girder flange thickness	3A	(in)	↑	
Girder flange width	3B	(in)	↓	
Is the end tapered	3C	(Y/N)	N	
Cover plate width	3D	(in)	↑	
Is there a weld across end of cover plate	3E	(Y/N)	N	
Shelf Plate Welded to Girder Web	4	(Y/blank)	Y	0
Weld termination and cope distances within Detail B41 allowable dim.	4A	(Y/N)	N	
Is any plate intersected or coped around shelf plate	4B	(Y/N)	Y	
Length of shelf plate	4C	(in)	↑	
Thickness of shelf plate	4D	(in)	↓	
Shelf plate transition radius	4E	(in)	↓	
Stringer or Truss Floor-beam Bracket	5	(Y/blank)	Y	0
Floor beam connected to stringer	5A	(Y/N)	Y	
Floor beam connected to truss	5B	(Y/N)	Y	
Welded Horizontal Stiffener	6	(Y/blank)	Y	0
Horizontal stiffener continues through tension zone	6A	(Y/N)	Y	
Multiple plates welded end-to-end or interrupted by vertical stiffener	6B	(Y/N)	Y	
Thickness of horizontal stiffener	6C	(in)	↑	
Horizontal stiffener transition radius	6D	(in)	↓	
Haunch Insert	7	(Y/blank)	Y	0
Connected using transverse PJP groove welds	7A	(Y/N)	Y	
Web Penetration	8	(Y/blank)	Y	0
Hole in box girder except at pier cap	8A	(Y/N)	Y	
Hole in open girder	8B	(Y/N)	Y	
Intersecting girders at pier cap	8C	(Y/N)	Y	
Plug-Welded Misplaced Hole	9	(Y/blank)	Y	0
Field Welded Splice	10	(Y/blank)	Y	0
Pin and Eyebar Truss or Pin and Hanger Assembly	11	(Y/blank)	Y	0

Lateral Bracing to Girder Bottom Flange	12	(Y/blank)	Y	0
Cantilever Floor-Beam Bracket	13	(Y/blank)	Y	0
Backing Bar	14	(Y/blank)	Y	0
Intermittent Weld	15	(Y/blank)	Y	0
Tack Weld	16	(Y/blank)	Y	0
Tied Arch Floor-Beam	17	(Y/blank)	Y	0
A514 Steel (T1 Steel)	18	(Y/blank)	Y	0
,			Bridge Rank	0

Figure 8.1 Program Input Variables

The units or Y/N symbols defined in parenthesis show what parameters to fill in each blank in the "INPUT" column. Once the user notes the detail exists (places a Y next to the detail), all the other blanks must be filled in for that detail. It is important to keep all cells other than the INPUT column locked, so that the formulas cannot be altered. If any "N/A" terms show up in the output or the bridge rank number is not computed, then either the detail should not be classified as a fracture or fatigue susceptible detail, or an input was not in the correct units, i.e. a "Y" was placed in a slot where a length was required.

The program allows someone to input the data directly or collected from a paper copy into a single column. Then the rank number for each detail as well as the entire bridge is shown. The reason for filling in all data for each of the present details is so the program will run properly. Furthermore, the most severe geometry for each of the 18 details should be used, resulting in the highest rank number. The most severe situations are found in the column "WORST CASE." If it so happens that a detail has a geometric constraint worse than a second of the same type, and the second has a different geometric constraint worse than the first, then judgment must be made using the classification table to determine which one will result in the highest rank number.

The output of the program is a single, comma-delineated line of information which can be easily transferred to databases as a text file. The format for the output line is as follows: Bridge rank #, Detail 1 rank #, Detail 2 rank #, . . . , Detail 18 rank #, Input 1 (Transverse Stiffener Web Gap), Input 2 (Gap flange), . . . , Input 45 (A514 Steel). If no data is input, the corresponding "Input" places will be blank, i.e. commas will be directly next to each other. If no data is input, the line will still give a rank number of 0, indicating that there are no noted cracking issues for that detail. The program follows with an example in Figure 8.24.

BRIDGE NUMBER ___________

DETAIL NAME	INPUT	INPUT TYPE	WORST CASE	RANK NO.
Transverse Stiffener Web Gap		(Y/blank)	Y	0
Gap flange		(top/bot)	top	
Bridge span skew		(deg)	↑	
Span length containing web gap		(ft)	↓	
Web gap height		(in)	↓	
Girder web thickness		(in)	↑	
Insufficient Cope Radius		(Y/blank)	Y	0
Is there a cope radius		(Y/N)	N	
Cope radius (0 if none)		(in)	↓	
Partial Length Cover Plate		(Y/blank)	Y	0
Girder flange thickness		(in)	↑	
Girder flange width		(in)	↓	
Is the end tapered		(Y/N)	N	
Cover plate width		(in)	↑	
Is there a weld across end of cover plate		(Y/N)	N	
Shelf Plate Welded to Girder Web		(Y/blank)	Y	0
Weld termination and cope distances within Detail B41 allowable dim.		(Y/N)	N	
Is any plate intersected or coped around shelf plate		(Y/N)	Y	
Length of shelf plate		(in)	↑	
Thickness of shelf plate		(in)	↓	
Shelf plate transition radius		(in)	↓	
Stringer or Truss Floor-beam Bracket		(Y/blank)	Y	0
Floor beam connected to stringer		(Y/N)	Y	
Floor beam connected to truss		(Y/N)	Y	
Welded Horizontal Stiffener		(Y/blank)	Y	0
Horizontal stiffener continues through tension zone		(Y/N)	Y	
Multiple plates welded end-to-end or interrupted by vertical stiffener		(Y/N)	Y	
Thickness of horizontal stiffener		(in)	↑	
Horizontal stiffener transition radius		(in)	↓	
Haunch Insert		(Y/blank)	Y	0
Connected using transverse PJP groove welds		(Y/N)	Y	
Web Penetration		(Y/blank)	Y	0
Hole in box girder except at pier cap		(Y/N)	Y	
Hole in open girder		(Y/N)	Y	
Intersecting girders at pier cap		(Y/N)	Y	
Plug-Welded Misplaced Hole		(Y/blank)	Y	0
Field Welded Splice		(Y/blank)	Y	0
Pin and Eyebar Truss or Pin and Hanger Assembly		(Y/blank)	Y	0
Lateral Bracing to Girder Bottom Flange		(Y/blank)	Y	0

Cantilever Floor-Beam Bracket		(Y/blank)	Y	0
Backing Bar		(Y/blank)	Y	0
Intermittent Weld		(Y/blank)	Y	0
Tack Weld		(Y/blank)	Y	0
Tied Arch Floor-Beam		(Y/blank)	Y	0
A514 Steel (T1 Steel)		(Y/blank)	Y	0
,			Bridge Rank	0
0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,0,,,,				

Figure 8.2 Blank program input sheet

8.3 Examples

Following are examples for details that were taken from Mn/DOT bridge plans or photographs when plans do not show details. All of the necessary information is gathered from these documents and read into a program example. A single document (Figure 8.24) holds all the details, although in practice only the details on each individual bridge would be inputted at one time; the rest left blank. The resulting bridge number shows how the sum would be computed if all of these details had been on the same bridge.

Transverse Stiffener Web Gap (Bridge # 9330)

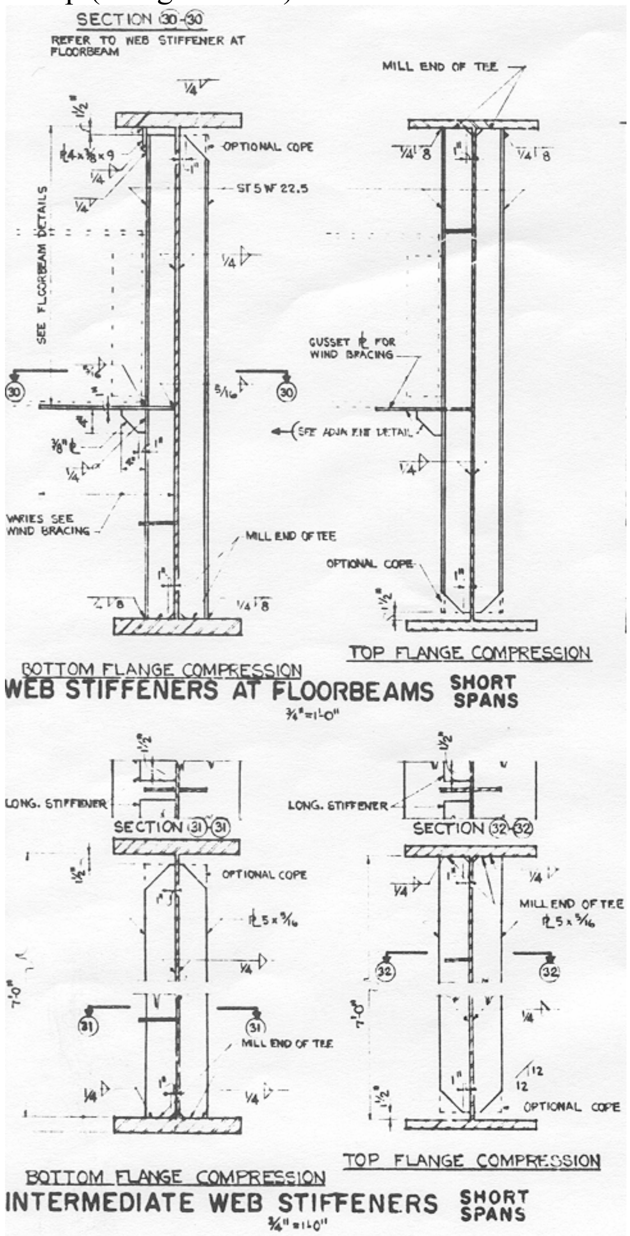


Figure 8.3 Transverse stiffener web gap (Bridge # 9330)

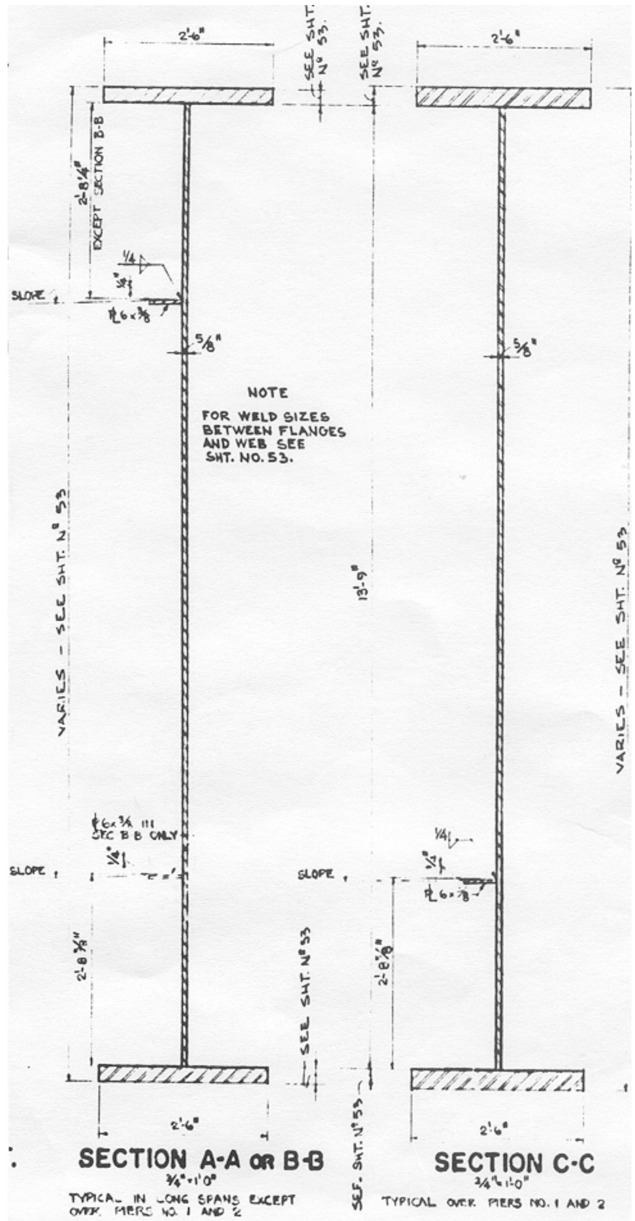


Figure 8.4 Transverse stiffener web gap – girders (Bridge # 9330)

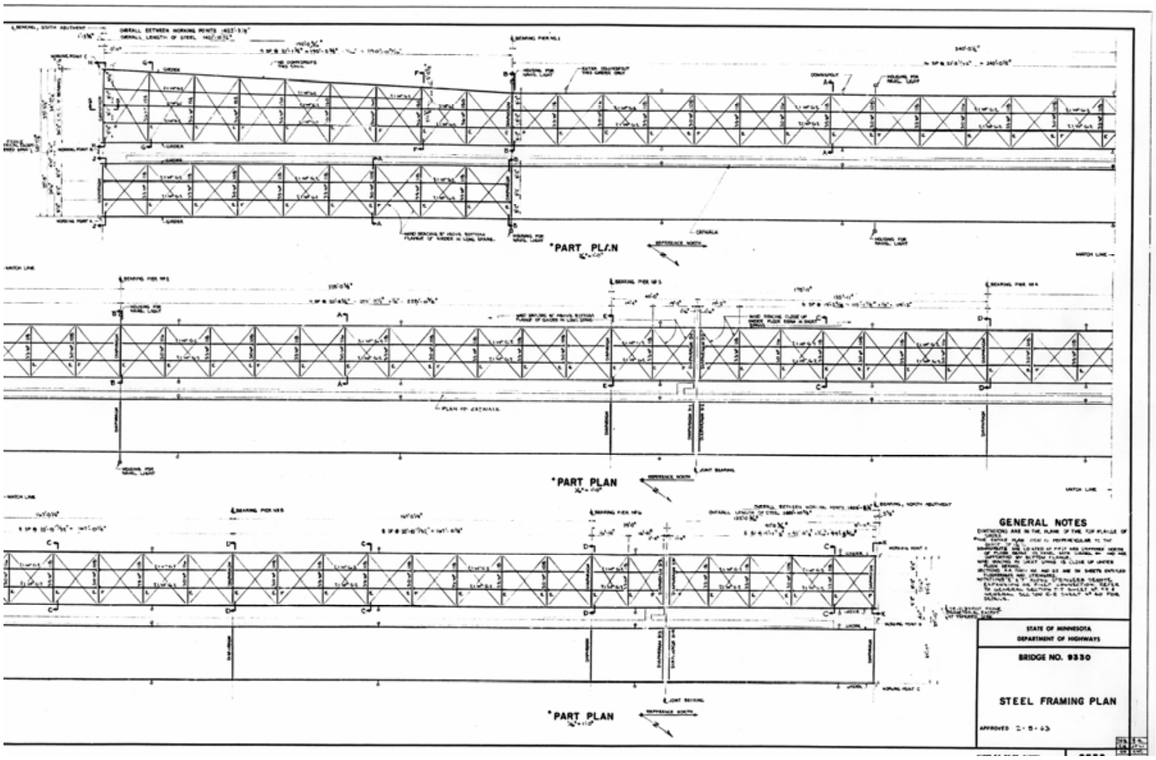


Figure 8.5 Transverse stiffener web gap – framing plan (Bridge # 9330)

Insufficient Cope Radius (Bridge # 9320) – Bridge plans do not usually show cope radii.

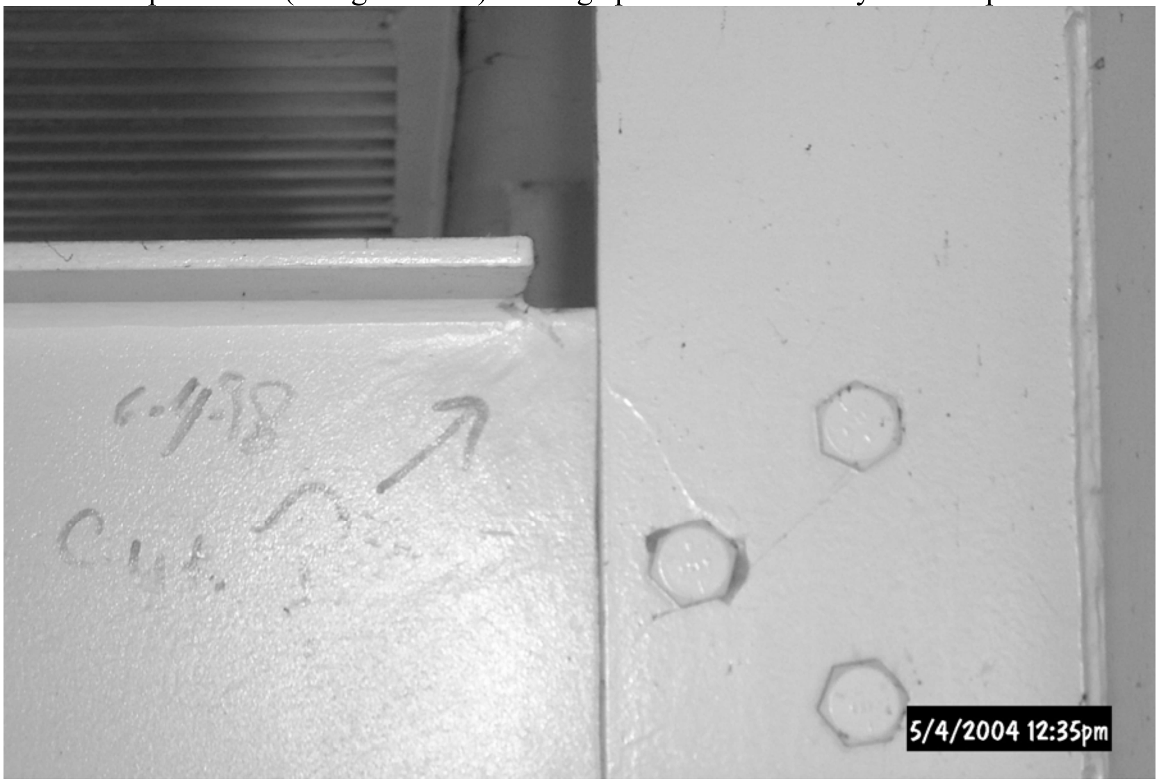


Figure 8.6 Insufficient cope radius (Bridge # 9320)

Partial Length Cover Plate (Bridge # 02803)

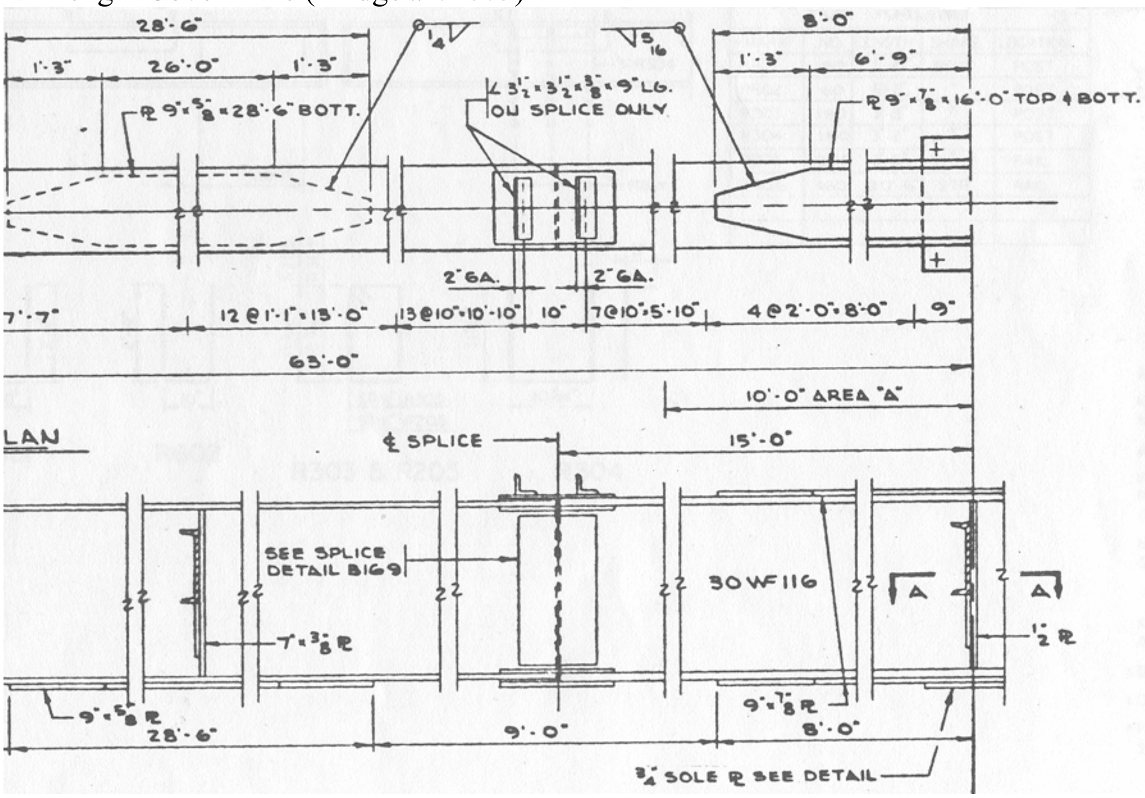


Figure 8.7 Partial length cover plate (Bridge # 02803)

Shelf Plate Welded to Girder Web (Bridge # 9320)

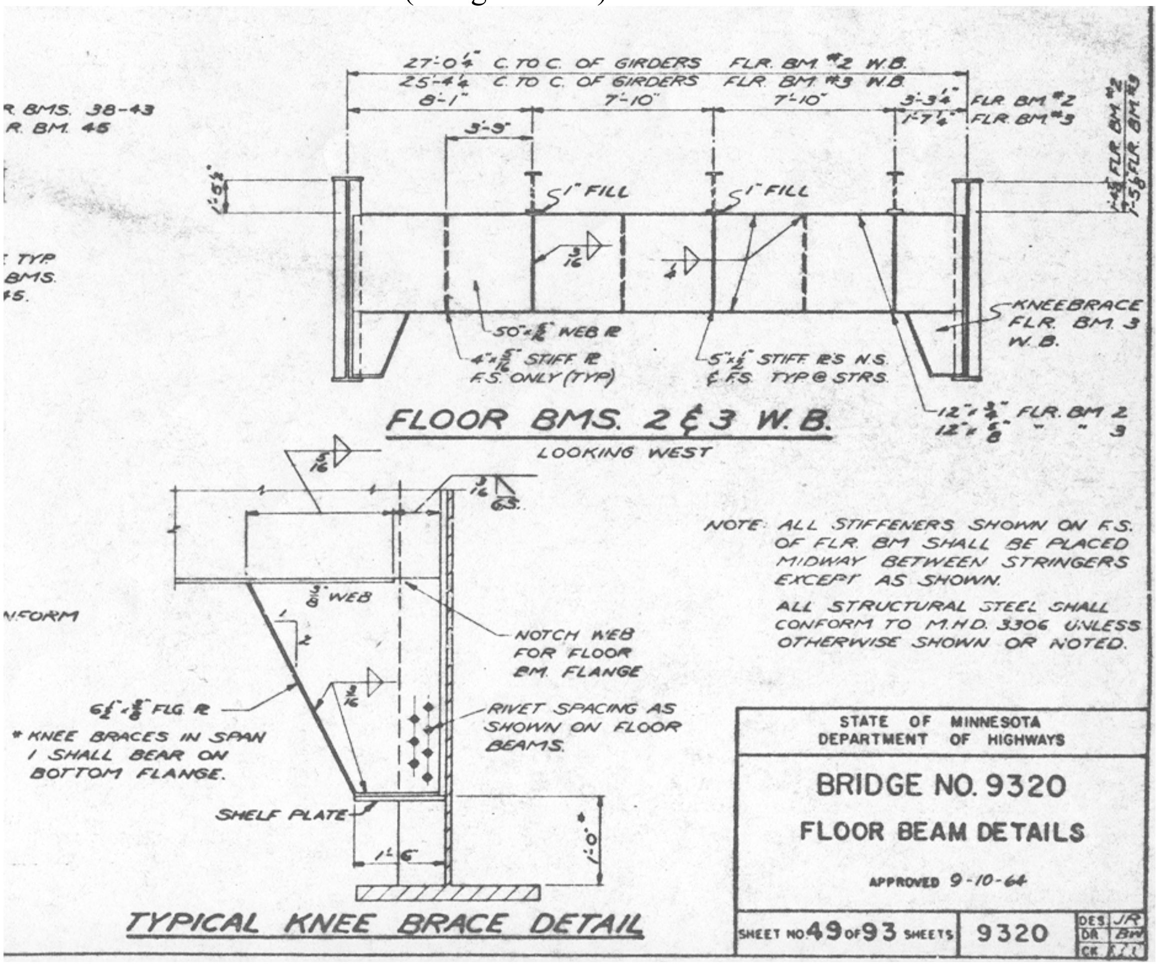


Figure 8.8 Shelf plate welded to girder web (Bridge # 9320)

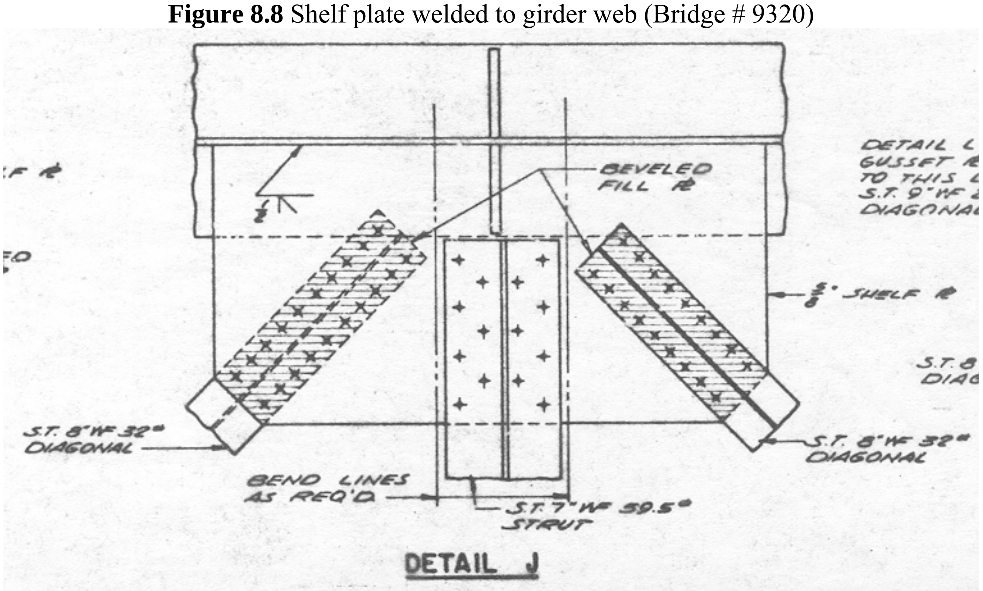


Figure 8.9 Shelf plate welded to girder web – plan view (Bridge # 9320)

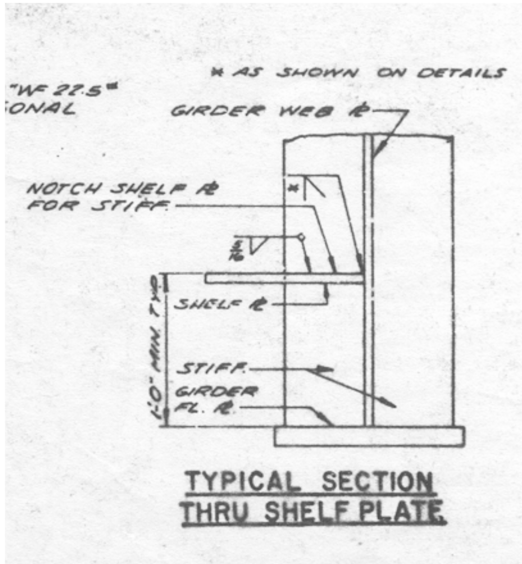


Figure 8.10 Shelf plate welded to girder web – welding detail (Bridge # 9320)

Floor-beam Bracket (Bridge # 5947)

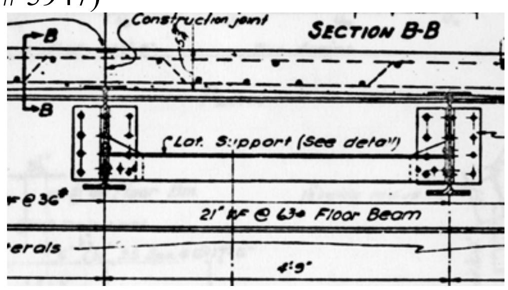


Figure 8.11 Floor-beam bracket (Bridge # 5947)

Welded Horizontal Stiffener (Bridge # 9800)

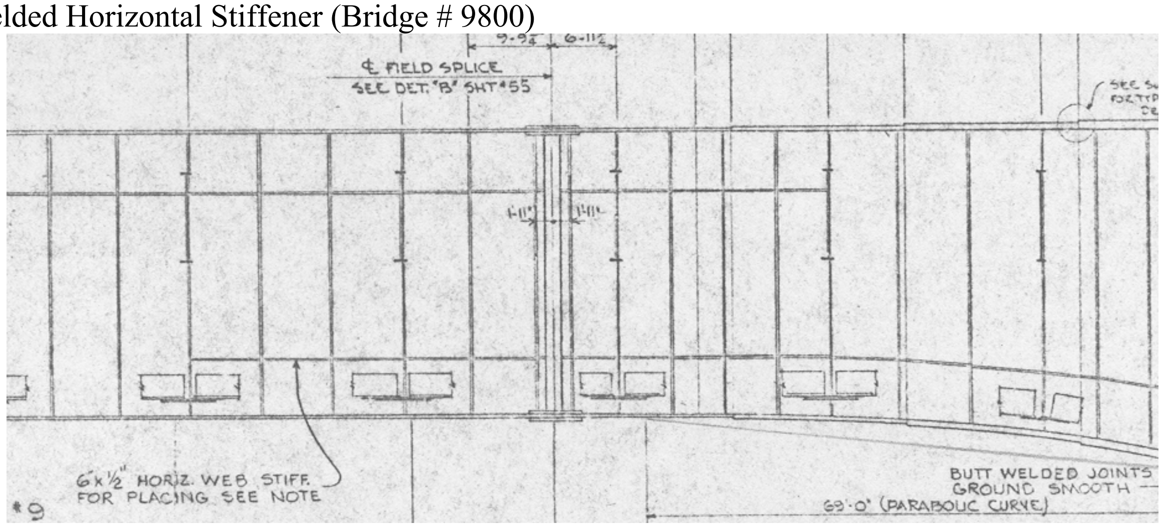


Figure 8.12 Welded horizontal stiffener (Bridge #9800)

Haunch Insert (Bridge # 9779)

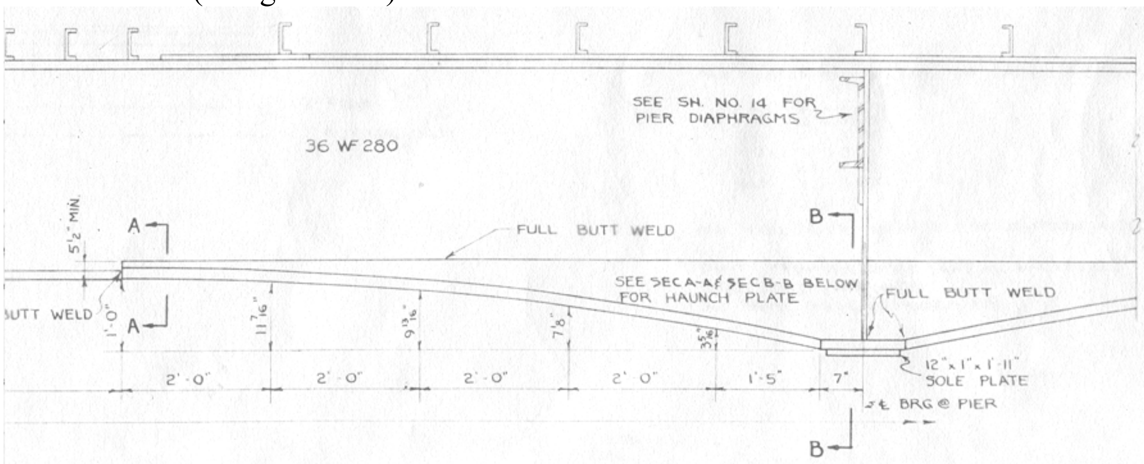


Figure 8.13 Haunch insert (Bridge # 9779)

Web Penetration (Bridge # 27788)

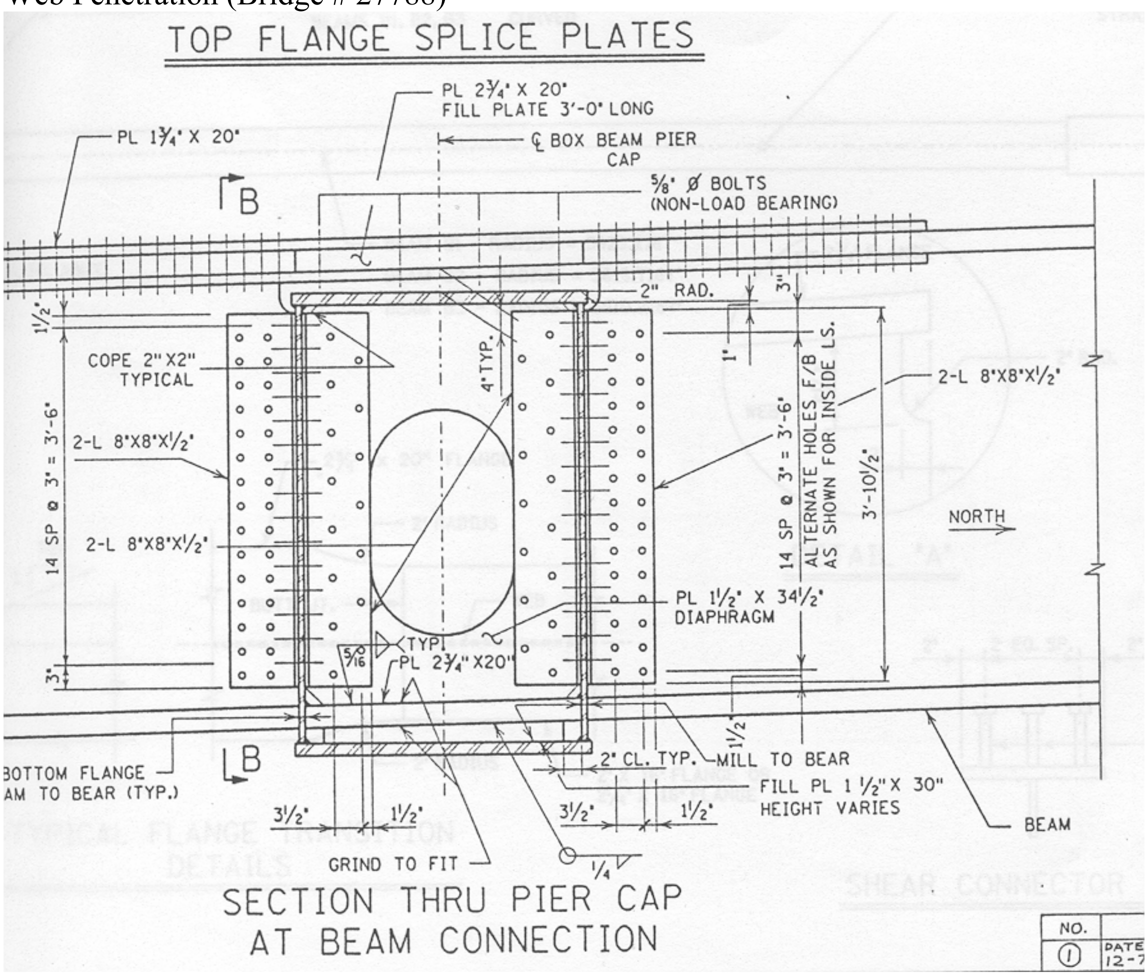


Figure 8.14 Web penetration (Bridge # 27788)

Plug-Welded Misplaced Hole (Bridge # 27552)

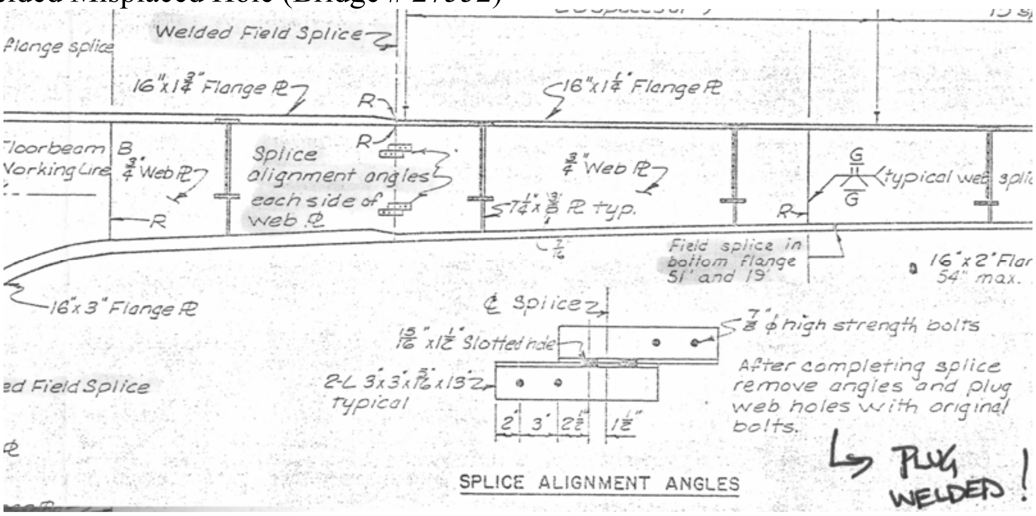


Figure 8.15 Plug-welded misplaced hole (Bridge # 27552)

Field Welded Splice (Bridge # 90856)

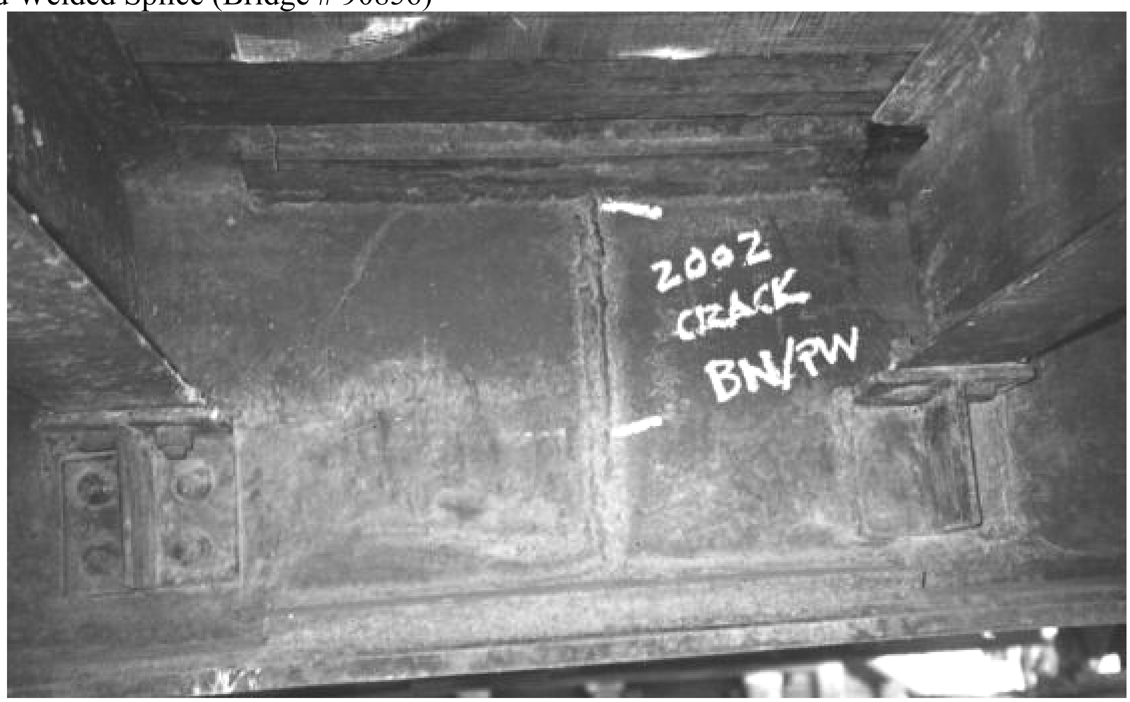


Figure 8.16 Field welded splice (Bridge # 90856)

Pin and Eyebar Truss or Pin and Hanger Assembly (Bridge # 9320)

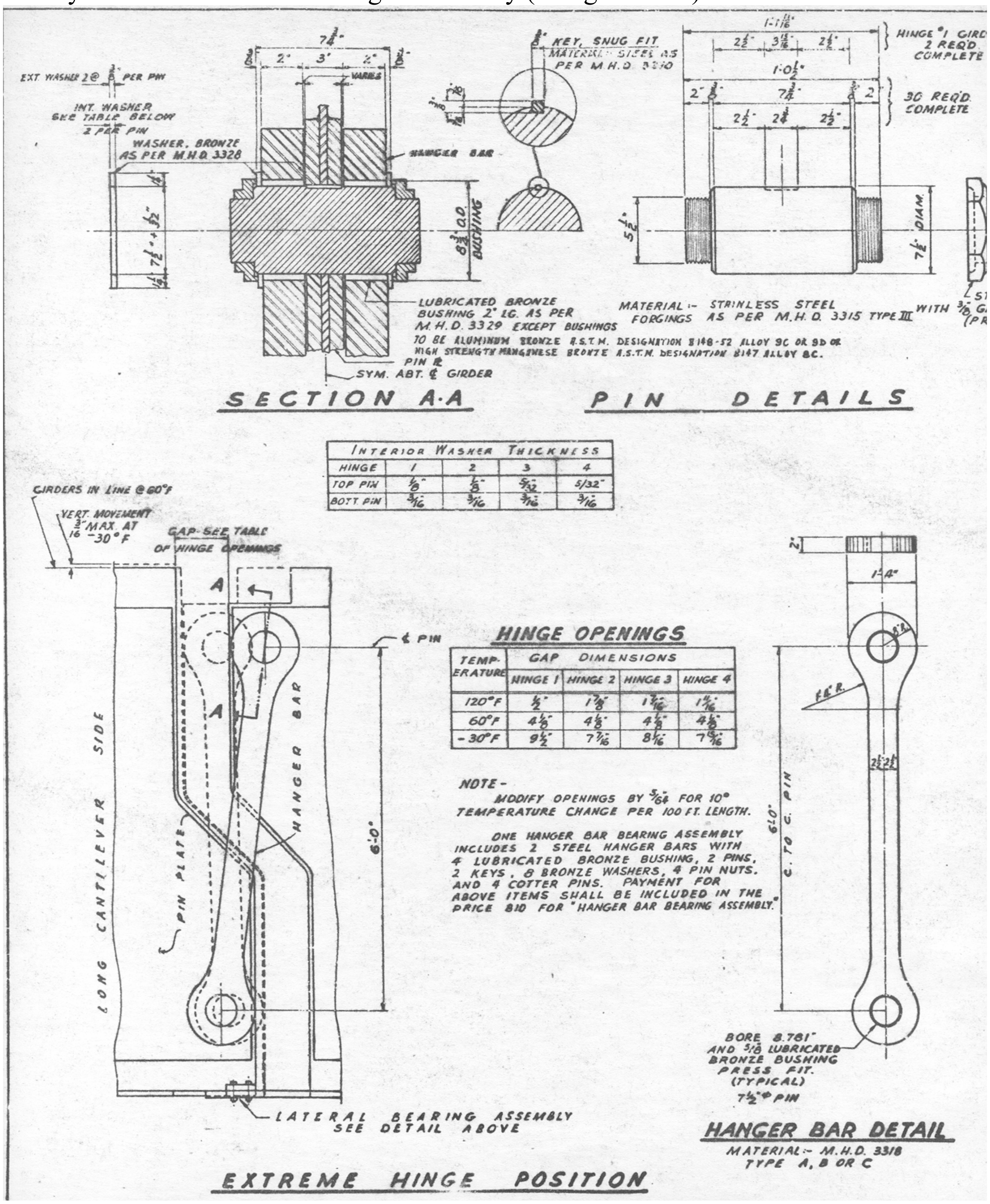


Figure 8.17 Pin and eyebar truss or pin and hanger assembly (Bridge # 9320)

Lateral Bracing to Girder Bottom Flange – No example found. However there is at least one case in the bridge inventory (Wilson, March 2007).

Cantilever Floor-Beam Bracket (Bridge # 5993)

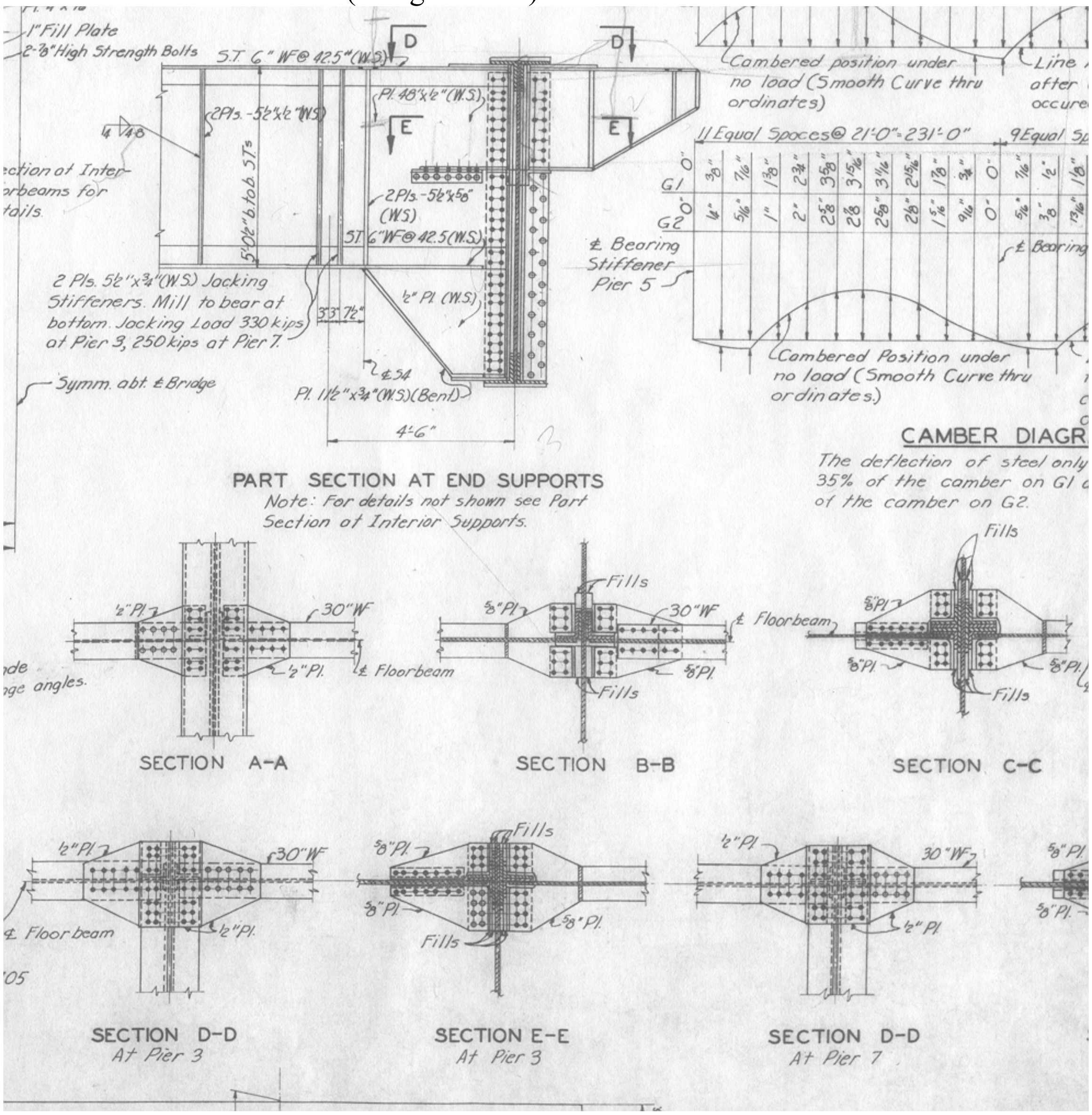


Figure 8.18 Cantilever floor-beam bracket (Bridge # 5993)

Backing Bar (Bridge # 27636)

Figure 8.19 Backing bar (Bridge # 27636)

Intermittent Weld (Bridge # 6566)

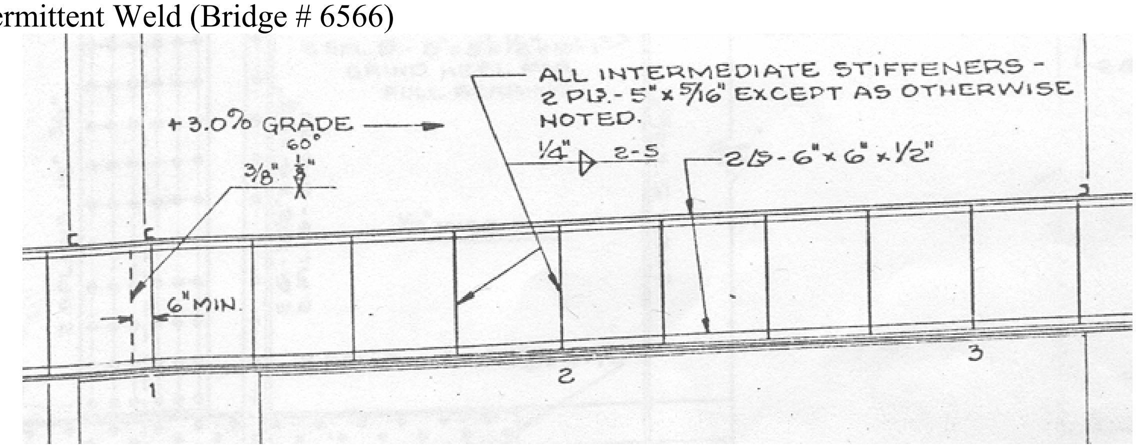


Figure 8.20 Intermittent weld (Bridge # 6566)

Tack Weld (Bridge # 55804)

Figure 8.21 Tack weld (Bridge # 55804)

Tied Arch Floor-Beam (Bridge # 9600S)

Figure 8.22 Tied arch bridge (Bridge # 9600N)

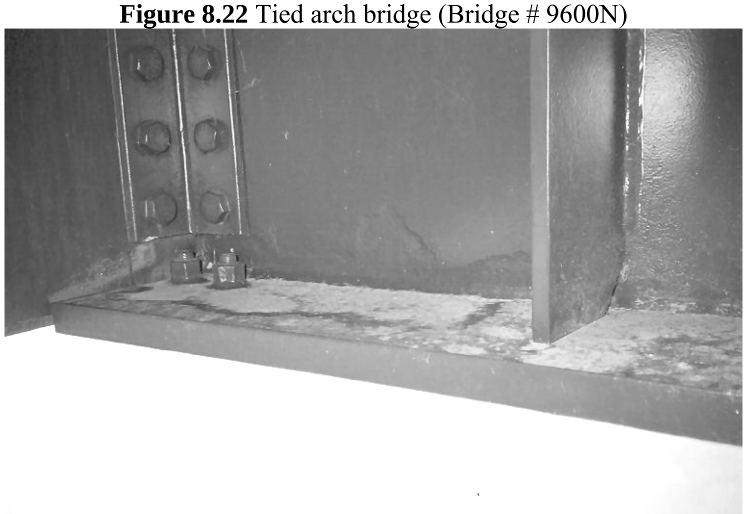


Figure 8.23 Tied arch floor beam (Bridge # 9600S)

A514 Steel (T1 Steel) (Bridge # 9360) – The Washington Avenue Bridge contains this type of fracture sensitive steel.

BRIDGE NUMBER ___________

DETAIL NAME	INPUT	INPUT TYPE	WORST CASE	RANK NO.
Transverse Stiffener Web Gap	Y	(Y/blank)	Y	1
Gap flange	top	(top/bot)	Top	
Bridge span skew	0	(deg)	↑	
Span length containing web gap	167	(ft)	↓	
Web gap height	1.5	(in)	↓	
Girder web thickness	0.625	(in)	↑	
Insufficient Cope Radius	Y	(Y/blank)	Y	2
Is there a cope radius	N	(Y/N)	N	
Cope radius (0 if none)	0	(in)	↓	
Partial Length Cover Plate	Y	(Y/blank)	Y	3
Girder flange thickness	0.850	(in)	↑	
Girder flange width	10.5	(in)	↓	
Is the end tapered	Y	(Y/N)	N	
Cover plate width	9	(in)	↑	
Is there a weld across end of cover plate	N	(Y/N)	N	
Shelf Plate Welded to Girder Web	Y	(Y/blank)	Y	3
Weld termination and cope distances within Detail B41 allowable dim.	Y	(Y/N)	N	
Is any plate intersected or coped around shelf plate	Y	(Y/N)	Y	
Length of shelf plate	18	(in)	↑	
Thickness of shelf plate	0.625	(in)	↓	
Shelf plate transition radius	0	(in)	↓	
Stringer or Truss Floor-beam Bracket	Y	(Y/blank)	Y	1
Floor beam connected to stringer	Y	(Y/N)	Y	
Floor beam connected to truss	N	(Y/N)	Y	
Welded Horizontal Stiffener	Y	(Y/blank)	Y	3
Horizontal stiffener continues through tension zone	N	(Y/N)	Y	
Multiple plates welded end-to-end or interrupted by vertical stiffener	Y	(Y/N)	Y	
Thickness of horizontal stiffener	0.5	(in)	↑	
Horizontal stiffener transition radius	0	(in)	↓	
Haunch Insert	Y	(Y/blank)	Y	2
Connected using transverse PJP groove welds	N	(Y/N)	Y	
Web Penetration	Y	(Y/blank)	Y	2
Hole in box girder except at pier cap	N	(Y/N)	Y	
Hole in open girder	N	(Y/N)	Y	
Intersecting girders at pier cap	Y	(Y/N)	Y	
Plug-Welded Misplaced Hole	Y	(Y/blank)	Y	4
Field Welded Splice	Y	(Y/blank)	Y	4
Pin and Eyebar Truss or Pin and Hanger Assembly	Y	(Y/blank)	Y	4
Lateral Bracing to Girder Bottom Flange	Y	(Y/blank)	Y	3

Cantilever Floor-Beam Bracket	Y	(Y/blank)	Y	3
Backing Bar	Y	(Y/blank)	Y	3
Intermittent Weld	Y	(Y/blank)	Y	3
Tack Weld	Y	(Y/blank)	Y	2
Tied Arch Floor-Beam	Y	(Y/blank)	Y	1
A514 Steel (T1 Steel)	Y	(Y/blank)	Y	3
,			Bridge Rank	47

47,1,2,3,3,1,3,2,2,4,4,4,3,3,3,3,2,1,3,Y,top,0,167,1.5,0.625,Y,N,0,Y,0.85,10.5,Y,9,N,Y,Y,Y,
18,0.625,0,Y,Y,N,Y,N,Y,0.5,0,Y,N,Y,N,N,Y,Y,Y,Y,Y,Y,Y,Y,Y,Y,Y

Figure 8.24 Example program operation

CHAPTER 9 – CONCLUSIONS

Fracture and fatigue susceptible details are still present in bridges across the country; namely those constructed before the 1990's. Cracking of these details may occur, requiring repair or retrofit. An estimation of the number of susceptible details as well as the frequency of occurrence is crucial in order to budget for future rehabilitation. Because cracking of certain details can be catastrophic, leading to the partial or total collapse of a bridge, an index of cracking severity assists in identifying bridges that require careful monitoring, at a minimum.

9.1 Summary

The research performed achieves the stated goals by providing a list of details that have an identified potential for cracking. Furthermore, rank numbers are assigned to each of these details on the basis of frequency of cracking and consequence of occurrence. A finer breakdown of the details is constructed, utilizing geometric properties of the individual detail as well as properties of the bridge itself. This enabled the ranking numbers, zero through four, to correspond with the expected conditions of specific details.

Literature review of many sources provided the overall ranking given in Table 7.1. First, fatigue in steel bridges was analyzed by examining the AASHTO *S-N* categories used to approximate fatigue life. Next, an evaluation of distortional fatigue was conducted, which accounts for about one-half of all cracking cases. Past research by the University of Minnesota quantified stresses in some of these details (Jajich et al. 2000, Berglund and Schultz 2002, Severtson et al. 2004, Li and Schultz 2005). Then, case studies involving both fracture and fatigue from a well-known book by Fisher (1984) were considered. A method for quantifying fatigue life from stresses is developed in NCHRP 299 (Moses et al. 1987), but was not used in this study.

After gathering background information, a list was compiled of details that are considered to be prone to cracking problems. A further breakdown of more common details, according to geometric properties, was done using information from codes, case studies, and prior experience. After that, the experiences of several state departments of transportation were collected; initially by looking at bridge plans and inspection reports from Mn/DOT, and then by conducting a nationwide survey. The survey asked agencies to order the flagged details depending on their frequency of cracking and also provide any extra details that were not already being considered in the study. The last sources that were examined were the multiple codes which involved construction and engineering design specifications for the details under consideration during the period in question. Rather than simply scrutinizing the current code, examination of all codes since 1973 allowed tracking of code changes, enabling estimation of years in which bridges might contain certain problematic details.

By putting together the information gathered, the classification table (Table 7.1), lists all of the flagged details. Each is supplied with a description and picture for easy identification. A year is given which can be used to approximate the end of use for each detail, along with its cited source. Following this are geometric constraints with appropriate rank number next to them.

There are eighteen details in total, including many that can be found on bridge plans and some that can only be identified in the field.

In the application stage of this study, a procedure is developed which allows bridge owners to arrange their bridges by composite rank number through use of data collection and a computer program. The Microsoft EXCEL-based program enables either inspectors or engineers examining bridge plans to input geometric data for flagged details. Once the worst detail of each type is entered into the spreadsheet, the program computes rank numbers for each detail and a composite rank for the bridge. This information can be used in bridge management databases to estimate budgeting for repairs and retrofits.

9.2 Conclusions

Cracking in steel bridges with inadequate details is an issue that will be overcome with time. As bridges are designed with larger girders to avoid using many of the local-strength-increasing details from the past, fracture and fatigue vulnerability is all but eliminated. This progression, in fact, can be followed by codes, which eliminate problem areas after research has been performed to describe the problem. Details that are seeing cracking today are those that are approaching their fatigue life after many decades of service. Thus, the evolution of design lags behind the identification and resolution of problems unless research testing is performed.

The number of details experiencing cracking with immediate risk of bridge failure is only a handful in each state. The number of details experiencing cracking with some risk of partial bridge collapse if not repaired is only slightly higher. However, the number of details that have possibility for cracking is large; and the cost of retrofit if small cracks are allowed to progress is substantial. Risk is reduced by inspecting and eliminating flagged details present in this study.

From examination of the classification table, the highest rank number for the eighteen flagged details result in five 4's, nine 3's, three 2's, and one 1. Although code provisions may have allowed these problematic details in the past, engineering judgment eliminated many of the worst-case scenarios. Therefore, although the distribution of the eighteen details among the categories seems fairly even, more 2's will be found than any other rank number. This occurs because most details fall into the less severe categories 1 and 2, and the rank 2 is the only one associated with high frequency of occurrence.

9.3 Recommendations

The research performed in the current study is for assisting in evaluating bridges for their fracture and fatigue susceptibility. Used with sound engineering judgment, the information contained herein will assist in identifying many, but not all, details with fracture and fatigue issues. Other fracture of fatigue susceptible details not listed in this report may require careful attention as well. The ranking system is based on qualitative judgment and can be used for assessing what issues exist in the bridge inventory but should not be substituted for inspections or other measures currently in place.

That being said, with proper application, this research may contribute to knowledge of the extent and distribution of susceptible details that exist in a bridge inventory and how frequently they crack. Incorporation of the program in 0 can enable databases to become much more comprehensive by noting the existence and geometry for a uniform set of problematic details. It is recommended that personnel in state departments of transportation, with the help of an engineer and/or inspector, examine bridge plans, gather problematic details, and enter them into a program for incorporation into bridge management databases. Also, because many of the details cannot be found on bridge plans, i.e. field-welded splices, inspectors should take printed copies of the program spreadsheet, to be used as a questionnaire, to fill in any details they encounter in the field.

The program file can be adapted to include extra information such as more details or member sizes. However, it is noted that addition of ranked details to the program will require that previously ranked bridges be ranked again, because bridge rank number is a composite score and more details may account for larger composite rank numbers. This means that, unless they are ranked again, any bridges that were previously ranked, before the addition of new details to the program, would be ranked on a different scale. It is also recommended that cells in the spreadsheet program, other than those that receive input data, remain locked so that they cannot be changed. Otherwise, adjustments to rank numbers could be made by the user based on their judgment, and this may skew the results.

Incorporation of the classification table's "date of code prohibition" column to eliminate looking for details should not be done. The column is for purposes of showing what to expect certain bridges to have. Since adoption of code requirements into actual bridge construction practice takes many years and varies by application, it is difficult to estimate the true time that the prohibition of a detail becomes effective. Furthermore, in some cases details continued to be used well after their common use was no longer allowed by code. An example of this is the incorporation of pin and hanger assemblies into the 1980's.

The national survey was an effort to bring together many state departments of transportation for the purpose of cooperation in identifying problematic details. Because the current research is based heavily upon history of actual cracking cases, the largest set of bridges possible should be examined. The help from the agencies that replied to the survey allowed for this. Aggregation of information from all state departments of transportation in the United States would be ideal for the most complete collection of data.

Some DOTs replied to the survey with other details that were not included in the classification table. These were all taken into account, but most of them were not present in Minnesota and not enough information was known about them. Further research should be done to quantify all the details with which other DOTs are having problems and to investigate their causes. The end result could be a national ranking system that is applicable to all states and can be incorporated into code.

The rank numbers are based upon qualitative conclusions concerning cracking history. The most thorough knowledge was gathered from the many studies conducted on the web gap detail. For this detail, enough information is known to be able to estimate the expected stress levels under traffic loading. This enables a more complete ranking than the current study does. If more details were researched as methodically as the web gap, a better ranking could be conducted allowing for fatigue stresses to be calculated. However, to be able to calculate stresses and account for traffic loading, a large computational effort would be required.

The ranking system developed in this report offers useful information in a simple format. However, for certain bridges alternative ranking scales may be necessary depending on the planned use for the rank. The composite bridge rank developed in Chapter 7 is most useful for the purpose of identifying those bridges with multiple problematic details. Calculation by summation of the individual ranks is most effective for identifying these bridges; in a sense, a high composite rank would indicate multiple problems, most likely both severe and mild. However, for bridges that do not rank as highly, it may be important to understand whether the rank comes from a multitude of different mild-severity details, or a single, severe detail.

If the bridge owner wants to examine details with higher risk, then a bridge with a single rank-4 detail would carry more importance than one with four different rank-1 details. To determine how to spend repair funds in a cost-effective manner, it may be more beneficial to repair a bridge with one type of severe detail rather than a bridge with many different details that have a lower likelihood of cracking. Thus, an alternative rank may be needed here which offers an indication of detail severity rather than an aggregation of all detail types. For example, a new rank, G, may be defined on the basis of the total number of problematic details in the bridge, that is $G = \sum_{i=1}^{4} n_i r_i / N$, where n_i is the number of details of rank r_i, and $N = \sum_{i=1}^{4} n_i$.

Another bridge ranking may focus on the expected frequency of crack occurrence, where rank-2 details would hold more weight. This type of rank may be useful for evaluating and mitigating propagation of cracks when combined with bridge age and traffic loading statistics. It may also be important to understand whether or not details are more fracture prone or fatigue sensitive. For the most part, a higher detail rank means it is more fracture sensitive. Separating different rank numbers in this way could assist in evaluating maximum loading or considerations in changing bridge load ratings.

Understanding the properties of traffic loading, such as traffic counts, vehicle weights, axle distributions, and traffic patterns, is important to better understand the definition and significance of different rank numbers. Load magnitudes and numbers of cycles are key components in fatigue evaluation, and incorporating them can be done efficiently by using the traffic spectrum concept which offers the relation between ESALs (equivalent single-axle loads) and the corresponding expected ADT (or ADTT). By assuming that all bridges on the trunk highway system (i.e., those considered in this study) carry the same traffic, the current ranking system equates quantities that are not equal. For any particular fatigue detail, less traveled roads may never reach their fatigue lives, where as busy bridges may reach it in only a couple of years. The magnitude of vehicle loads is also important because bridges with small loads may be insensitive to their problematic details if stresses are below the endurance limit (i.e., stress value for infinite fatigue life). To more realistically rank details for fracture and fatigue, a ranking scheme that incorporates that traffic spectrum concept may be necessary.

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APPENDIX A: Steel Designations and Years in Use

Steel Description	Steel Designation		Years in Use
	American Society for Testing and Materials (ASTM)	American Association of State Highway and Transportation Officials (AASHTO)	
Structural Carbon Steel	A7	M94	1900-1967
Structural Nickel Steel	A8	M96	1912-1962
Structural Steel	A36	M183	1960-Present
Structural Silicon Steel	A94	M95	1925-1965
Structural Steel	A140		1932-1933
Structural Rivet Steel	A141	M97	1932-1966
High-Strength Structural Rivet	A195	M98	1936-1966
High-Strength Low-Alloy Steel	A242	M161	1941-Present
Low and Intermediate Tensile Strength Carbon Steel Plates	A283/A284		1946-Present
Steel Sheet Piling	A328	M202	1950-Present
Structural Steel for Welding	A373	M165	1954-1965
High-Strength Structural Steel	A440	M187	1959-1979
High-Strength Low-Alloy Structural Manganese Vanadium Steel	A441	M188	1954-1989
High-Yield-Strength, Quenched and Tempered Alloy Steel Plate (Suitable for Welding)	A514 A709 Grade100/100W	M244 M270 Grade 100/100W	1964-Present 1974-Present
Hi Strength Low-Alloy Columbium-Vanadium Steel of Structural Quality	A572 (A709 Grade 50)	M223 (M270 Grade 50)	1966-Present (1974-Present)
Hi-Strength Low-Alloy Structural Steel with 50 ksi Minimum Yield Point to 4” thick	A588 (A709 Grade 50W)	M222 (M270 Grade 50W)	1968-Present (197 4-Present)
High-Strength Low-Alloy Steel H-Piles and Sheet Piling	A690		1974-Present
Quenched and Tempered Low-Alloy Structural Steel Plate with 70 ksi Minimum Yield Strength to 4” thick	A852 (A709 Grade 70W)	M313 (M270 Grade 70W)	1985-Present (1985-Present)

APPENDIX B: Calculations for Web Gap Stresses

Equation 2.11: $$\sigma_{wg} = 2.5E\left(\frac{t_w}{g}\right)\left(\frac{A_1 \cdot L^2 + A_2 \cdot L + A_3}{L}\right)$$

"*A*" constants from Table 2.19:

(*L* in inches)			
Deg.	A_1	A_2	A_3
20	-3.3700E-07	0.001486	-0.3399
40	-3.1150E-07	0.001522	-0.4065
60	-4.3520E-07	0.002185	-0.9156

E (ksi)	***L*** (in.)	t_w (in.)	***g*** (in.)	***skew*** (˚)	σ_{wg} (ksi)
29,000	960	0.5	2	20	***14.7***
29,000	960	0.5	2	40	***14.5***
29,000	960	0.5	2	60	***14.7***
29,000	960	0.25	1	20	***14.7***
29,000	960	0.25	1	40	***14.5***
29,000	960	0.25	1	60	***14.7***
29,000	1920	0.5	2	20	12.0
29,000	1920	0.5	2	40	12.9
29,000	1920	0.5	2	60	***15.8***
29,000	1920	0.25	1	20	12.0
29,000	1920	0.25	1	40	12.9
29,000	1920	0.25	1	60	***15.8***

APPENDIX C: Survey Replies

Nevada

Adam,

Nevada's experience with fatigue damage has related mostly to out-of-plane bending due to perpendicular cross-frames in skewed bridges or interior "Z-bracing" in tub girders. Horizontal wind-bracing has been the next most prevalent cause of fatigue cracking, particularly where the horizontal connection plate has been welded particularly close to vertical web bracing and/or cross-frame connection plates. The remaining causes ranked below have had minimal occurrences and could virtually be ignored as significant causes of fatigue fractures.

If you have any further questions regarding these responses, feel free to contact me through any of the means listed below.

Marc Grunert
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NDOT - Bridge Division
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__1___Transverse Stiffener Web Gaps - Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to both flanges of the girders; instead a gap was left anywhere from 0" (bearing) to 2" or more. Cracking can occur in any number of elements in this area, including the welds, the girder flange, the girder web, or the plate.

__3___Cover plated beams and flange gussets -Cover plates are plates attached to the underside of girder flanges to increase the moment of inertia of girders locally. Cracks usually form at the ends of the cover plates, either in the cover plate, the weld, or the girder flange.

__0___Stringer to floor beam (truss) brackets - Cracking occurs in the angles connecting floor beams to other elements. Cracks can occur anywhere within the angle connector, especially around bolts or rivets.

__0___Cantilever floor-beam brackets - These plates are laid horizontally and usually bolted or riveted to the girders. They protrude out to the sides of the bridge and are connected to the top of short beams that extend out from the bridge to increase bridge width. Cracking can occur around tack welds, bolts, rivets, or across the plate parallel to girders.

__2___Web Connection Plates - Horizontal plates welded to the girder web used for diaphragms or other attachments. Cracks usually occur when the plate intersects a transverse stiffener.

__0___Transverse Groove Welds - Groove welds on girder webs. Usually these occur at the end of horizontal stiffeners, or welds connecting long sections of horizontal stiffeners together, or the end of haunch inserts (The bottom flange is cut out of the girder and replaced with a groove-welded, higher-depth section to increase moment of inertia around supports).

__0___Web penetrations - When a member passes through the web of another. Cracking is more common when backing bars are left in place. These details are common in pier caps or box girders.

__4___Coped members - Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually is caused by too small of a radius or no radius.

__0___Tied arch floor beams - Floor beams can exhibit separation of beam web and flange due to rotation of the beams under distortional fatigue.

__5___Continuous longitudinal welds - Commonly, these long welds connect plates or other shapes along their length, to form some sort of built-up-section. Cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds.

__0___Lamellar Tearing - Separation of layers of metal within a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

Georgia
Comments for the Georgia DOT are listed below.

Please call or e-mail me if you have any questions on the comments.

Thanks.

Paul V. Liles, Jr., P.E.
State Bridge Engineer
(404) 656-5280

__1___Transverse Stiffener Web Gaps - Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to both flanges of the girders; instead a gap was left anywhere from 0" (bearing) to 2" or more. Cracking can occur in any number of elements in this area, including the welds, the girder flange, the girder web, or the plate.

__0___Cover plated beams and flange gussets -Cover plates are plates attached to the underside of girder flanges to increase the moment of inertia of girders locally. Cracks usually form at the ends of the cover plates, either in the cover plate, the weld, or the girder flange.

__3___Stringer to floor beam (truss) brackets - Cracking occurs in the angles connecting floor beams to other elements. Cracks can occur anywhere within the angle connector, especially around bolts or rivets.

__0__Cantilever floor-beam brackets - These plates are laid horizontally and usually bolted or riveted to the girders. They protrude out to the sides of the bridge and are connected to the top of short beams that extend out from the bridge to increase bridge width. Cracking can occur around tack welds, bolts, rivets, or across the plate parallel to girders.

__0___Web Connection Plates - Horizontal plates welded to the girder web used for diaphragms or other attachments. Cracks usually occur when the plate intersects a transverse stiffener.

__0___Transverse Groove Welds - Groove welds on girder webs. Usually these occur at the end of horizontal stiffeners, or welds connecting long sections of horizontal stiffeners together, or the end of haunch inserts. (The bottom flange is cut out of the girder and replaced with a groove-welded, higher-depth section to increase moment of inertia around supports).

__0___Web penetrations - When a member passes through the web of another. Cracking is more common when backing bars are left in place. These details are common in pier caps or box girders.

__2___Coped members - Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually is caused by too small of a radius or no radius.

__0___Tied arch floor beams - Floor beams can exhibit separation of beam web and flange due to rotation of the beams under distortional fatigue.

__0___Continuous longitudinal welds - Commonly, these long welds connect plates or other shapes along their length, to form some sort of built-up-section. Cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds.

__0___Lamellar Tearing - Separation of layers of metal within a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

Thank you very much for your assistance. Your input will help advance the overall understanding of fatigue susceptible details. -Adam Lindberg

Part 2.

No further examples. Approximate number of known bridges with the type cracks over the last twenty years is as follows:

Transverse stiffener web gaps - ten
Coped members - two
Stringer to floor beams - one

Most have been repaired.

<u>Indiana</u>
Mr. Lindberg

I have answered the question on your survey. My answers are in red type.

Bill Dittrich
INDOT Bridge Inspection Engineer
(317) 232-5474

INDOT has had +- 15 bridges, over the last 20-years, (mostly in the early 1990's), that developed cracks similar to detail (a). Generally these cracks developed when the Stiffener had an X-Bracing attached, and the X-bracings were staggered due to skew, thus causing out-of plane bending cracking to develop. On advanced cracking, we would also get a horizontal crack along the toe of the weld between the web and the flange.____Transverse Stiffener Web Gaps – Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to both flanges of the girders; instead a gap was left anywhere from 0" (bearing) to 2" or more. Cracking can occur in any number of elements in this area.

The great majority of the welded coverplates on bridges in Indiana are tapered. We have only had a few small cracks in the welds at the toes of the terminal ends of the welds. Purdue University conducted quite a bit of research on these coverplates for us in the 1990's, and convinced us that the welds on these tapered coverplates will grow very slowly, thus giving our inspectors plenty of time to see and find them. They also developed a bolted retro-fit that we are using extensively on INDOT Bridges. _____Cover plated beams and flange gussets – cracks usually form at the ends of the cover plates, either in the cover plate, the weld, or the girder flange.

We have had very few of these types of cracks on INDOT bridges. Most occurred in the 1980's or earlier. These types of cracks are still found on some of our Local Bridges._____Stringer to floor beam (truss) brackets – Cracking occurs in the angles connecting floor beams to other elements. Cracks can occur anywhere within the angle connector.

_We have not had this type of problem on any of our INDOT Bridges, and I have not heard of any on our Local Bridges.____Cantilever floor-beam brackets – These plates are laid horizontally and usually bolted or riveted to the girders. They are also connected to the top of short beams that extend out from the bridge to increase bridge width. Cracking can occur around tack welds, bolts, rivets, or across the plate as shown.

INDOT has had four of these types of cracks over the last 23-years, (1983{I-70}, 1985 {I-65}, 1994 {I-64}, and May 2006 {I-70}). I believe that the 1983 crack was 48" long, the 1985 crack was 21" long, the 1994 crack was 70" long, and the 2006 crack was 9.5" long. The first three cracks began around the Web/Transverse Stiffener/Connection Plate intersecting weld area. The 2006 crack began around the end of the weld of the Horizontal Connection Plate – away from the Stiffener. All of these locations had X-Bracings and Lateral Bracings attached._____Web

Connection Plates – horizontal plates welded to the girder web. Cracks usually occur when plate intersects a transverse stiffener.

_INDOT has had three bridges with multiple cracks in Horizontal Web Stiffeners, all in the late 1990's. All of these cracks developed in poor quality splice welds in the stiffener plates, and NOT in the welds to the girder webs.____Transverse Groove Welds – Groove welds to web of girders. Usually occur at the end of horizontal stiffeners or the end of haunch inserts.

_INDOT has not had this type of cracking, and we only have a few bridges with this type of detail.____Web penetrations – When a member passes through the web of another. Cracking is more common when backing bars are left in place. These details are common in pier caps.

_INDOT has had many of these types of cracks, but mainly in the 1980's and 1990's. We have not had much of this lately, on INDOT Bridges, but are probably still having this problem on our Local Bridges. Most of our cracking resulted from significant section loss to the web, above or below the connection plate.___Coped members – Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually caused by small radius or no radius.

INDOT has not had this type of cracking on our Tied Arch bridges. We have had a number of crack indications in our Tie-Chords on one bridge, mainly due to welds flaws and shrinkage during fabrication. Only a few of these have resulted in actual cracks._____Tied arch floor beams – Separation of beam web and flange by rotation of beams.

_ INDOT has not had this type of cracking on the "Box Members" that make up the Tie-Chords of our Tied Arch bridges. We have had a number of crack indications in these welds, mainly due to welds flaws and shrinkage during fabrication, but no actual cracks._________Continuous longitudinal welds – cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds. Commonly, these welds connect plates or other shapes along their length.

NONE -- INDOT does not have many of these types of details._____Lamellar Tearing – Separation of layers of metal within a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

Mississippi
Please see the attached. We do not have other fatigue prone details that we plan to attach per your Part II.

Mitchell K. Carr, P.E.
Bridge Engineer
Mississippi Department of Transportation
Phone (601) 359-7200
Fax (601) 359-7070

__1__Transverse Stiffener Web Gaps – Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to both flanges of the girders; instead a gap was left anywhere from 0” (bearing) to 2” or more. Cracking can occur in any number of elements in this area.

__0__Cover plated beams and flange gussets – cracks usually form at the ends of the cover plates, either in the cover plate, the weld, or the girder flange.

__4__Stringer to floor beam (truss) brackets – Cracking occurs in the angles connecting floor beams to other elements. Cracks can occur anywhere within the angle connector.

__0__Cantilever floor-beam brackets – These plates are laid horizontally and usually bolted or riveted to the girders. They are also connected to the top of short beams that extend out from the bridge to increase bridge width. Cracking can occur around tack welds, bolts, rivets, or across the plate as shown.

__3__Web Connection Plates – horizontal plates welded to the girder web. Cracks usually occur when plate intersects a transverse stiffener.

__0__Transverse Groove Welds – Groove welds to web of girders. Usually occur at the end of horizontal stiffeners or the end of haunch inserts.

__0__Web penetrations – When a member passes through the web of another. Cracking is more common when backing bars are left in place. These details are common in pier caps.

__2__Coped members – Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually caused by small radius or no radius.

__0__Tied arch floor beams – Separation of beam web and flange by rotation of beams.

__0__Continuous longitudinal welds – cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds. Commonly, these welds connect plates or other shapes along their length.

__0__Lamellar Tearing – Separation of layers of metal within a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

Illinois
Mr. Lindberg,

Below is the completed survey response from the Illinois Department of Transportation. I hope this information is useful.

Carl Puzey
Illinois Department of Transportation
Bureau of Bridges and Structures
Bridge Investigations and Repair Plans Unit
Phone: (217) 785-4511
fax: (217) 782-7960
e-mail: Carl.Puzey@illinois.gov

__2___Transverse Stiffener Web Gaps - Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to both flanges of the girders; instead a gap was left anywhere from 0" (bearing) to 2" or more. Cracking can occur in any number of elements in this area, including the welds, the girder flange, the girder web, or the plate.

__4___Cover plated beams and flange gussets -Cover plates are plates attached to the underside of girder flanges to increase the moment of inertia of girders locally. Cracks usually form at the ends of the cover plates, either in the cover plate, the weld, or the girder flange.

__3___Stringer to floor beam (truss) brackets - Cracking occurs in the angles connecting floor beams to other elements. Cracks can occur anywhere within the angle connector, especially around bolts or rivets.

_____Cantilever floor-beam brackets - These plates are laid horizontally and usually bolted or riveted to the girders. They protrude out to the sides of the bridge and are connected to the top of short beams that extend out from the bridge to increase bridge width. Cracking can occur around tack welds, bolts, rivets, or across the plate parallel to girders.

__5___Web Connection Plates - Horizontal plates welded to the girder web used for diaphragms or other attachments. Cracks usually occur when the plate intersects a transverse stiffener.

_____Transverse Groove Welds - Groove welds on girder webs. Usually these occur at the end of horizontal stiffeners, or welds connecting long sections of horizontal stiffeners together, or the end of haunch inserts. (The bottom flange is cut out of the girder and replaced with a groove-welded, higher-depth section to increase moment of inertia around supports).

__6___Web penetrations - When a member passes through the web of another. Cracking is more common when backing bars are left in place. These details are common in pier caps or box girders.

__1___Coped members - Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually is caused by too small of a radius or no radius.

_____Tied arch floor beams - Floor beams can exhibit separation of beam web and flange due to rotation of the beams under distortional fatigue.

_____Continuous longitudinal welds - Commonly, these long welds connect plates or other shapes along their length, to form some sort of built-up-section. Cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds.

_____Lamellar Tearing - Separation of layers of metal within a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

Thank you very much for your assistance. Your input will help advance the overall understanding of fatigue susceptible details. -Adam Lindberg

Part 2 response:

Another detail which has experienced cracking is where the fillet weld connecting a longitudinal stiffener to girder web terminates at or near the fillet weld connecting a transverse (vertical) stiffener to the web. In recent years, since the failure of the Hoan Bridge, this is thought to have been a brittle fracture that is the result of tri-axial constraint rather than the result of fatigue. However, I understand that there may still be some discussion as to whether this brittle fracture may occur, at least in some cases, when there is a pre-existing flaw in the weld or web material or when there is a very small fatigue crack present. I know this survey relates to fatigue, but for these reasons I mention this crack type here.

Additional comment:

We have had webs crack where web connection plates, typically for lateral wind bracing connections, terminate at or are notched around transverse (vertical) web stiffeners. I have ranked this type of connection as #5 above. The cases we have experienced we believe to be brittle fracture similar to that of the Hoan Bridge rather than fatigue related; however, for the reasons stated above in our Part 2 response I have included it in our response.

Washington

Attached is Washington State DOT's response to your survey. Please let me know if there are any questions. Thank you.

Jugesh Kapur, PE, SE
State Bridge & Structures Engineer
360-705-7207

_4___Transverse Stiffener Web Gaps – Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to both flanges of the girders; instead a gap was left anywhere from 0" (bearing) to 2" or more. Cracking can occur in any number of elements in this area, including the welds, the girder flange, the girder web, or the plate.

_7___Cover plated beams and flange gussets – Cover plates are plates attached to the underside of girder flanges to increase the moment of inertia of girders locally. Cracks usually form at the ends of the cover plates, either in the cover plate, the weld, or the girder flange.

_6___Stringer to floor beam (truss) brackets – Cracking occurs in the angles connecting floor beams to other elements. Cracks can occur anywhere within the angle connector, especially around bolts or rivets.

_0___Cantilever floor-beam brackets – These plates are laid horizontally and usually bolted or riveted to the girders. They protrude out to the sides of the bridge and are connected to the top of short beams that extend out from the bridge to increase bridge width. Cracking can occur around tack welds, bolts, rivets, or across the plate parallel to girders.

_2___Web Connection Plates – Horizontal plates welded to the girder web used for diaphragms or other attachments. Cracks usually occur when the plate intersects a transverse stiffener.

_0___Transverse Groove Welds – Groove welds on girder webs. Usually these occur at the end of horizontal stiffeners, or welds connecting long sections of horizontal stiffeners together, or the end of haunch inserts (The bottom flange is cut out of the girder and replaced with a groove-welded, higher-depth section to increase moment of inertia around supports).

_0___Web penetrations – When a member passes through the web of another. Cracking is more common when backing bars are left in place. These details are common in pier caps or box girders.

_1___Coped members – Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually is caused by too small of a radius or no radius.

_5___Tied arch (or truss) floor beams – Floor beams can exhibit separation of beam web and flange due to rotation of the beams under distortional fatigue.

_3___Continuous longitudinal welds – Commonly, these long welds connect plates or other shapes along their length, to form some sort of built-up-section. Cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds.

_0___Lamellar Tearing – Separation of layers of metal within a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

****

Part 2:
This section is more important than the first. Please provide any other details that have led to (premature) fracture and which are not included in this list. Please provide any such details even if they are no longer allowed by code or have been eliminated from your bridge inventory. Indicate the approximate number of cases of fracture as well as the factors that you believe affected the failure of this detail (Examples: tapering of ends, position on bridge, weld geometry). Do not include corrosion failures.

Modular expansion joints. We have experienced a number of failures in the center bar to support bar connection welds of some of these units. In the cases I have seen, the welds create fixity between these members and the relatively high flexibility of the system then causes significant connection moments to develop until the weld cracks and the rotational restraint goes away. Occasionally, failures in center bar groove weld splices have also occurred.

Open-metal grid decks. The welds connecting the intersecting bars of several designs have failed often. Weld quality in these usually secondary connections is sometimes an issue. But, it is thought that failure is largely due to the overall flexibility of the units coupled with significant impact loading leading to relatively high fatigue stress ranges in the under-designed and often poorly constructed welds.

Fracture of component (channel) in a built-up riveted truss tension chord. (One occurrence.) Fracture most likely initiated at a punched rivet hole. Other cases were found before members fully fractured.

Secondary truss members. Welded gusset plate connections are subjected to low stress, high cycle vibration. Such members have very little damping.

Toe of welds where web stiffeners are welded to box girder bottom flanges. The relatively thin plates may be flexing out-of-plane under traffic.

Riveted stringer-to-floorbeam brackets. We have several bridges where rivet heads have sheared off.

Tennessee

Ed Wasserman <Ed.Wasserman@STATE.TN.US

Attached is a completed survey. We have no additional details to provide in part 2.

_2____Transverse Stiffener Web Gaps * Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to both flanges of the girders; instead a gap was left anywhere from 0" (bearing) to 2" or more. Cracking can occur in any number of elements in this area, including the welds, the girder flange, the girder web, or the plate.

_4____Cover plated beams and flange gussets *Cover plates are plates attached to the underside of girder flanges to increase the moment of inertia of girders locally. Cracks usually form at the ends of the cover plates, either in the cover plate, the weld, or the girder flange.

_0____Stringer to floor beam (truss) brackets * Cracking occurs in the angles connecting floor beams to other elements. Cracks can occur anywhere within the angle connector, especially around bolts or rivets.

_3____Cantilever floor-beam brackets * These plates are laid horizontally and usually bolted or riveted to the girders. They protrude out to the sides of the bridge and are connected to the top of short beams that extend out from the bridge to increase bridge width. Cracking can occur around tack welds, bolts, rivets, or across the plate parallel to girders.

_1____Web Connection Plates * Horizontal plates welded to the girder web used for diaphragms or other attachments. Cracks usually occur when the plate intersects a transverse stiffener.

_0____Transverse Groove Welds * Groove welds on girder webs. Usually these occur at the end of horizontal stiffeners, or welds connecting long sections of horizontal stiffeners together, or the end of haunch inserts. (The bottom flange is cut out of the girder and replaced with a groove-welded, higher-depth section to increase moment of inertia around supports).

_0____Web penetrations * When a member passes through the web of another. Cracking is more common when backing bars are left in place. These details are common in pier caps or box girders.

_5____Coped members * Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually is caused by too small of a radius or no radius.

_0____Tied arch floor beams * Floor beams can exhibit separation of beam web and flange due to rotation of the beams under distortional fatigue.

_0____Continuous longitudinal welds * Commonly, these long welds connect plates or other shapes along their length, to form some sort of built-up-section. Cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds.

_0____Lamellar Tearing * Separation of layers of metal within a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

Missouri
Paul & Mr. Lindberg

Here is the completed survey to the best of our knowledge.

Carl

Paul D Porter/SC/MODOT
06/19/2006 03:09 PM
To: Carlis J Callahan/SC/MODOT@MODOT, Kenneth R Foster/SC/MODOT@MODOT
cc: Ghanshyam D Gupta/SC/MODOT@MODOT, Paul W Kelly/SC/MODOT@MODOT
Subject: Fw: Steel Fatigue Details Enumeration Research

Carl, This was sent out to be forwarded to the "Director of Inspections," but it does look like something Maintenance would have the most information on and be in the best position to respond. The researcher is looking for state's experience in ranking the relative frequency of different fatigue prone details on steel Bridges. We would appreciate a copy of any response for our information as well. Thanks.

__1___Transverse Stiffener Web Gaps – Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to both flanges of the girders; instead a gap was left anywhere from 0" (bearing) to 2" or more. Cracking can occur in any number of elements in this area.

___7__Cover plated beams and flange gussets – cracks usually form at the ends of the cover plates, either in the cover plate, the weld, or the girder flange.

__2___Stringer to floor beam (truss) brackets – Cracking occurs in the angles connecting floor beams to other elements. Cracks can occur anywhere within the angle connector.

__0___Cantilever floor-beam brackets – These plates are laid horizontally and usually bolted or riveted to the girders. They are also connected to the top of short beams that extend out from the bridge to increase bridge width. Cracking can occur around tack welds, bolts, rivets, or across the plate as shown.

__4___Web Connection Plates – horizontal plates welded to the girder web. Cracks usually occur when plate intersects a transverse stiffener.

__3___Transverse Groove Welds – Groove welds to web of girders. Usually occur at the end of horizontal stiffeners or the end of haunch inserts.

__6___Web penetrations – When a member passes through the web of another. Cracking is more common when backing bars are left in place. These details are common in pier caps.

___5__Coped members – Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually caused by small radius or no radius.

___0__Tied arch floor beams – Separation of beam web and flange by rotation of beams.

__0___Continuous longitudinal welds – cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds. Commonly, these welds connect plates or other shapes along their length.

__8___Lamellar Tearing – Separation of layers of metal within a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

New Jersey
Attached please find our response to your survey. Should you need further information, please contact Richard Dunne, Manager of Structural Engineering, at Richard.Dunne@dot.state.nj.us or (609) 530-2557.

Thank you.

X. Hannah Cheng, Ph.D., P.E.
Bureau of Structural Engineering,
New Jersey Department of Transportation
1035 Parkway Ave., Trenton, NJ 08625
Phone (609) 530-2464
Fax (609) 530-5777
Xiaohua.Cheng@dot.state.nj.us

Part 1: Below are details that are known to have caused cracking. Please rank them by the order of occurrence experienced in your state or area, where 1 is the most common, up to the least common. If you have never experienced cracking (failure) of a certain detail, use 0.

Part 2: This section is more important than the first. Please provide any other details that have led to (premature) fracture and which are not included in this list. Please provide any such details even if they are no longer allowed by code or have been eliminated from your bridge inventory. Indicate the approximate number of cases of fracture as well as the factors that you believe affected the failure of this detail (Examples: tapering of ends, position on bridge, weld geometry). Do not include corrosion failures.

BEGIN NUMBERING NOW

_#2 (Yes)__Transverse Stiffener Web Gaps – Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to both flanges of the girders; instead a gap was left anywhere from 0” (bearing) to 2” or more. Cracking can occur in any number of elements in this area.

_0 (No) – Not yet anyway_Cover plated beams and flange gussets – cracks usually form at the ends of the cover plates, either in the cover plate, the weld, or the girder flange.

_0 (No)_ Stringer to floor beam (truss) brackets – Cracking occurs in the angles connecting floor beams to other elements. Cracks can occur anywhere within the angle connector.

_0 (No)__Cantilever floor-beam brackets – These plates are laid horizontally and usually bolted or riveted to the girders. They are also connected to the top of short beams that extend out from the bridge to increase bridge width. Cracking can occur around tack welds, bolts, rivets, or across the plate as shown.

_0 (No)__Web Connection Plates – horizontal plates welded to the girder web. Cracks usually occur when plate intersects a transverse stiffener.

_#1 (Yes) – Our cracks occur in horizontal stiffener groove welds between sections of stiffeners due to poor welds (lack of fusion) Transverse Groove Welds – Groove welds to web of girders. Usually occur at the end of horizontal stiffeners or the end of haunch inserts.

_0 (No)__Web penetrations – When a member passes through the web of another. Cracking is more common when backing bars are left in place. These details are common in pier caps.

_#3 (Yes)__Coped members – Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually caused by small radius or no radius.

_0 (No)__Tied arch floor beams – Separation of beam web and flange by rotation of beams.

0 (No) - should cracking be transverse? Continuous longitudinal welds – cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds. Commonly, these welds connect plates or other shapes along their length.

_0 (No)__Lamellar Tearing – Separation of layers of metal within a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

Thank you very much for your assistance. Your input will help advance the overall understanding of fatigue susceptible details. -Adam Lindberg

Delaware
Adam,

You will find the Delaware Department of Transportation's rankings incorporated into the text below.

-Doug Robb

-----Original Message-----
From: Finney Doug (DelDOT)
Sent: Monday, July 10, 2006 3:58 PM
To: Robb Douglass (DelDOT)
Subject: RE: Steel Fatigue Details Enumeration Research - small file size

Doug,

I ranked the details below. I can't think of any that are not on the list.

Doug

-----Original Message-----
From: Robb Douglass (DelDOT)
Sent: Wednesday, July 05, 2006 1:15 PM
To: Finney Doug (DelDOT)
Subject: FW: Steel Fatigue Details Enumeration Research - small file
size

Doug,

Can you or anyone in your group provide any information to assist with this survey?

-Doug

Subject: Steel Fatigue Details Enumeration Research - small file size

This email contains no attachments or pictures, to reduce the file size for those unable to open the previous survey. If you were able to open the attachment in the previous email, delete this message. Thank you to all of the DOTs who have already submitted their input into this study. Results should be turned in preferably by the end of July.

PLEASE FORWARD THIS MESSAGE TO THE INTENDED PARTY

Attn: Director of Inspections, Department of Transportation

Regarding: Steel Fatigue Details Research

Adam Lindberg
M.S. Bridge Fatigue Researcher
University of Minnesota
Civil Engineering Department

Research is being performed at the University of Minnesota with the sponsorship of the Minnesota Department of Transportation (Mn/DOT) to assist in enumerating and ranking fatigue-susceptible details that may affect the performance of steel bridges. The goal of this research is to rank details with a history of cracking, so as to alert inspectors and facilitate budget estimations. By collecting a comprehensive list of details prone to fracture, a more precise evaluation will result, thus safety is not compromised for the sake of economy.

If you are willing to assist the collection of these fatigue-susceptible steel bridge details, please fill out the following survey. Your time is very much appreciated. If you would prefer to provide information in a different format, please contact me at xxx-xxx-xxxx or lind0990@umn.edu (preferred).

Thank you,

Adam Lindberg

Part 1: Below are details that are known to have caused cracking. Please rank them by the order of occurrence experienced in your state or area, where 1 is the most common, up to the least common. If you have never experienced cracking (failure) of a certain detail, use 0.

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BEGIN NUMBERING NOW (Non-picture version - for any further explanations, please contact Mr. Lindberg at lind0990@umn.edu)

__3___Transverse Stiffener Web Gaps - Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to both flanges of the girders; instead a gap was left anywhere from 0" (bearing) to 2" or more. Cracking can occur in any number of elements in this area, including the welds, the girder flange, the girder web, or the plate.

__1___Cover plated beams and flange gussets -Cover plates are plates attached to the underside of girder flanges to increase the moment of inertia of girders locally. Cracks usually form at the ends of the cover plates, either in the cover plate, the weld, or the girder flange.

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__0___Web Connection Plates - Horizontal plates welded to the girder web used for diaphragms or other attachments. Cracks usually occur when the plate intersects a transverse stiffener.

__4___Transverse Groove Welds - Groove welds on girder webs. Usually these occur at the end of horizontal stiffeners, or welds connecting long sections of horizontal stiffeners together, or the end of haunch inserts. (The bottom flange is cut out of the girder and replaced with a groove-welded, higher-depth section to increase moment of inertia around supports).

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__2___Coped members - Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually is caused by too small of a radius or no radius.

__0___Tied arch floor beams - Floor beams can exhibit separation of beam web and flange due to rotation of the beams under distortional fatigue.

__0___Continuous longitudinal welds - Commonly, these long welds connect plates or other shapes along their length, to form some sort of built-up-section. Cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds.

__0___Lamellar Tearing - Separation of layers of metal within a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

Thank you very much for your assistance. Your input will help advance the overall understanding of fatigue susceptible details. -Adam Lindberg

Texas
Attached is the Texas Department of Transportation's reply to the fatigue detail research. Part 2 ask for any other details that we might see that are not in the list. We have not seen any other type of details that cause fractures in Texas.

If you have any other questions, please contact me.

Alan Kowalik
Bridge Inspection Branch Manager
Texas Department of Transportation
Bridge Division
125 East 11th Street
Austin Texas, 78701

Work Phone (512) 416-2208
Fax (512) 416-2402

akowali@dot.state.tx.us

Part 1: Below are details that are known to have caused cracking. Please rank them by the order of occurrence experienced in your state or area, where 1 is the most common, up to the least common. If you have never experienced cracking (failure) of a certain detail, use 0.

Part 2: This section is more important than the first. Please provide any other details that have led to (premature) fracture and which are not included in this list. Please provide any such details even if they are no longer allowed by code or have been eliminated from your bridge inventory. Indicate the approximate number of cases of fracture as well as the factors that you believe affected the failure of this detail (Examples: tapering of ends, position on bridge, weld geometry). Do not include corrosion failures.

BEGIN NUMBERING NOW

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__2__Web penetrations – When a member passes through the web of another. Cracking is more common when backing bars are left in place. These details are common in pier caps.

__3__Coped members – Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually caused by small radius or no radius.

__0__Tied arch floor beams – Separation of beam web and flange by rotation of beams.

__0_Continuous longitudinal welds – cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds. Commonly, these welds connect plates or other shapes along their length.

__0__Lamellar Tearing – Separation of layers of metal within a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

Thank you very much for your assistance. Your input will help advance the overall understanding of fatigue susceptible details. -Adam Lindberg

Montana
Adam,

I am sorry it took me so long to get this to you.

<<FW Steel Fatigue Details.txt>>

Mike Murphy, P.E.
Bridge Management Engineer
Montana Department of Transportation
PO Box 201001
2701 Prospect
Helena, MT 59620-1001
406-444-6264
Fax 406-444-6155
From: Barnes, Kent
Sent: Thursday, June 22, 2006 1:38 PM
To: Murphy, Mike
Subject: FW: Steel Fatigue Details Enumeration Research - small file size

Follow Up Flag: Follow up
Flag Status: Red

Mike, is this something you can help with?

Thanks,
Kent

-----Original Message-----
From: lind0990@umn.edu [mailto:lind0990@umn.edu]
Sent: Thursday, June 22, 2006 10:55 AM
Subject: Steel Fatigue Details Enumeration Research - small file size

This email contains no attachments or pictures, to reduce the file size for those unable to open the previous survey. If you were able to open the attachment in the previous email, delete this message. Thank you to all of the DOTs who have already submitted their input into this study. Results should be turned in preferably by the end of July.

PLEASE FORWARD THIS MESSAGE TO THE INTENDED PARTY

Attn: Director of Inspections, Department of Transportation

Regarding: Steel Fatigue Details Research

Adam Lindberg

M.S. Bridge Fatigue Researcher
University of Minnesota
Civil Engineering Department

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If you are willing to assist the collection of these fatigue-susceptible steel bridge details, please fill out the following survey. Your time is very much appreciated. If you would prefer to provide information in a different format, please contact me at xxx-xxx-xxxx or lind0990@umn.edu (preferred).

Thank you,

Adam Lindberg

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BEGIN NUMBERING NOW (Non-picture version – for any further explanations, please contact Mr. Lindberg at lind0990@umn.edu)

__5___Transverse Stiffener Web Gaps – Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to both flanges of the girders; instead a gap was left anywhere from 0” (bearing) to 2” or more. Cracking can occur in any number of elements in this area, including the welds, the girder flange, the girder web, or the plate.

__1___Cover plated beams and flange gussets –Cover plates are plates attached to the underside of girder flanges to increase the moment of inertia of girders locally. Cracks usually form at the ends of the cover plates, either in the cover plate, the weld, or the girder flange.

__3___Stringer to floor beam (truss) brackets – Cracking occurs in the angles connecting floor beams to other elements. Cracks can occur anywhere within the angle connector, especially around bolts or rivets.

__7___Cantilever floor-beam brackets – These plates are laid horizontally and usually bolted or riveted to the girders. They protrude out to the sides of the bridge and are connected to the top of short beams that extend out from the bridge to increase bridge width. Cracking can occur around tack welds, bolts, rivets, or across the plate parallel to girders.

__2___Web Connection Plates – Horizontal plates welded to the girder web used for diaphragms or other attachments. Cracks usually occur when the plate intersects a transverse stiffener.

__6___Transverse Groove Welds – Groove welds on girder webs. Usually these occur at the end of horizontal stiffeners, or welds connecting long sections of horizontal stiffeners together, or the end of haunch inserts (The bottom flange is cut out of the girder and replaced with a groove-welded, higher-depth section to increase moment of inertia around supports).

__0___Web penetrations – When a member passes through the web of another. Cracking is more common when backing bars are left in place. These details are common in pier caps or box girders.

__9___Coped members – Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually is caused by too small of a radius or no radius.

__0___Tied arch floor beams – Floor beams can exhibit separation of beam web and flange due to rotation of the beams under distortional fatigue.

__4___Continuous longitudinal welds – Commonly, these long welds connect plates or other shapes along their length, to form some sort of built-up-section. Cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds.

__8___Lamellar Tearing – Separation of layers of metal within a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

Thank you very much for your assistance. Your input will help advance theoverall understanding of fatigue susceptible details. -Adam Lindberg

Part 2:

We have had some very serious cracks occur in welded plate girders, which initiated at the intersection of or near intersection of fillet welds connecting longitudinal stiffeners and transverse stiffeners. The cracks were sudden and explosive starting near the weld intersections and heading up to very near the top flange and down and through a good portion (2/3's) of the bottom flange.

See a report entitled "Evaluation and Retrofit of Highway Bridges to Prevent Constraint-Induced Fracture From Web Attachments" by William J. Wright , Turner-Fairbanks Highway Research Center, and John W. Fisher, Robert Conner, Lehigh University ATLSS Center, to get a good description of what we feel happened.

US Army Corps of Engineers – New England Division
Mr. Lindberg:

Your message was forwarded to me from North Atlantic Division, US Army Corps of Engineers. I've filled out the survey form below.

John Kedzierski, P.E.
Sr. Structural Engineer/
Bridge Inspction Program Manager
New England District
US Army Corps of Engineers
978-318-8521

Subject: Steel Fatigue Details Enumeration Research - small file size

This email contains no attachments or pictures, to reduce the file size for those unable to open the previous survey. If you were able to open the attachment in the previous email, delete this message. Thank you to all of the DOTs who have already submitted their input into this study. Results should be turned in preferably by the end of July.

PLEASE FORWARD THIS MESSAGE TO THE INTENDED PARTY

Attn: Director of Inspections, Department of Transportation

Regarding: Steel Fatigue Details Research

Adam Lindberg
M.S. Bridge Fatigue Researcher
University of Minnesota
Civil Engineering Department

Research is being performed at the University of Minnesota with the sponsorship of the Minnesota Department of Transportation (Mn/DOT) to assist in enumerating and ranking fatigue-susceptible details that may affect the performance of steel bridges. The goal of this research is to rank details with a history of cracking, so as to alert inspectors and facilitate budget estimations. By collecting a comprehensive list of details prone to fracture, a more precise evaluation will result, thus safety is not compromised for the sake of economy.

If you are willing to assist the collection of these fatigue-susceptible steel bridge details, please fill out the following survey. Your time is very much appreciated. If you would prefer to provide information in a different format, please contact me at xxx-xxx-xxxx or lind0990@umn.edu (preferred).

Thank you,

Adam Lindberg

Part 1: Below are details that are known to have caused cracking. Please rank them by the order of occurrence experienced in your state or area, where 1 is the most common, up to the least common. If you have never experienced cracking (failure) of a certain detail, use 0.

Part 2: This section is more important than the first. Please provide any other details that have led to (premature) fracture and which are not included in this list. Please provide any such details even if they are no longer allowed by code or have been eliminated from your bridge inventory. Indicate the approximate number of cases of fracture as well as the factors that you believe affected the failure of this detail (Examples: tapering of ends, position on bridge, weld geometry). Do not include corrosion failures.

BEGIN NUMBERING NOW (Non-picture version - for any further explanations, please contact Mr. Lindberg at lind0990@umn.edu)

__0___Transverse Stiffener Web Gaps - Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to both flanges of the girders; instead a gap was left anywhere from 0" (bearing) to 2" or more. Cracking can occur in any number of elements in this area, including the welds, the girder flange, the girder web, or the plate.

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__0___Stringer to floor beam (truss) brackets - Cracking occurs in the angles connecting floor beams to other elements. Cracks can occur anywhere within the angle connector, especially around bolts or rivets.

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__0___Transverse Groove Welds - Groove welds on girder webs. Usually these occur at the end of horizontal stiffeners, or welds connecting long sections of horizontal stiffeners together, or the end of haunch inserts (The bottom flange is cut out of the girder and replaced with a groove-welded, higher-depth section to increase moment of inertia around supports).

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__0___Continuous longitudinal welds - Commonly, these long welds connect plates or other shapes along their length, to form some sort of built-up-section. Cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds.

__0___Lamellar Tearing - Separation of layers of metal within a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

Thank you very much for your assistance. Your input will help advance the overall understanding of fatigue susceptible details. -Adam Lindberg

Sorry, please see the attached documents

Thank you for your interest in furthering this study,

Adam Lindberg

Wyoming
On Aug 9 2006, Gregg Fredrick wrote:

> Please see the attachments, as a response from the Wyoming Department of
> Transportation.
>
> Let me know if you have any questions.
>
>
>
> Gregg C. Fredrick, P.E.
> State Bridge Engineer
> Wyoming Department of Transportation
>
>
>>>> <lind0990@umn.edu> 6/19/2006 12:53 PM >>>
> PLEASE FORWARD THIS MESSAGE TO THE INTENDED PARTY
>
> Attn: Director of Inspections, Department of Transportation
>
> Regarding: Steel Fatigue Details Research
>
>   Adam Lindberg
>   M.S. Bridge Fatigue Researcher
>   University of Minnesota
>   Civil Engineering Department
>
> Research is being performed at the University of Minnesota with the
> sponsorship of the Minnesota Department of Transportation (Mn/DOT) to

> assist in enumerating and ranking fatigue-susceptible details that may
> affect the performance of steel bridges. The goal of this research is
> to rank details with a history of cracking, so as to alert inspectors and
> facilitate budget estimations. By collecting a comprehensive list of
> details prone to fracture, a more precise evaluation will result, thus
> safety is not compromised for the sake of economy.
>
> If you are willing to assist the collection of these fatigue-susceptible
> steel bridge details, please fill out the attached document. Your time
> is very much appreciated. If you would prefer to provide information in a
> different format, please contact me at xxx-xxx-xxxx or lind0990@umn.edu
> (preferred).
>
> Thank you,
>

>
> Adam Lindberg
>
Part 1: Below are details that are known to have caused cracking. Please rank them by the order of occurrence experienced in your state or area, where 1 is the most common, up to the least common. If you have never experienced cracking (failure) of a certain detail, use 0.

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BEGIN NUMBERING NOW

1 Transverse Stiffener Web Gaps – Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to both flanges of the girders; instead a gap was left anywhere from 0” (bearing) to 2” or more. Cracking can occur in any number of elements in this area.

3 Cover plated beams and flange gussets – cracks usually form at the ends of the cover plates, either in the cover plate, the weld, or the girder flange.

4 Stringer to floor beam (truss) brackets – Cracking occurs in the angles connecting floor beams to other elements. Cracks can occur anywhere within the angle connector.

0 Cantilever floor-beam brackets – These plates are laid horizontally and usually bolted or riveted to the girders. They are also connected to the top of short beams that extend out from the bridge to increase bridge width. Cracking can occur around tack welds, bolts, rivets, or across the plate as shown.

0 Web Connection Plates – horizontal plates welded to the girder web. Cracks usually occur when plate intersects a transverse stiffener.

0 Transverse Groove Welds – Groove welds to web of girders. Usually occur at the end of horizontal stiffeners or the end of haunch inserts.

0 Web penetrations – When a member passes through the web of another. Cracking is more common when backing bars are left in place. These details are common in pier caps.

2 Coped members – Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually caused by small radius or no radius.

0 Tied arch floor beams – Separation of beam web and flange by rotation of beams.

0 Continuous longitudinal welds – cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds. Commonly, these welds connect plates or other shapes along their length.

5 Lamellar Tearing – Separation of layers of metal within a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

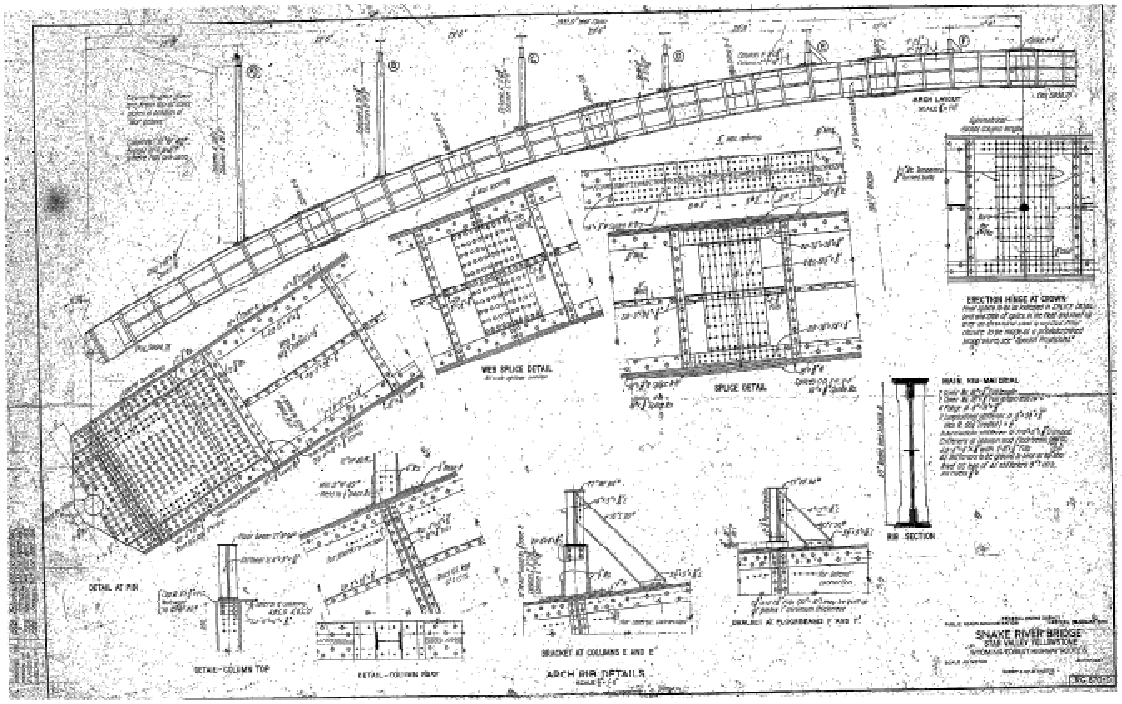
Thank you very much for your assistance. Your input will help advance the overall understanding of fatigue susceptible details.
-Adam Lindberg

Part 2:

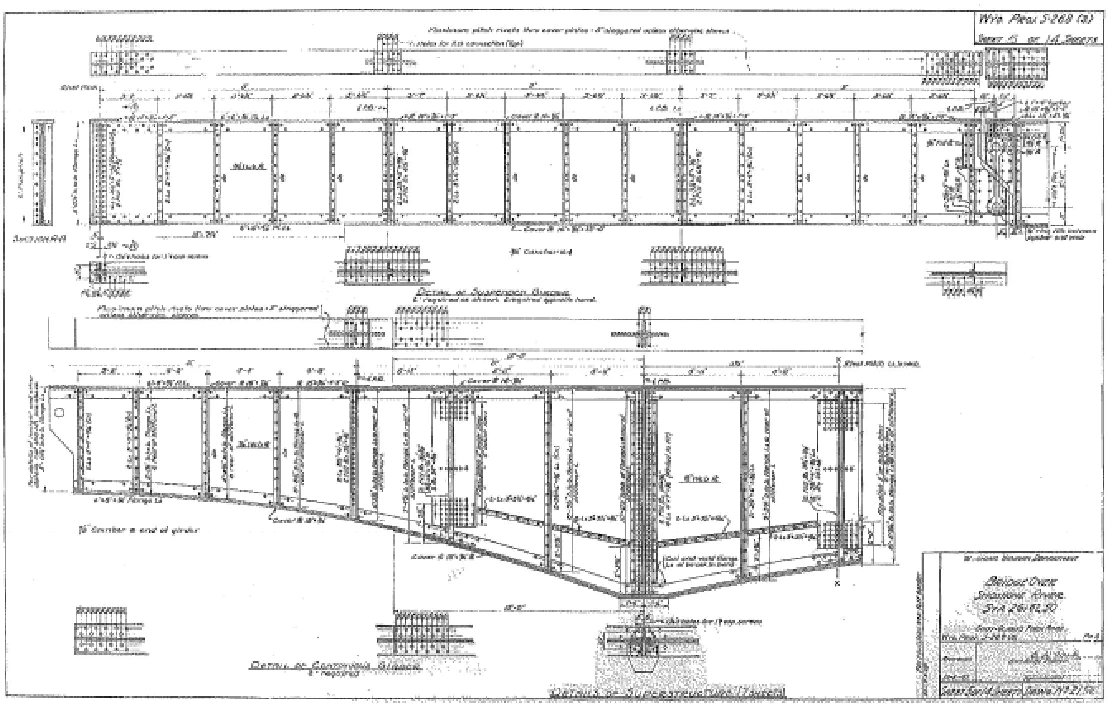
Wyoming has experienced fatigue failures with two details not included in the survey. The attached structural drawings include these details.

1) The first is the bracket detail at columns E and E' as shown in the middle of the lower half of drawing RG870-D. This bracket supports a floor beam above a steel arch. It is attached to the steel arch by an angle section with a single row of bolts. Tension in the bracket due to movement of the floor beam created a prying action on the angle-to-arch connection which ultimately caused the angle to fracture through the single line of bolts.

2) The second is a dog-bone-shaped hanger as pictured in the suspended span details of Drawing Number 2156. After some years in service, two of these hangers fractured across the width of the member where the round portion transitions to the straight sided shaft. It is believed that the geometrical transition of this member was abrupt enough to cause a stress concentration sufficient to fail the hanger. These hangers were replaced with units having straight sides for full length thereby eliminating the stress riser. The new hangers performed as required until the bridge was replaced many years later.



WEB SPLICE DETAIL
SPLICE DETAIL
ERECTION HINGE AT CROWN
RIB SECTION
DETAIL AT PIN
DETAIL-COLUMN TOP
BRACKET AT COLUMNS E AND E'
ARCH RIB DETAILS
SNAKE RIVER BRIDGE
STAR VALLEY YELLOWSTONE



Wyo. Proj. 5-269 (3)
Detail of Suspension Girder
Detail of Continuous Girder
Details of Superstructure

<u>California</u>
My name is Rosme Aguilar and I am the Supervisor of the California Department of Transportation (Caltrans) Fracture Critical Inspection Team. We are willing to assist with the collection of data for your research and more importantly on getting a copy of you results. I believe that the presentation of the information in the format shown in your survey is very useful for the training of new inspectors and anybody associated with Fracture Critical Inspections. I have seen a similar format on a report of a researched performed by The Welding Institute under NCHRP project 12-27. I have a partial copy of this report (It is missing the first pages) and I notice that some of your pictures are similar to the one presented in this report. Unfortunately, we do not have statistics on the number of times we have found specific crack details. That is one of the project I have in my to do list. I have been thinking about that for some time but have not have the manpower to do it. If it is of any use to you, we can provide you with a list the cracked fatigue-susceptible details we have found but I cannot give you exact numbers. Please let me know if this is OK and when do you need it. Thanks.

Barton Newton

To: michael.b.johnson@dot.ca.gov@DOT, Rosme Aguilar/HQ/Caltrans/CAGov@DOT
06/19/2006 12:19 cc: PM
Subject: Fw: Steel Fatigue Details Enumeration Research
Forwarded by Barton Newton/HQ/Caltrans/CAGov on 06/19/2006 12:18 PM

Mr. Lindberg,

Sorry for taking so long. Attached is our response to your survey.

(See attached file: Steel Fatigue Details Research.doc)

Rosme Aguilar
(916) 227-0719 Office
(916) 799-2954 Cell

Part 1: Below are details that are known to have caused cracking. Please rank them by the order of occurrence experienced in your state or area, where 1 is the most common, up to the least common. If you have never experienced cracking (failure) of a certain detail, use 0.

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BEGIN NUMBERING NOW

__Most common_ Transverse Stiffener Web Gaps – Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to both flanges of the girders; instead a gap was left anywhere from 0" (bearing) to 2" or more. Cracking can occur in any number of elements in this area.

_Some cases___ Cover plated beams and flange gussets – cracks usually form at the ends of the cover plates, either in the cover plate, the weld, or the girder flange.

__Some cases___ Stringer to floor beam (truss) brackets – Cracking occurs in the angles connecting floor beams to other elements. Cracks can occur anywhere within the angle connector.

_Few cases____ Cantilever floor-beam brackets – These plates are laid horizontally and usually bolted or riveted to the girders. They are also connected to the top of short beams that extend out from the bridge to increase bridge width. Cracking can occur around tack welds, bolts, rivets, or across the plate as shown.

__Few cases___ Web Connection Plates – horizontal plates welded to the girder web. Cracks usually occur when plate intersects a transverse stiffener.

___No__ Transverse Groove Welds – Groove welds to web of girders. Usually occur at the end of horizontal stiffeners or the end of haunch inserts.

Have had cracking from flame cut holes (Very common) Web penetrations – When a member passes through the web of another. Cracking is more common when backing bars are left in place. These details are common in pier caps.

__Some cases___ Coped members – Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually caused by small radius or no radius.

__Some cases___Tied arch floor beams – Separation of beam web and flange by rotation of beams.

__No___ Continuous longitudinal welds – cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds. Commonly, these welds connect plates or other shapes along their length.

_No____ Lamellar Tearing – Separation of layers of metal within a solid piece. Cracking is usually found in highly restrained members or in cantilevers.

Thank you very much for your assistance. Your input will help advance the overall understanding of fatigue susceptible details. -Adam Lindberg

Arkansas

Part 1: Below are details that are known to have caused cracking. Please rank them by the order of occurrence experienced in your state or area, where 1 is the most common, up to the least common. If you have never experienced cracking (failure) of a certain detail, use 0.

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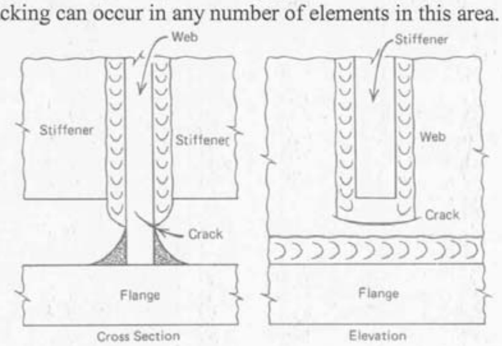
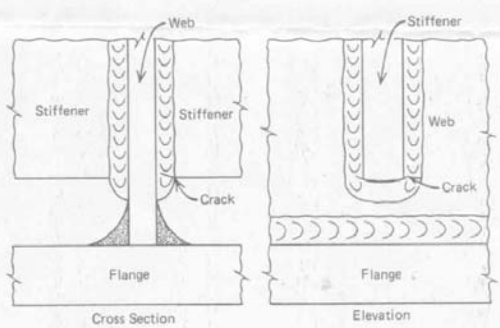
BEGIN NUMBERING NOW

__1__ *Transverse Stiffener Web Gaps* – Diaphragms or cross-braces between girders are connected to plates which are welded to the girders. Prior to 1985 these plates were not welded to *both* flanges of the girders; instead a gap was left anywhere from 0" (bearing) to 2" or more. Cracking can occur in any number of elements in this area.

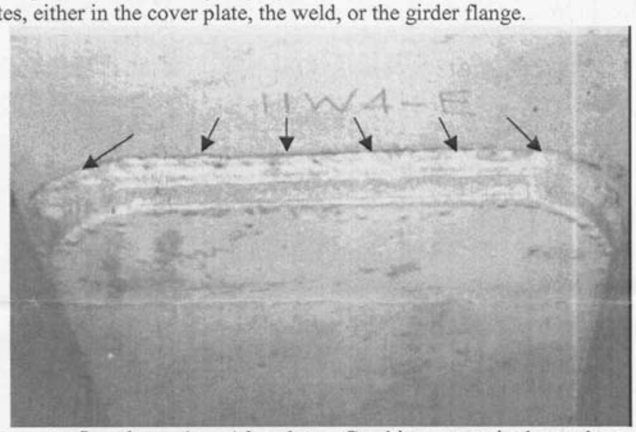


(a)

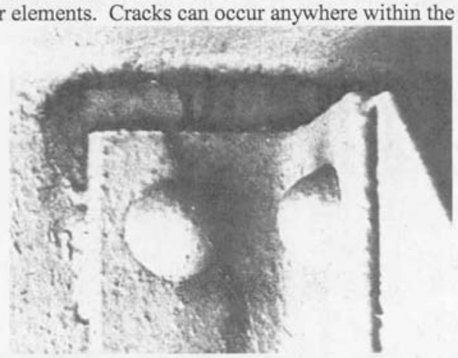


(b)

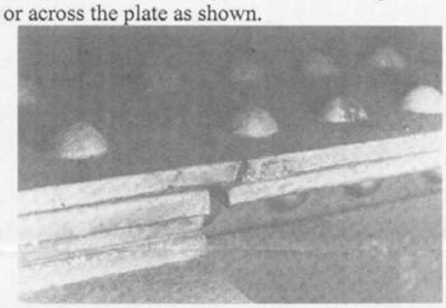
__5__ *Cover plated beams and flange gussets* – cracks usually form at the ends of the cover plates, either in the cover plate, the weld, or the girder flange.



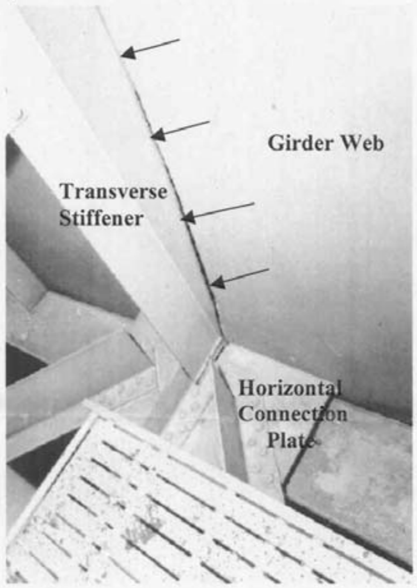
__2__ *Stringer to floor beam (truss) brackets* – Cracking occurs in the angles connecting floor beams to other elements. Cracks can occur anywhere within the angle connector.



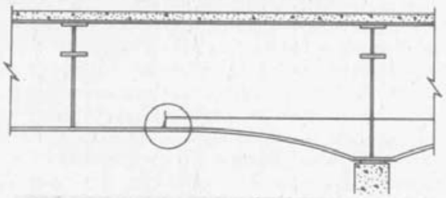
__6__ *Cantilever floor-beam brackets* – These plates are laid horizontally and usually bolted or riveted to the girders. They are also connected to the top of short beams that extend out from the bridge to increase bridge width. Cracking can occur around tack welds, bolts, rivets, or across the plate as shown.

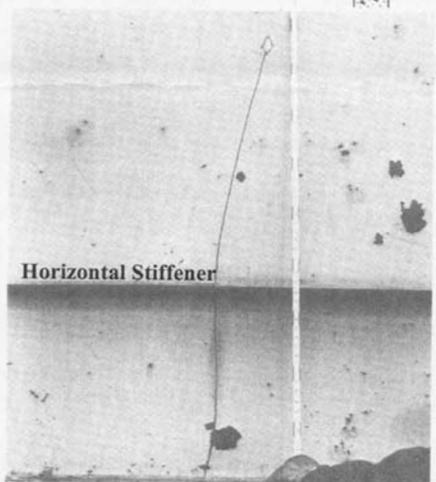


__4__ *Web Connection Plates* – horizontal plates welded to the girder web. Cracks usually occur when plate intersects a transverse stiffener.

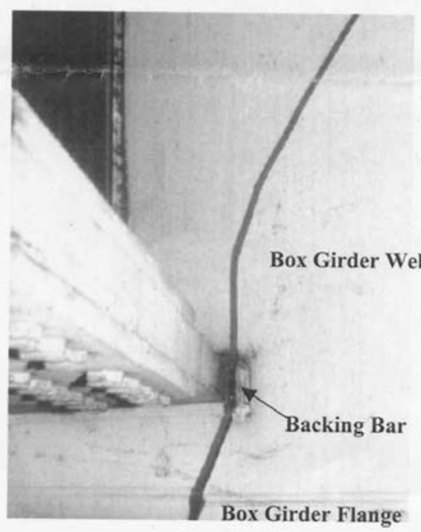


__0__ *Transverse Groove Welds* – Groove welds to web of girders. Usually occur at the end of horizontal stiffeners or the end of haunch inserts.



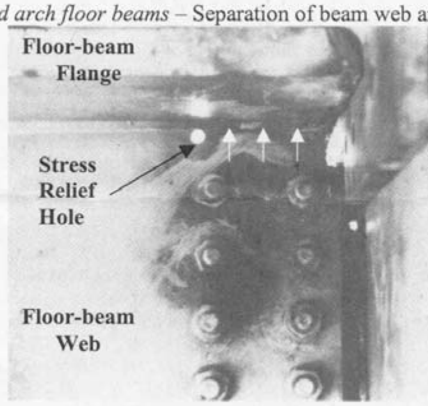


__7__ *Web penetrations* – When a member passes through the web of another. Cracking is more common when backing bars are left in place. These details are common in pier caps.

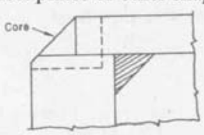


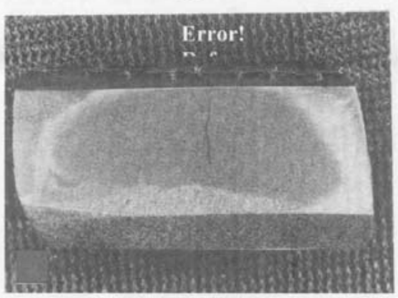
__3_ *Coped members* – Cracks initiating from the fillet of the two re-entrant cuts. Cracking usually caused by small radius or no radius.

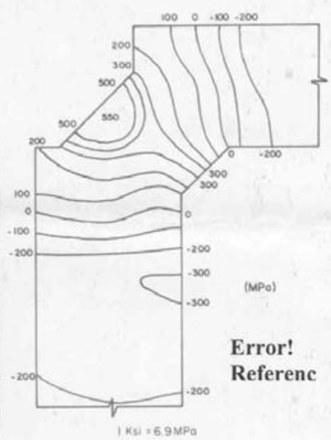
__0__ *Tied arch floor beams* – Separation of beam web and flange by rotation of beams.



__0__ *Continuous longitudinal welds* – cracking occurs parallel to the longitudinal welds. Cracking is usually caused by improper welding or too large of welds. Commonly, these welds connect plates or other shapes along their length.

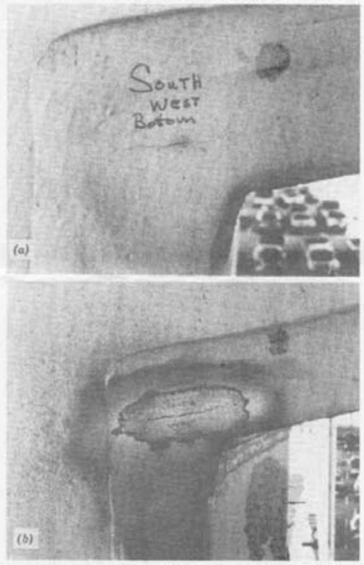






Error!
Referenc

__0_ *Lamellar Tearing* – Separation of layers of metal *within* a solid piece. Cracking is usually found in highly restrained members or in cantilevers.



Thank you very much for your assistance. Your input will help advance the overall understanding of fatigue susceptible details.
-Adam Lindberg

No additional details that resulted in premature fracture. "Transverse Stiffener Web Gaps" and "Stringer to floor beam brackets" are details that cause us the most problems.