FHWA/IN/JTRP-2009/19

Final Report

**FATIGUE STRENGTH OF STEEL BRIDGE
MEMBERS WITH INTERSECTING WELDS**

**Davit T. DeLong
Mark D. Bowman**

July 2010

INDOT Research

TECHNICAL *Summary*

Technology Transfer and Project Implementation Information

TRB Subject Code: Design

Publication No. FHWA/IN/JTRP-2009-19, SPR-2942

July 2010

Final Report

Fatigue Strength of Steel Bridge Members with Increasing Welds

Introduction

A number of factors can be responsible for the development of a brittle fracture in steel girder highway bridge structures. Some of the common factors that often contribute to a brittle fracture include the use of low fracture toughness steel, environments with low service temperatures, high-rate loading rates, high residual stresses from welding, extensive constraint, and loading environments that cause significant service-level stresses. The use of welded details with several members or components that come together at a given location can often introduce many of the factors that have a high susceptibility to brittle fracture.

The brittle fracture of the Hoan Bridge in December 2000 raised further concerns about the fracture susceptibility of highway bridge details that contain intersecting welds. In an effort to assess the severity of the Hoan type detail, the FHWA requested that all states inspect their inventory of bridges to determine the number of bridges in service with intersecting welds and geometries, including those similar to the Hoan Bridge detail. The survey of bridge structures in Indiana indicated that four different intersecting, or nearly intersecting, weld details were currently being used on state or

interstate highway bridges.

The purpose of this study was to examine the effect of intersecting weld details commonly used in Indiana on the fatigue strength and resistance. While the concern was initially centered on the fracture susceptibility of intersecting weld details, any detail that has welds that intersect or nearly intersect will restrain the steel in the vicinity of the weld intersection and will introduce additional welding residual stresses. A question then arises as to the severity of the detail, even though it does not fit the exact nature of the Hoan type detail. It is possible that an intersecting weld detail with less constraint than the Hoan bridge detail may lead to fatigue crack initiation earlier than otherwise anticipated, but not trigger a brittle fracture. To assess this condition a series of experimental test were conducted to examine the fatigue susceptibility of intersecting weld details commonly used in Indiana. Furthermore, the performance of drilled hole retrofits used to extend the fatigue life of these details was studied.

Findings

Nine steel beams containing details with varying degrees of weld intersection were tested under constant amplitude cyclic loading. Three different basic detail types were tested: a vertical stiffener welded to the web and flange, a welded horizontal stiffener terminating near a welded vertical stiffener, and a welded horizontal gusset plate coped to fit around a welded vertical connection plate. The fatigue strength of the details with intersecting welds was compared to that of details

without intersecting welds. For each detail type, there were several conditions tested with varying gaps between perpendicular welds. The results of the tests were examined to determine if the web gap size had an effect on fatigue behavior, and more importantly, to determine if this effect could result in a fatigue strength below the appropriate design fatigue strength. For some of the details, the effectiveness of drilled hole retrofits in extending the fatigue life of details that had

developed fatigue cracking was also examined.

Based on the experimental results and accompanying evaluation of the details in this study, the following observations and conclusions can be made:

- No reduction in fatigue strength was detected for intersecting welds connecting a vertical stiffener to the web and flange of a rolled shape. Fatigue tests reported in the literature for similar details on welded girders with longitudinal flange to web welds also sustained little decrease in fatigue strength.
- The web gap for horizontal gusset plates that intersect a vertical stiffener or connection plate did not appear to cause a significant reduction in the fatigue

strength. The critical location was found to be the outside edges of the gusset plates.

- The size of the web gap region for horizontal stiffener details that terminates near a vertical stiffener may affect the fatigue strength. Insufficient data were available to conclude that details with small web gaps have lower fatigue strength than details with large web gaps.
- Retrofit holes were effective in extending the cyclic life of details that had developed fatigue cracks. The retrofit hole diameter should be sized to prevent crack re-initiation if possible. In spite of not being able to achieve this condition for the tests conducted, it was found that on average an additional 64 percent of the loading cycles could be applied before crack re-initiation occurred.

Implementation

Based upon the experimental results and evaluation conducted in this study, the following actions are recommended for implementation on steel highway bridge structures in Indiana.

- First, no retrofit action is needed for welded stiffener or connection plate details where the vertical web welds intersect, or nearly intersect, with horizontal welds on the flange.
- Second, horizontal web attachment details with welds that are situated near a vertical connection plate or stiffener should be regularly inspected to determine if fatigue cracking has occurred at the end of the attachment plate weld toe that is situated next to the vertical plate.
- Third, horizontal gusset plate details that are coped to fit around a vertical connection plate should be inspected to determine if fatigue cracking has occurred at the weld toe located at the outside ends

of the gusset plate. The weld toe in the web gap region where the plate is coped should also be inspected to determine if fatigue cracking has occurred.

- Fourth, fatigue cracks that are detected can typically be repaired using drilled retrofit holes. The retrofit hole diameter should be sized large enough to minimize re-initiation of the fatigue cracking. If the hole diameter required is too large, then a bolted splice repair should be considered as an effective alternative.
- Lastly, web gaps in regions where multiple welds intersect, or nearly intersect, should be modified if necessary to increase the web gap distance to ¼-in or larger between adjacent weld toes. This distance should be sufficient to minimize the likelihood of constraint-induced brittle fracture.

Contacts

For more information:

Professor Mark D. Bowman

Principal Investigator
School of Civil Engineering
Purdue University
550 Stadium Mall Drive
West Lafayette IN 47907
Phone: (765) 494-2220
E-mail: bowmanmd@purdue.edu

Indiana Department of Transportation

Division of Research
1205 Montgomery Street
P.O. Box 2279
West Lafayette, IN 47906
Phone: (765) 463-1521
Fax: (765) 497-1665

Purdue University

Joint Transportation Research Program
School of Civil Engineering
550 Stadium Mall Drive
West Lafayette, IN 47907
Phone: (765) 494-9310
Fax: (765) 496-7996
E-mail: jtrp@ecn.purdue.edu
<http://www.purdue.edu/jtrp>

Final Report

FHWA/IN/JTRP – 2009/19

Fatigue Strength of Steel Bridge Members with Intersecting Welds

by

David T. DeLong
Graduate Research Assistant

and

Mark D. Bowman
Professor of Civil Engineering

School of Civil Engineering
Purdue University

Joint Transportation Research Program
Project No: C-36-56YYY
File No: 7-4-76
SPR-2942

Conducted in Cooperation with the
Indiana Department of Transportation
And the U.S. Department of Transportation
Federal Highway Administration

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Federal Highway Administration or the Indiana Department of Transportation. This report does not constitute a standard, specification, or regulation.

Purdue University
West Lafayette, Indiana
July 2010

TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No. FHWA/IN/JTRP-2009/19	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle Fatigue Strength of Steel Bridge Members with Intersecting Welds		5. Report Date July 2010	
		6. Performing Organization Code	
7. Author(s) David T. DeLong and Mark D. Bowman		8. Performing Organization Report No. FHWA/IN/JTRP-2009/19	
9. Performing Organization Name and Address Joint Transportation Research Program 550 Stadium Mall Drive Purdue University West Lafayette, Indiana 47907-2051		10. Work Unit No.	
		11. Contract or Grant No. SPR-2942	
12. Sponsoring Agency Name and Address Indiana Department of Transportation State Office Building 100 North Senate Avenue Indianapolis, IN 46204		13. Type of Report and Period Covered Final Report	
		14. Sponsoring Agency Code	
15. Supplementary Notes Prepared in cooperation with the Indiana Department of Transportation and Federal Highway Administration.			
16. Abstract The brittle fracture of the Hoan Bridge in December 2000 highlighted the fracture susceptibility of details containing intersecting welds in steel bridges. Though it is now known that details with intersecting welds should be avoided, many existing bridges in Indiana contain details with intersecting welds. Little experimental data on the fatigue performance of these details exists. The purpose of this study was to evaluate the fatigue strength of several details commonly found in steel bridges in Indiana that contain intersecting welds. Large-scale experimental fatigue testing were performed using nine steel beams to study the behavior of three basic types of welded details: a vertical stiffener, a horizontal stiffener terminating near a vertical stiffener, and a horizontal gusset plate coped to fit around a vertical stiffener. The welded details in this study were fabricated with varying degrees of web gap, defined as the distance between perpendicular weld toes. The results of the experimental testing were examined and compared to prior testing to determine if the size of the web gap affected the fatigue strength. Furthermore, the performance of drilled hole retrofits used to extend the fatigue life of these details was studied. Within the scope of the fatigue testing program, the presence of intersecting welds was not found to significantly affect the fatigue strength of vertical stiffener and horizontal gusset details. However, the size of the web gap may have had an effect on the fatigue strength of details where a horizontal stiffener terminated near a vertical stiffener. Further experimental testing is needed to more clearly define the relationship between web gap and fatigue strength for this case. Drilled hole retrofits were successful at prolonging the lives of these details to varying degrees, but it should be noted that the stress redistribution resulting from these holes may cause fatigue cracking at nearby weld toes.			
17. Key Words Steel, fatigue, fracture, weld, bridge, girder, stiffener, gusset plate, retrofit, drilled holes, remaining life		18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 204	22. Price

ACKNOWLEDGMENTS

This project was financially supported by the Indiana Department of Transportation (INDOT) in cooperation with Joint Transportation Research Program. Special thanks are extended to individuals associated with both of these organizations for making this research possible. The members of the Research Project Study Advisory Committee are kindly acknowledged for their guidance and input throughout the project study. Included are Bill Dittrich, Sherwood Garrison, Donald Leonard, James Mickler, Scott Newbolds, and George Snyder of the Indiana Department of Transportation, and Keith Hoernschemeyer of the Federal Highway Administration. The efforts of Mr. Harry Tidrick, Bowen Laboratory Technician, are gratefully acknowledged for his ongoing guidance and assistance in the experimental testing portion of this study.

Finally, the authors would like to thank Mr. Thomas McQuillan for assistance with the literature search and in developing plans for the testing frame, Dr. Necip Onder Akinci for assistance in setting up the data acquisition instrumentation for the tests, and several other Civil Engineering graduate students who assisted in conducting various tasks related to the experimental testing.

TABLE OF CONTENTS

	Page
LIST OF TABLES	vii
LIST OF FIGURES	ix
CHAPTER 1. INTRODUCTION	1
1.1. Problem Statement	1
1.2. Objectives and Scope	4
CHAPTER 2. LITERATURE REVIEW	6
2.1. Case Studies	6
2.1.1. Horizontal Stiffeners Welded to the Web	6
2.1.2. Horizontal Gusset and Transverse Stiffener Welded to the Web	7
2.2. Analytical and Experimental Studies	11
2.2.1. Transverse Stiffener Welded to the Web and Flange	11
2.2.2. Horizontal Stiffener Welded to the Web	14
2.2.3. Horizontal Gusset and Vertical Stiffener Welded to the Web	15
2.3. Specification Requirements	19
CHAPTER 3. EXPERIMENTAL PROGRAM	46
3.1. Design Variables	46
3.1.1. Detail Type	46
3.1.2. Stress Range	47
3.1.3. Detail Geometry	47
3.1.4. Test Variables	47
3.2. Test Arrangement	47
3.2.1. Vertical Stiffener Specimens	48
3.2.2. Horizontal Stiffener Specimens	49
3.2.3. Laterally Braced Specimens	49

3.3. Fabrication of Welds	51
3.4. Test Procedure.....	52
3.5. Data Collection	52
CHAPTER 4. EXPERIMENTAL RESULTS	73
4.1. VS Test Series	73
4.1.1. Specimen VS-1	73
4.1.1.1. Crack History	74
4.1.1.2. Data Collection	75
4.1.2. Specimen VS-2	75
4.1.2.1. Crack History	76
4.1.2.2. Data Collection	77
4.1.3. Specimen VS-3	77
4.1.3.1. Crack History	78
4.1.3.2. Data Collection	79
4.2. HS Test Series	79
4.2.1. Specimen HS-1	79
4.2.1.1. Crack History	80
4.2.1.2. Data Collection	81
4.2.2. Specimen HS-2	81
4.2.2.1. Crack History	82
4.2.2.2. Data Collection	82
4.3. HG Test Series	83
4.3.1. Specimen HG-1	83
4.3.1.1. Crack History	83
4.3.1.2. Data Collection	84
4.3.2. Specimen HG-2.....	84
4.3.2.1. Crack History	85
4.3.2.2. Data Collection	85
4.3.3. Specimen HG-3.....	86
4.3.3.1. Crack History	87

4.3.3.2. Data Collection	87
4.3.4. Specimen HG-4.....	88
4.3.4.1. Crack History	89
4.3.4.2. Data Collection	89
CHAPTER 5. DISCUSSION OF EXPERIMENTAL BEHAVIOR	137
5.1. Experimental Behavior of Welded Fatigue Details	137
5.1.1. VS Series.....	138
5.1.1.1. Specimen VS-1	139
5.1.1.2. Specimen VS-2	140
5.1.1.3. Specimen VS-3	140
5.1.2. HS Series.....	141
5.1.2.1. Specimen HS-1	142
5.1.2.2. Specimen HS-2	143
5.1.3. HG Series	143
5.1.3.1. Specimen HG-1	144
5.1.3.2. Specimen HG-2.....	145
5.1.3.3. Specimen HG-3	145
5.1.3.4. Specimen HG-4.....	147
5.2. Experimental Behavior of Drilled Hole Retrofits	147
5.2.1. Stress Intensity Factors	148
5.2.2. Retrofit Hole Lives.....	149
5.3. Bolted Splice Retrofit.....	151
5.4. Summary Comments	151
5.4.1. VS Series.....	151
5.4.2. HS Series.....	151
5.4.3. HG Series	152
5.4.4. Retrofit Holes	153
CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS	173
6.1. Summary and Conclusions.....	173
6.2. Implementation Recommendations.....	175

6.3. Future Research Needs.....	176
BIBLIOGRAPHY	177
APPENDICES	
Appendix A. MATERIAL PROPERTIES.....	180
Appendix B. WEB GAP MEASUREMENTS.....	184
Appendix C. CRACK SIZE AND NUMBER OF CYCLES	193

LIST OF TABLES

Table	Page
Table 3.1 Experimental Program	53
Table 4.1 Classification of details on fatigue specimens	91
Table 4.2 Mean and standard deviation of the data collected from Specimen VS-1	92
Table 4.3 Mean and standard deviation of the data collected from Specimen VS-2	92
Table 4.4 Mean and standard deviation of the data collected from Specimen VS-3	92
Table 4.5 Mean and standard deviation of the data collected from Specimen HS-1	92
Table 4.6 Mean and standard deviation of the data collected from Specimen HS-2	92
Table 4.7 Mean and standard deviation of the flange strain data collected from Specimen HG-1	93
Table 4.8 Mean and standard deviation of the load, stroke, and web strain data collected from Specimen HG-1	93
Table 4.9 Mean and standard deviation of the flange strain data collected from Specimen HG-2 prior to first cracking	93
Table 4.10 Mean and standard deviation of the load, stroke, and web strain data collected from Specimen HG-2 prior to first cracking	93
Table 4.11 Mean and standard deviation of the flange strain data collected from Specimen HG-3 prior to first cracking	94
Table 4.12 Mean and standard deviation of the load, stroke, and web strain data collected from Specimen HG-3 prior to first cracking	94
Table 4.13 Mean and standard deviation of the flange strain data collected from Specimen HG-4 prior to first cracking	94
Table 4.14 Mean and standard deviation of the load, stroke, and web strain data collected from Specimen HG-4 prior to first cracking	94

Table 5.1 Constants used for calculation of the design and mean fatigue lives of a detail.....	155
Table 5.2 Experimental Results	156
Table 5.3 Comparison of strains with and without straps attached	157
Table 5.4 Strains on the secondary beam after a 50 kip static loading of Specimen HG-1	157
Table 5.5 Summary of drilled hole retrofit calculated values	157
Table 5.6 Cyclic life results for drilled hole retrofits	158
Appendix Table.....	Page
Table A.1 ASTM Requirements for Mechanical Properties of A36 Steel (used in all welded attachments).....	181
Table A.2 Chemical Composition of Gr. 50 Steel (Barth and Bowman, 2002)	181
Table A.3 Mechanical Properties of Steel (Barth and Bowman, 2002).....	182
Table A.4 AWS Requirements for Mechanical Properties of E7018 Weld Electrodes..	183
Table C.1 Observations from the testing of specimen VS-1.....	194
Table C.2 Observations from the testing of specimen VS-2.....	195
Table C.3 Observations from the testing of specimen VS-3.....	196
Table C.4 Observations from the testing of specimen HS-1 (up to 1 million cycles)	197
Table C.5 Observations from the testing of specimen HS-1 (1 million cycles onward)	198
Table C.6 Observations from the testing of specimen HS-2.....	199
Table C.7 Observations from the testing of specimen HG-1	200
Table C.8 Observations from the testing of specimen HG-2	201
Table C.9 Observations from the testing of specimen HG-3	202
Table C.10 Observations from the testing of specimen HG-4	203

LIST OF FIGURES

Figure	Page
Figure 2.1 Elevation view of crack after brittle fracture and cross sectional view of Quinnipiac River Bridge girder (Fisher et. al., 1980).....	22
Figure 2.2 Detailed cross sectional view of Quinnipiac River Bridge girder fracture (Fisher et. al., 1980).....	23
Figure 2.3 Photograph taken of the fractured stiffener and fascia girder of the I-95 Bridge over the Brandywine River (Chajes et. al., 2005)	23
Figure 2.4 Elevation view of crack after brittle fracture and cross sectional view of Lafayette Street Bridge girder (Fisher et. al., 1977)	24
Figure 2.5 Detailed cross sectional view of Lafayette Street Bridge girder cracking (Fisher et. al., 1977)	25
Figure 2.6 Elevation view of crack after brittle fracture and cross sectional view of the Third Street Viaduct girder (Demers and Fisher, 1990)	26
Figure 2.7 Plan and section views of the Third Street Viaduct gusset detail (Demers and Fisher, 1990).....	26
Figure 2.8 Detail of web crack passing through the web gap between the vertical stiffener and the horizontal attachment plate and a second crack through the horizontal attachment plate (Bowman, 2002)	27
Figure 2.9 Plan view of the details causing fracture in the fascia and interior girders of the Hoan Bridge (Fisher et. al., 2001)	28
Figure 2.10 Elevation and profile views of specimens tested in NCHRP Report 147 (Fisher et. al., 1974)	29
Figure 2.11 Schematic of crack growth of Type 3 details tested in NCHRP Report 147 (Fisher et. al., 1974)	30

Figure 2.12 Details tested in fatigue by Gurney (1960).....	31
Figure 2.13 Specimens tested by ORE cited in NCHRP 286 (Keating and Fisher, 1986)	31
Figure 2.14 Specimens tested in fatigue at Kansai University (Sakano et. al., 1992)	32
Figure 2.15 Crack growth starting at the weld intersect in Type 4 specimens tested at Kansai University (Sakano et. al., 1992)	32
Figure 2.16 Section of specimen 4 modeled in FEM by Sakano et. al. (1993)	33
Figure 2.17 Photo of a stiffener with intersecting welds intended to prevent corrosion (Spadea and Frank, 2002)	34
Figure 2.18 Transverse stiffener details tested by Ruge and Woesle (1962).....	34
Figure 2.19 Transverse stiffener details tested by Glinka and Krzyzeczki (1981).....	35
Figure 2.20 Lateral gusset details tested in NCHRP Report 227 (Fisher et. al., 1980)	36
Figure 2.21 Web attachment detail simulating a flange framing into a beam web (Fisher et. al., 1980).....	37
Figure 2.22 Web attachment geometries tested in FHWA-RD-79-138 (Keating, 1986) .	37
Figure 2.23 Small-scale longitudinal attachment specimens, English data (Keating, 1986)	38
Figure 2.24 Simulated longitudinal attachment specimens (left), ICOM data (Keating, 1986)	38
Figure 2.25 Large-scale web attachment details, ICOM data (Keating, 1986).....	39
Figure 2.26 Non-penetrating web attachments (right), Canadian data (Keating, 1986) ...	39
Figure 2.27 Test setup with lateral bracing used in NCHRP Report 227 (Fisher et. al., 1980)	40
Figure 2.28 Fatigue crack in girder web from web gusset (Fisher et. al., 1980)	41
Figure 2.29 Plan and Elevation of details tested at the University of Alberta (Comeau and Kulak, 1979)	42
Figure 2.30 Gusset details tested in NCHRP Report 336 (Fisher et. al., 1990).....	43
Figure 2.31 Test setup for NCHRP Report 336 with driving rod (Fisher et. al., 1990)....	44
Figure 2.32 Finite Element Model of transverse stiffeners and lateral gussets welded to the web (Mahmoud et. al., 2005).....	44

Figure 2.33 Schematic of Category C' details (AASHTO, 2007)	45
Figure 2.34 Schematic of Category E details (AASHTO, 2007)	45
Figure 3.1 VS Specimen section views.....	54
Figure 3.2 Steel straps laterally bracing the compression flange of VS and HS specimens	55
Figure 3.3 Sketch of VS and HS specimen test setup (elevation view)	56
Figure 3.4 Photo of VS and HS specimen test setup (elevation view)	57
Figure 3.5 Sketch of VS and HS specimen test setup (profile view)	58
Figure 3.6 Photo of VS and HS specimen test setup (profile view)	59
Figure 3.7 HS specimen details.....	59
Figure 3.8 HG specimen details	60
Figure 3.9 Horizontal gusset dimensions	60
Figure 3.10 Steel strap laterally bracing the spreader beam for HG specimens	61
Figure 3.11 Bolted cross bracing for HG specimens	61
Figure 3.12 Sketch of HG specimen test setup (plan view)	62
Figure 3.13 Strap running between HG specimen and secondary beam.....	63
Figure 3.14 Sketch of HG specimen test setup (elevation view)	64
Figure 3.15 Photo of HG specimen test setup (elevation view).....	65
Figure 3.16 Sketch of HG specimen test setup (profile view)	66
Figure 3.17 Photo of HG specimen test setup (profile view).....	67
Figure 3.18 Tack welds holding an attachment in place for welding	68
Figure 3.19 Downhand horizontal placement of a stiffener-to-web	69
Figure 3.20 Removing the slag at the end of a weld with a slag hammer	70
Figure 3.21 Weld passes with the slag removed at the end	71
Figure 3.22 Downhand vertical placement of a stiffener-to-flange weld	72
Figure 4.1 Elevation view of specimen types	95
Figure 4.2 Location of details on beam specimens (profile view)	96
Figure 4.3 Cracking along the weld toe at Location D of Specimen VS-1 (highlighted). ..	97
Figure 4.4 Cracking at Location D of Specimen VS-1 propagated through the flange.....	98

Figure 4.5 Bolted splice of Location D of Specimen VS-1	99
Figure 4.6 Cracking along the weld toe at Location B of Specimen VS-1	100
Figure 4.7 Cracking at Location B of Specimen VS-1 after propagating through the flange.....	101
Figure 4.8 Placement of strain gages for Specimen VS-1	102
Figure 4.9 Local warping of the cross-section of VS specimens.....	103
Figure 4.10 Retrofit to alleviate warping of Specimens VS-2 and VS-3.....	104
Figure 4.11 Retrofit to alleviate warping of Specimens VS-2 and VS-3.....	105
Figure 4.12 Cracking at Location B of Specimen VS-2 (highlighted)	106
Figure 4.13 Cracking at Location D of Specimen VS-2 (highlighted)	107
Figure 4.14 Cracking at Location C of Specimen VS-2 (highlighted)	108
Figure 4.15 Cracking along the weld toe at Location C of Specimen VS-3 (highlighted)	109
Figure 4.16 Cracking at Location C of Specimen VS-3 propagated through the tension flange	110
Figure 4.17 Placement of strain gages for Specimen VS-3	111
Figure 4.18 Web gap at Location C of Specimen HS-1.....	112
Figure 4.19 Web gap at Location D of Specimen HS-1	113
Figure 4.20 Web gap at Location A of Specimen HS-1	114
Figure 4.21 Web gap at Location B of Specimen HS-1.....	115
Figure 4.22 Magnetic drill used to perform retrofits on VS and HS specimens.....	116
Figure 4.23 Location C of Specimen HS-1 after retrofit	117
Figure 4.24 Location B of Specimen HS-1 after retrofit	118
Figure 4.25 Placement of strain gages for Specimen HS-1	119
Figure 4.26 Placement of strain gages for Specimen HS-2	120
Figure 4.27 Web gap at Location B of Specimen HG-1	120
Figure 4.28 Horizontal weld termination at Location A of Specimen HG-1.....	121
Figure 4.29 Placement of flange strain gages on HG specimens.....	122
Figure 4.30 Placement of web strain gages for HG specimens	122
Figure 4.31 Web gap at Location B of Specimen HG-2.....	123

Figure 4.32 Weld turnaround at Location D of Specimen HG-2	124
Figure 4.33 Location D of Specimen HG-2 after being retrofitted with a bolted web splice.....	125
Figure 4.34 Magnetic drill used to drill retrofits on HG specimens	126
Figure 4.35 Location A of Specimen HG-2 after being retrofitted with a bolted web splice and two 0.8125 diameter retrofit holes	127
Figure 4.36 Web gap at Location C of Specimen HG-3	128
Figure 4.37 Bolts on the south end of the gusset of Specimen HG-3 working free	129
Figure 4.38 Retrofit holes drilled at Location A of Specimen HG-3 (seen from opposite side of web).....	130
Figure 4.39 Retrofit holes drilled at Location D of Specimen HG-3 (seen from opposite side of web).....	131
Figure 4.40 Reinforcement of retrofit of Location A of Specimen HG-3 with a large plate clamped to the tension flange	132
Figure 4.41 Web gap at Location B of Specimen HG-4	133
Figure 4.42 Strap attached to the north end of Specimen HG-4 to prevent lateral movement.....	133
Figure 4.43 Strap attached to the south end of Specimen HG-4 to prevent lateral movement.....	134
Figure 4.44 Horizontal weld termination at Location D of Specimen HG-4.....	134
Figure 4.45 Retrofit holes at Location D of Specimen HG-4	135
Figure 4.46 Horizontal weld termination at Location A of Specimen HG-4.....	136
Figure 5.1 VS series test results	160
Figure 5.2 VS series test results (NG details only)	161
Figure 5.3 HS series test results	162
Figure 5.4 Test results for HG series	163
Figure 5.5 Illustration of stress flow in HG specimens prior to cracking	164
Figure 5.6 Stress flow in the gusset after cracking at the edges	164
Figure 5.7 Web strains over time for specimen HG-2	165
Figure 5.8 Plan view of additional stiffeners on specimen HG-3	166

Figure 5.9 Web strains over time for specimen HG-3	167
Figure 5.10 Web strains over time for specimen HG-4	168
Figure 5.11 Condition for a center-cracked plate subjected to a uniform tension (Rooke, 1993)	169
Figure 5.12 Loading of a retrofitted HG detail	169
Figure 5.13 Condition for a center-cracked plate subjected to pure bending (Rooke, 1993).....	170
Figure 5.14 Condition for a center-cracked plate subjected to a linearly varying stress (pure bending)	170
Figure 5.15 Increase in fatigue life of details by drilled hole retrofits.....	171
Figure 5.16 $\Delta K/\sqrt{p}$ vs. fatigue life increase	172
Appendix Figure	Page
Figure B.1 Measured web gaps at Locations A and B of Specimen HS-1	185
Figure B.2 Measured web gaps at Locations C and D of Specimen HS-1	186
Figure B.3 Measured web gaps at Locations A and B of Specimen HS-2	187
Figure B.4 Measured web gaps at Locations C and D of Specimen HS-2	188
Figure B.5 Measured web gaps at Locations B and C of Specimen HG-1	189
Figure B.6 Drawing of weld overlaps at Locations B and C of Specimen HG-2	190
Figure B.7 Drawing of weld overlaps at Locations B and C of Specimen HG-3	191
Figure B.8 Drawing of weld overlaps at Locations B and C of Specimen HG-4	192
Figure C.1 Location of details on beam specimens (profile view)	204

CHAPTER 1. INTRODUCTION

1.1. Problem Statement

Steel bridges commonly require welded attachment plates as stiffeners or connection plates that must be designed both for strength and fatigue performance according to the AASHTO LRFD Bridge Design Specification (2007). Some of these details may have a geometry where two or even three welds intersect at a point. Though it is now known that intersecting welds should be avoided in detailing, many existing bridges in Indiana have intersecting weld details.

The welding process by its very nature often produces large residual stresses in the base metal. Where welds intersect, it is likely that there is residual stress in two or even three directions, a condition known as stress triaxiality. In addition, the very geometry may restrain the steel from being able to yield at all before experiencing brittle fracture. There is presently little experimental data on the fatigue strength of these fracture-critical details.

Moreover, concerns have been expressed recently about the fatigue and fracture susceptibility of steel bridges that contain welded details that “intersect” or “nearly intersect”. Although it has long been known that intersecting welds may pose some danger, the fracture susceptibility of this type of structural detail was dramatically demonstrated in December 2000 by the brittle fracture of three large steel plate girders in the Hoan Bridge in Milwaukee, Wisconsin (Fisher et al., 2001). The brittle fracture initiated at the intersection of vertical and horizontal welded connection plates. The fatigue cracks were very, very small (if present at all) prior to the initiation of the brittle fracture, which caused two of the three plate girder members to fracture through-out their

entire depth, while the third (and last) plate girder developed a three-foot long crack emanating from the same intersection weld detail. The detail exhibited very little fatigue crack growth prior to fracture due to a number of factors: the restraint and deleterious residual stresses caused by the intersecting welds, the constraint afforded by the thickness of the members merging together, stresses generated by the traffic, the fracture toughness of the steel, and low temperatures experienced at the site. In an effort to assess the severity of the Hoan type detail, the FHWA requested that all states inspect their inventory of bridges to determine the number of bridges in service with details similar to the problematic Hoan Bridge detail.

A brittle fracture occurred at a similar type of detail in southern Indiana in 1994. The fracture occurred on an I-64 bridge over the Blue River in Harrison County. As in the Hoan Bridge, the Blue River I-64 Bridge experienced a brittle fracture that emanated from the intersection region of a horizontal gusset plate and a vertical connection plate, as shown in Figure 2.8. This fracture, which was studied by Bowman (2002), was the result of numerous factors including a large lack of fusion weld flaw in the gusset plate weld, constraint from the intersecting weld details, and extremely low temperatures during January 1994. However, it should be noted that this fracture was notably different than the Hoan Bridge fracture. The main members as well as the bracing members were not nearly as large, and the distortion-induced forces were undoubtedly less. Also, significant rust on the fracture surface indicated that fatigue cracks were likely present prior to the fracture. A similarity with the Hoan Bridge, however, is the influence of intersecting welds near the point of brittle fracture initiation.

Intersecting welds are not restricted to “Hoan type” bridge details. While these weld details are certainly important, and their behavior and repair should be fully understood, they are not the only details that may cause fatigue and/or brittle fracture problems. Any detail that has welds that intersect or nearly intersect will restrain the steel in the vicinity of the weld intersection and will introduce additional welding residual stresses. A question then arises as to the severity of the detail, even though it does not fit the exact

nature of the Hoan type detail. It is possible that an intersecting weld detail with less constraint than the Hoan bridge detail may lead to fatigue crack initiation earlier than otherwise anticipated, but not trigger a brittle fracture. Clearly, the geometry of the detail is very important and will determine the fatigue and fracture susceptibility.

In general, there are four types of details in Indiana that contain intersecting weld details (Dittrich, 2004). These include the following detail types:

- Hoan-type details where a horizontal gusset plate for lateral bracing is welded to the web and intersects a vertical connection plate that is also welded to the web and used to attach a cross frame.
- Details where the weld of a vertical connection plate for a floor beam or diaphragm intersects the longitudinal flange-to-web weld. Another variation of this detail includes the flange weld for a transverse intermediate stiffener that intersects the longitudinal flange-to-web weld.
- Longitudinal stiffeners that intersect a vertical stiffener or vertical connection plate.
- Lateral bracing attachment plates that are directly welded to the top of the bottom flange of a girder and nearly intersect with the longitudinal flange-to-web welds.

The breakdown on the number of bridges that contain the above four details are 70 bridges for the Hoan type detail, numerous bridges for the stiffener weld to longitudinal weld, 27 bridges with longitudinal web stiffeners (that may or may not intersect a vertical stiffener), and 9 bridges that contain a welded attachment plate on the bottom flange.

The fatigue strength of an existing detail or connection in a beam or girder can be assessed using the fatigue strength provisions in the Guide Specifications for Fatigue Assessment (AASHTO,1990) or the Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges (AASHTO, 2003). Both of these approaches rely upon the use of standard categories to evaluate the severity of a particular structural detail. These range from Category A for plain material such as rolled beams to

Category E' for the terminal end of a welded partial length cover plate detail. The category ranking assessment of bridge details, however, evaluates details based on empirically collected fatigue test data for particular structural details. In general, the fatigue category ranking does not include a wealth of data related to details with intersecting welds. Consequently, additional data on common details with intersecting welds that may not be as severe as the Hoan-type detail has not been thoroughly studied. Additional data on these types of details would be extremely useful since it would provide information needed to properly assess (categorize) the severity of existing details. This information could then be used to assist in making decisions regarding useful remaining life and repair/retrofit options.

1.2. Objectives and Scope

The purpose of this study is to evaluate the fatigue behavior of three different details in Indiana bridges that contain intersecting welds: a vertical stiffener welded to the web and flange of a girder; a welded horizontal stiffener terminating near a welded vertical stiffener; and a welded horizontal gusset plate coped to fit around a welded vertical stiffener. Due to limited usage, it was decided that tests would not be conducted on details with a lateral bracing attachment plate that is welded to the top of the bottom flange and which intersects the longitudinal flange-to-web weld.

Nine beams with welded details were tested under constant amplitude cyclic loading. Details with both intersecting and nearly-intersecting welds were tested and their fatigue behavior examined to determine if the size of the gap between perpendicular welds has an effect on fatigue resistance.

Chapter 2 includes a literature review of the fatigue behavior of the three details listed above, including case studies of service cracking and experimental testing. Chapter 3 describes the experimental testing program utilized in this study, while the experimental results are presented in Chapter 4. A detailed discussion of the experimental results is

provided in Chapter 5. Lastly, Chapter 6 provides the conclusions, recommendations and implantation plans for the study.

CHAPTER 2. LITERATURE REVIEW

The literature survey presented in this chapter discusses the behavior of details found in steel bridges that contain two or more intersecting or nearly intersecting welds. Comparisons will be drawn between the fatigue performance of details with intersecting welds and those without. Included are case studies of bridges in service, analytical and experimental testing of both large-scale and small-scale specimens, and a review of the current code specifications for both design and maintenance of highway bridges.

2.1. Case Studies

2.1.1. Horizontal Stiffeners Welded to the Web

Horizontal stiffeners are used in plate girders to increase web stiffness against compression forces. Though only useful in compressive areas, usually near concentrated loads, they may be found placed along the entire girder length or in areas of tension to provide symmetry and a pleasing appearance. These welded longitudinal plates can lead to intersecting weld conditions when they are interrupted by welded vertical stiffeners or when two plate sections are joined by a welded butt splice. These points may be critical as a possible source of fracture of the girder if not properly detailed and constructed.

In the case of the Quinnipiac River Bridge (Figure 2.1) the critical crack originated from a poorly fused butt weld connecting two horizontal stiffeners in the lower half of the web (Fisher et. al., 1980). It is likely that this crack initiated and began propagating during transport and erection as well as during service. Brittle fracture occurred once the crack had completely severed the butt weld connecting the stiffener plates and had grown

across the web thickness. This sudden fracture led to cracking partway up the web plate and downward into the tension flange, where it was arrested in the zone of compressive residual stress that resulted from the flange-to-web weld. The crack then continued to grow in fatigue, propagating through the entire thickness of the tension flange before being discovered (Figure 2.2). This girder fracture may have been prevented if a full penetration weld had been used for the longitudinal stiffener plate rather than a partial penetration weld. Another option would have been to avoid the use of a stiffener in the tension zone of the girder, since it acts like a flange.

A similar case occurred in the bridge carrying I-95 over the Brandywine River in Delaware (Chajes et. al., 2005), but in this case the brittle fracture severed the entire bottom tension flange and all but the top 12 in. of the 94 in. deep web plate before arresting (Figure 2.3). Testing of specimens from the failed girder found that the flange had a lower fracture toughness than the web, therefore the fracture did not arrest in the flange as was the case in the Quinnipiac River Bridge. At the time of the fracture, the fascia girder was experiencing the highest live load fluctuations of its service life because traffic had been shifted onto the shoulder directly above the girder due to reconstruction in the median of the bridge. Since the crack originated from lack of fusion in a full joint penetration weld splicing the longitudinal stiffener, part of the retrofit involved drilling a 3 in. diameter core through the back side of the web at the splice. This effectively isolated the splice joint area from the girder web and prevented any cracks that developed there from leading to large-scale damage.

2.1.2. Horizontal Gusset and Transverse Stiffener Welded to the Web

Lateral bracing is used in multi-girder steel bridges as a way to distribute lateral forces among the girders. This bracing usually consists of built-up crossframes of steel angles or WT-shapes spanning adjacent girders in K or X formations. These members are bolted or welded to lateral gusset plates that are welded to the girders. To be most effective, this bracing is placed as close to the bottom of the girder as possible. To this end, lateral gusset plates may be fillet or groove welded to the girder web just above the

flange, welded directly to the top of the bottom flange, or even welded to the underside of the bottom flange.

Welded vertical stiffeners are used in steel bridges to stiffen the web against buckling due to shear or local bearing. They can also double as connection plates for floorbeams or transverse diaphragms. Floorbeams act to distribute load from the deck to the girders while transverse diaphragms help distribute loads between adjacent girders and provide lateral stability during construction. Vertical stiffener plates are normally fillet welded to the girder web and may be full-depth or cut short of either flange. If a positive connection to the flange is desired to prevent distortion-induced cracking, it can be achieved either by introducing a bolted or welded connection angle or welding directly to the flange.

Horizontal gussets must be welded to an area of the web that is stiffened to prevent large displacements from the lateral force transfer. Therefore, the detail of a horizontal gusset intersected by a vertical stiffener is one commonly found in welded steel bridges. This geometry can be detailed in several ways. Most commonly, a single horizontal gusset plate is connected to the web and coped to fit around the vertical stiffener. Sometimes twin gusset plates are used on either side of the stiffener instead of a single coped gusset. A positive attachment can be made between the vertical stiffener and horizontal gusset plates to limit distortion stresses in the web. This can be done either through direct welding or through a bolted or welded angle. In certain cases, this detail can contain two or three perpendicular welds that either intersect or nearly intersect. Several instances of in-service fracture due to this condition have been documented.

One instance occurred on the Lafayette Street Bridge over the Mississippi River in St. Paul, Minnesota (Fisher et. al., 1977). In May 1975, a large crack was found in a fascia girder near the center of the main span of the bridge. At this location, there was a lack of fusion area where the transverse groove weld connecting a horizontal gusset to a vertical stiffener did not fully penetrate to the back-up bar. Since this initial flaw was

perpendicular to the primary bending stress, a crack soon initiated and propagated across the gusset plate. The crack was able to grow into the girder web because the horizontal gusset-to-stiffener groove weld directly intersected a vertical stiffener-to-web fillet weld. Once the crack grew in fatigue across the thickness of the web a brittle fracture occurred, severing the entire bottom flange and much of the lower web. After this brittle fracture, cracking propagated upward into the girder web and continued outward across the gusset plate. As the crack grew, the remaining gusset plate yielded and could no longer slow the crack growth in the web. Eventually, a second brittle fracture occurred and severed the web to a point only 7.5 in. short of the top flange (Figures 2.4 and 2.5).

A similar event occurred a few years later on the Third Street Viaduct spanning Conrail tracks in Columbus, Ohio (Demers and Fisher, 1990). In March 1978, a large crack was found which had severed the bottom flange and lower web of a fascia girder (Figure 2.6). Within a few days, the crack had propagated to a point just a few inches below the top flange. The crack initiated from the intersection of a longitudinal gusset-to-web groove weld and a transverse stiffener-to-web fillet weld (Figure 2.7). There was a cold lap, or lack of fusion, at this interface that acted as an initial flaw. This cold lap was likely caused as a transverse gusset bolted to the vertical stiffener buckled outward during the shrinkage of the groove weld connecting the horizontal gusset to the web. Once fatigue cracking began at this initial flaw, it quickly propagated across the web and reached a critical size, triggering a brittle fracture through much of the girder.

A fracture originating from a positive stiffener-to-gusset connection occurred in 1993 on the bridge carrying I-64 over the Blue River in Harrison County, Indiana (Bowman, 2002). In this case, the gusset plate was chamfered to keep three perpendicular fillet welds – the stiffener-to-web weld, the gusset-to-web weld, and the gusset-to-stiffener weld – from intersecting (Figure 2.8). However, the tight cope of the gusset to fit around the stiffener did not allow proper fit up during erection so it was widened with a torch cut. As a result, there was insufficient fusion of the gusset-to-stiffener welds. Just as in the case of the Lafayette Street Bridge, this initial crack-like flaw led to a fatigue crack

which propagated transversely across the gusset. However, in this case there were no intersecting welds to give the crack a direct path into the web. Once the gusset fractured, the crack was isolated but the girder web in the area experienced additional out-of-plane stress from the force in the lateral bracing. A fatigue crack most likely initiated from the stress concentration at the inside termination of the gusset-to-web weld. This area was very susceptible to fracture for several reasons: in addition to the aforementioned stress concentration and out-of-plane forces, there existed a high degree of constraint in the web gap. There was only a 0.25 in. space separating the weld toes so the web could not neck down in yielding as the crack grew. With very little fatigue growth, this crack reached a critical size and the girder experienced a sudden brittle fracture that severed the bottom flange and all but 1 in. of the web plate.

On December 13, 2000, a fracture originating from an intersecting weld location completely severed two of the girders of the Hoan Bridge in Milwaukee, Wisconsin and left a 3 ft. crack in the web of the third girder (Fisher et. al., 2001). The cracks in all three girders originated from the same location: the intersection of the welds connecting a coped lateral gusset plate and a floorbeam connection plate to the girder web. The cope in the gusset was sized to accommodate a 0.125 in. gap between the toe of the vertical stiffener-to-web fillet weld and the horizontal gusset-to-web groove weld. Much like the critical detail in the Blue River Bridge, this short web gap is very fracture susceptible due to the stress concentrations at the weld toes and the high degree of constraint. The first fracture was believed to occur in the middle girder, where the geometry of the detail was made even more severe by two factors. First, the gusset plate had been shifted laterally during erection, leading to a zero weld gap condition on one side of the stiffener and a 0.25 in. gap on the other. Second, a 2 in. hole had been drilled in the gusset immediately adjacent to the stiffener on the 0.25 in. weld gap side. Both of these factors raised the stress in the web and created a crack like-condition all along the gusset to stiffener interface at the zero web gap location (Figure 2.9). The geometry of this detail was such that a sudden brittle fracture could begin in the web with little or no measurable fatigue

cracking beforehand. A comprehensive fractographic analysis of the fracture surface confirmed that this was the case.

2.2. Analytical and Experimental Studies

2.2.1. Transverse Stiffener Welded to the Web and Flange

NCHRP Report 147 (Fisher et. al., 1974) discusses a series of tests performed at Lehigh University to determine the fatigue performance of several details found in welded steel bridges. The study involved testing of several large-scale specimens and examining the experimental data to determine what variables had the largest effect on fatigue life. The specimens consisted of plate-girders with several different welded attachments in various geometries. Of particular interest to this study is the case of a full-depth vertical stiffener fillet welded to the web and both flanges. All details of this type had plates with a 1 in. chamfer at both ends resulting in a gap of at least 0.25 in. between the transverse vertical stiffener welds and the longitudinal flange-to-web weld. Elevation and profile views of the specimens can be seen in Figure 2.10.

Analysis of the data showed that the main factor affecting the fatigue life of this type of detail is the stress range at the top of the bottom flange. This is the most likely site of crack initiation due to the residual stresses and potential initial discontinuities from the welding process as well as the stress concentration factor arising from the weld toe geometry. Other factors such as minimum stress, attachment of diagonal bracing, steel type, and flange and web thickness were found to have little discernible effect on fatigue behavior.

Under fatigue loading, these details all exhibited similar behavior. Several small cracks initiated at different sites along the toes of the transverse fillet weld connecting the stiffener to the tension flange (Figure 2.11). These cracks propagated downward into the flange in a semielliptical shape perpendicular to the primary bending stress. They

eventually coalesced to form one large crack that split the tension flange and part of the web until the specimen failed by yielding of the remaining cross section. None of the details tested exhibited brittle fracture. Though all of the specimens in the study were plate girders, the report cites data from earlier work by Gurney (1960) on the fatigue behavior of vertical stiffeners welded to the web and flange of rolled shapes (Figure 2.12).

In NCHRP Report 286 (Keating and Fisher, 1986) much of the full-scale fatigue test data collected since NCHRP Report 147 were gathered and compared to the AASHTO fatigue design curves. Several welded vertical stiffener tests are cited in the report. They include studies performed by the Office of Research and Experiments of the International Union of Railways (ORE) that tested plate girders with vertical stiffeners containing intersecting welds (Figure 2.13). However, within the scope of that study, there was no detrimental effect found from intersecting the welds and all of the details equaled or surpassed the Category C design life.

Large-scale tests were performed at Kansai University (Sakano et. al., 1992) to better understand the fatigue performance of several welded vertical stiffener variations. The details tested included full-depth vertical stiffeners welded to the web and flange with and without a 30 mm scallop (Figure 2.14). The latter detail leads to a condition with three perpendicular, intersecting welds. The fatigue behavior of the details was very similar to that of previous tests. Stress range at the top of the tension flange was the main factor affecting fatigue life. During testing, several small cracks occurred along the weld toe and eventually coalesced into one large crack that grew downward through the flange. It is noteworthy, however, that the details with three intersecting welds experienced crack growth that initiated at the intersection of the welds (Figure 2.15). In addition, within the tests performed, these intersecting weld details had a shorter fatigue life than those without intersecting welds.

The researchers at Kansai University utilized an analytical model to explain the cause for this difference in the fatigue life of the specimens (Sakano et. al., 1993). They developed 2D finite element models of the details with different angles and protrusions at the weld toe. They then used a crack growth model to calculate stress intensity factors and predict the fatigue life of the detail. A 3D finite element model was developed for the intersecting weld detail (Figure 2.16) to determine the effect of the extra protrusion of the weld material where the three welds intersected. This caused an even higher stress concentration, and it is postulated that this may have resulted in a shorter fatigue life for the test specimens. It should be noted that the test matrix for the experimental results was relatively small and the fatigue lives of these intersecting weld specimens was still above the Category C design life for the applied stress.

Spadea and Frank (2002) performed a study of the fatigue behavior of certain fillet-welded transverse stiffener details chosen to prevent corrosion. One way to prevent moisture from getting between a vertical stiffener and the flange is to use a detail with a small clip on the stiffener and seal this area off with weld metal. This leads to a case with three perpendicular, intersecting welds as seen in Figure 2.17. An FHWA report by Verma et. al. (2001) found that this technique is used by other countries. The study cites research by Ruge and Woesle (1962) on the fatigue performance of three different full-depth stiffener weld conditions seen in Figure 2.18: A.) a very small clip with intersecting welds, B.) a large clip with no turnaround on the inside edge and C.) a large scallop to allow a turnaround on the inside edge. The fatigue behavior of the experimental tests suggested that the intersecting weld condition had no detrimental effect on fatigue life. The intersecting weld details did not have a significantly shorter life than those with a clip and all cracking initiated at the transverse stiffener-to-flange weld toe away from the longitudinal flange-to-web weld.

Full-scale fatigue tests by Glinka and Krzyzecki (1981) produced similar results. In testing plate girders with the transverse stiffener details seen in Figure 2.19, they concluded that there was no significant difference between the fatigue performance of

stiffeners with a cope and those without. In the entire test series, only one crack occurred at the point of intersecting welds. Most cracking occurred at the outside edge of the stiffener-to-flange weld toe.

2.2.2. Horizontal Stiffener Welded to the Web

Though no tests were found with nearly intersecting horizontal and vertical stiffeners welded to the web, tests on horizontal stiffeners alone will be used as a basis of comparison.

Some of the welded attachments tested in NCHRP Report 227 (Fisher et. al., 1980) have a similar geometry to the typical horizontal web stiffeners found in Indiana bridges. The type 3 web gussets seen in Figure 2.20 have a similar thickness but lower aspect ratio (longitudinal divided by transverse length) than a typical horizontal stiffener. They also have bracing members attached, which leads to out-of-plane effects not experienced by horizontal stiffeners. The study also tests some details that simulate a beam flange framing into the web (Figure 2.21). These have an aspect ratio close to that commonly found in horizontal stiffeners but – at 1 in. thick – they are much thicker than horizontal stiffeners, which usually have a thickness on the order of 0.25 in. The result of these larger thicknesses and smaller aspect ratios is that the attachment almost acts as a secondary flange and takes a lot more stress than a thinner, shorter horizontal stiffener would. This results in a higher stress concentration at the toe of the connecting welds and, as a result, both of these details have a lower fatigue resistance than the typical stand-alone horizontal stiffener.

Several of the tests cited in the NCHRP Report 286 (Keating and Fisher, 1986) collection of previous research have web attachments of interest to this test. The curved girders tested in FHWA-RD-79-138 included different web attachment details (Figure 2.22) to see the effects of different terminations on fatigue life. The English fatigue data included longitudinal attachment plates (Figure 2.23) but, since they were small-scale tests, it is likely that they overestimated fatigue performance. Small-scale specimens usually do not

have the same degree of constraint found in large-scale specimens and as a result have lower residual stresses and therefore a longer fatigue life. The Swiss Federal Institute of Technology (ICOM) tested both small-scale (Figure 2.24) and large-scale (Figure 2.25) web attachment specimens. The full-scale ICOM specimens had a geometry that was closer to that of horizontal stiffeners found in Indiana bridges. The web attachments seen in the Canadian fatigue data (Figure 2.26) have a much lower aspect ratio than this geometry. All of these test data performed at or above the AASHTO Category E classification.

2.2.3. Horizontal Gusset and Vertical Stiffener Welded to the Web

For NCHRP Report 227, Fisher et. al. (1980) performed large-scale laboratory testing on girders with vertical stiffeners intersecting horizontal gussets to rate their fatigue performance. Gusset plates with and without a positive connection to the vertical stiffener were tested (Figure 2.20). In all cases, the gusset plate was attached to the girder web with top and bottom longitudinal fillet welds. Also, all details were fabricated with a significant gap, at least 2", between the gusset-to-web and stiffener-to-web fillet weld toes. In an attempt to simulate out-of-plane forces seen in highway bridge girder webs, WT bracing was bolted to the gusset and attached to a secondary beam made much stiffer than the test specimen (Figure 2.27).

Much like the previous full-scale test results, fatigue life was mainly dependent on the nominal bending stress range at the level of the detail. All of the critical cracks leading to failure initiated at the toe of the wraparound of the fillet weld connecting the gusset plate to the web (Figure 2.28). These cracks propagated across the web in a semielliptical shape perpendicular to the primary bending stress. No cracking occurred in the web at the fillet weld toe adjacent to the vertical stiffener. A finite element model of the gusset showed that the stress concentration at the end of the gusset plate is higher than that in the web gap. Though there is an initial crack-like condition at the gusset-to-stiffener attachment, none of cracking at this location reached the surface by the time the specimen failed.

Experimental testing was also performed by Comeau and Kulak (1979) at the University of Alberta to examine the fatigue behavior of lateral bracing attachment details. More specifically, how much of an effect, if any, the presence and relative stiffness of lateral bracing has on fatigue life.

The testing involved seven W16x36 hot-rolled specimens with various welded attachments loaded in four-point bending on a 10 ft. simply supported span. Each beam had a welded vertical stiffener intersecting a welded lateral gusset plate staggered on both sides of the web. The horizontal gussets were connected to the vertical stiffeners and web by 0.25 in. fillet welds. Each gusset fit tight around the vertical stiffener had 1 in. chamfers cut out to prevent the intersection of the gusset-to-stiffener and gusset-to-web welds. Plan and elevation views of the details can be seen in Figure 2.29. For each beam, lateral bracing was attached only to the gusset on one side of the web. The bracing consisted of two angles oriented at 45° bolted to the gusset at one end and clamped to a fixed pedestal at the opposite end.

Strain gages were placed on the gusset and static load tests, performed with and without lateral bracing attached, showed that the bracing introduced significant stresses into the gusset. However, this seemed to have no discernable effect on the fatigue life of the specimens. The main variable affecting fatigue was still the nominal stress range in the web at the level of the detail.

In all cases, fatigue cracking occurred at the outside edges of the gusset plate. Cracks began at the weld toe and grew through the web in a semi-elliptical shape perpendicular to the primary bending stress. No cracking occurred in any other location during the tests. The failure criterion was the appearance of a discernable length of cracking on the opposite side of the web. Once this occurred, the crack was reinforced with weld material and a doubler plate was clamped to the flange below the crack so the test could continue until cracking occurred at other locations.

A field study of two bridges near Edmonton found that the rotation of the specimens caused by the lateral bracing was at the very least comparable to that found in the field. The relative stiffness of the bracing in the experiment was also found to be comparable to that found in actual bridges.

In 1990, Fisher et. al. published NCHRP Report 336, a study done at Lehigh on fatigue cracking caused by secondary distortion stresses in steel bridges. A major aim of this study was to compare the fatigue performance of three different gusset-stiffener details: one where the gusset is welded directly to the stiffener, one where the gusset is attached to the stiffener with a bolted angle, and one with no positive attachment between the gusset and stiffener (Figure 2.30). All specimens were fabricated with a gap of at least 1 in. between any perpendicular welds. All of the details were subjected to a nominal in-plane bending stress of 6 ksi. The out of plane stress, introduced by a driving rod bolted to the vertical stiffener (Figure 2.31), was varied. The two levels of out-of-plane stress were “low” (achieving a secondary stress of 10 to 15 ksi in the web gap) and “high” (achieving a secondary stress of 18 to 31.5 ksi in the web gap). These stress levels were achieved by varying the length and angle of the driving rod until the strain gages in the web gap output the desired values.

The details subjected to “low” secondary stresses did not experience any visible fatigue crack growth even though they were subjected to over 10 million loading cycles with a primary bending stress range of more than double the design endurance limit (2.9 ksi for a Category E detail). The details subjected to “high” secondary stresses all experienced crack growth at the toe of the vertical stiffener-to-web weld. If the primary bending and secondary distortion stresses were added and the resulting stress range compared to existing fatigue data, the details had an expected life comparable to AASHTO Category C. Details without a positive attachment between the stiffener and gusset experienced shorter fatigue lives and cracks that reinitiated in fatigue retrofit holes. Cracking near

details with a positive attachment occurred more slowly and could be arrested with stop holes.

Mahmoud et. al. (2005) used finite element modeling to investigate the web stresses that occur in highly constrained details in steel bridges. One of the most common conditions that leads to a high degree of constraint is the intersection of two or more perpendicular welds. The paper looked at models of three different geometries with increasing levels of constraint: 1.) a plate girder web with a transverse fillet weld, 2.) case 1 with a transverse stiffener added, and 3.) case 2 with a longitudinal welded plate added, welded to the web but not the vertical stiffener (see Figure 2.32). In each case, a uniform tensile stress of 40 ksi was applied along the cross section of the web. Elastic and inelastic conditions were explored for each case and the resulting stresses in the web gap were reported. For each case, a triaxiality factor was calculated. This factor is a function of the three principal stresses output by the model and is related to the amount of yielding that would occur in the detail before the onset of brittle fracture. This information is valuable for a discussion of fracture in constrained joints but the most important information pertaining to this study is the principal stress that the model predicts. Fatigue crack growth is directly related to the maximum applied principal stress. If a higher principal stress is reported in a small web gap as opposed to a large one, then this can have a significant effect on fatigue life of the detail. The stress concentration factor predicted by the model for the case of a welded transverse stiffener is 1.125. The addition of a longitudinal gusset with a small web gap results in a stress concentration factor of 1.82.

Connor et al. (2007) conducted additional studies to examine the influence of triaxiality on constraint-induced brittle fracture. Triaxiality factors were used to assess the likelihood that a structure would fracture in a brittle manner rather than yield in a ductile manner. Based upon the case studies examined in the paper and the finite element work cited in a related study by Mahmoud et al. (2005), the authors indicate that,

“In the presence of a 6.4 mm web gap, the level of constraint is reduced to a level such that brittle fracture should not occur. It is critical that the web gap length be

taken as the distance between the weld toes of the connected components and not the distance between the plates (i.e., 6.4 mm toe to toe).”

Suggestions for the retrofit of lateral gusset plate details and longitudinal stiffener details are also described. They recommend the use of 2-in diameter drilled holes in the web gap area with the transition to the web ground smooth to provide smooth surface finish.

2.3. Specification Requirements

The current AASHTO LRFD Bridge Design Specification (4th ed., 2007) divides steel bridge details into categories based on their fatigue performance in order to simplify the design of a bridge that will be subjected to a predicted amount of truck traffic over its lifetime.

The nominal fatigue resistance $(\Delta F)_n$ of a detail can be calculated using the equation:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{1/3} \geq \frac{1}{2}(\Delta F)_{TH}$$

Where N is the expected number of stress cycles from truck passage over the lifetime of the detail, A is the detail category constant, and $(\Delta F)_{TH}$ is the threshold stress range. The primary stress range at the detail generated by the passage of a fatigue truck shall be kept below this nominal fatigue resistance limit.

The base metal at the toe of transverse stiffener-to-web or stiffener-to-flange welds (Figure 2.33) is classified as a Category C' detail for design. The constant-amplitude fatigue threshold $(\Delta F)_{TH}$ for this detail is 12.0 ksi and the detail category constant (A) is $44.0 \times 10^8 \text{ ksi}^3$.

Both horizontal stiffeners and lateral gussets are grouped as longitudinally loaded fillet-welded attachments (Figure 2.34). For the vast majority of these cases (length greater than 4.0 in. and thickness less than 1.0 in.) the base metal at the toe of the welds is

classified as Category E. The constant-amplitude fatigue threshold for these details is 4.5 ksi and the detail category constant is $11.0 \times 10^8 \text{ ksi}^3$.

The design specification also has provisions (in Section 6.6.1.3.1) to safeguard against large secondary stresses caused by out-of-plane distortion. Vertical connection plates should be welded to both girder flanges as well as the web to prevent horizontal cracking in the unstiffened web area. Horizontal gussets should always be placed at a stiffened area of the web at a distance at least one half of the flange width above the flange and they should always be centered on the transverse stiffener. The ends of the lateral bracing members attached to the gusset should also be kept at least 4.0 in. from both the web and vertical stiffener. All of these requirements aim to prevent large stresses that can occur if lateral deformations must be accommodated in short, unstiffened lengths of the relatively thin web in a plate girder.

The current AASHTO Guide Manual for Condition, Evaluation, and Load and Resistance Factor Rating of Highway Bridges (1st ed. and 2005 Interim) uses the same classification system as the design specifications to calculate the remaining useful life of bridges in service. To find this life, first the effective stress range $(\Delta f)_{eff}$ is found by multiplying the measured effective stress range by R_s , the partial load factor. This partial load factor is 1.0 for the case of a simple analysis using the LRFD Fatigue Truck but can be lowered with a more refined analysis and/or more accurate stress range data from weigh-in-motion data or bridge instrumentation.

The detail is considered to have infinite life if the maximum stress range applied (taken as $2.0 * (\Delta f)_{eff}$ in absence of more accurate data) is less than the constant amplitude fatigue limit. If this is not the case, the remaining fatigue life, Y (in years), of a bridge can be calculated using the following equation:

$$Y = \frac{R_R A}{365 n (ADTT)_{SL} (\Delta f)_{eff}^3}$$

where R_R is a resistance factor found in Table 7-2 of the manual, n is the number of stress range cycles per truck passage, and $(ADTT)_{SL}$ is the single-lane average daily truck traffic. Based on this calculated remaining life, an engineer can make decisions concerning future inspection, repair, retrofit, or replacement of a bridge.

Figure 2.1 Elevation view of crack after brittle fracture and cross sectional view of Quinnipiac River Bridge girder (Fisher et. al., 1980)

Figure 2.2 Detailed cross sectional view of Quinnipiac River Bridge girder fracture (Fisher et. al., 1980)

Figure 2.3 Photograph taken of the fractured stiffener and fascia girder of the I-95 Bridge over the Brandywine River (Chajes et. al., 2005)

Figure 2.4 Elevation view of crack after brittle fracture and cross sectional view of Lafayette Street Bridge girder (Fisher et. al., 1977)

Figure 2.5 Detailed cross sectional view of Lafayette Street Bridge girder cracking
(Fisher et. al., 1977)

Figure 2.6 Elevation view of crack after brittle fracture and cross sectional view of the Third Street Viaduct girder (Demers and Fisher, 1990)

Figure 2.7 Plan and section views of the Third Street Viaduct gusset detail (Demers and Fisher, 1990)

Figure 2.8 Detail of web crack passing through the web gap between the vertical stiffener and the horizontal attachment plate and a second crack through the horizontal attachment plate (Bowman, 2002)

Figure 2.9 Plan view of the details causing fracture in the fascia and interior girders of the Hoan Bridge (Fisher et. al., 2001)

Figure 2.10 Elevation and profile views of specimens tested in NCHRP Report 147 (Fisher et. al., 1974)

Stage 1: Part - Through Crack

Stage 2: Through Crack

Specimen SGB 312

Figure 2.11 Schematic of crack growth of Type 3 details tested in NCHRP Report 147 (Fisher et. al., 1974)

Figure 2.12 Details tested in fatigue by Gurney (1960)

TRANSVERSE STIFFENER DETAIL TYPES

Figure 2.13 Specimens tested by ORE cited in NCHRP 286 (Keating and Fisher, 1986)

Figure 2.16 Section of specimen 4 modeled in FEM by Sakano et. al. (1993)

Figure 2.17 Photo of a stiffener with intersecting welds intended to prevent corrosion (Spadea and Frank, 2002)

Figure 2.18 Transverse stiffener details tested by Ruge and Woesle (1962)

Figure 2.19 Transverse stiffener details tested by Glinka and Krzyzecki (1981)

All Dimension in Inches (1" = 25.4 mm)

Figure 2.20 Lateral gusset details tested in NCHRP Report 227 (Fisher et. al., 1980)

Figure 2.21 Web attachment detail simulating a flange framing into a beam web (Fisher et. al., 1980)

Figure 2.22 Web attachment geometries tested in FHWA-RD-79-138 (Keating, 1986)

Figure 2.23 Small-scale longitudinal attachment specimens, English data (Keating, 1986)

Figure 2.24 Simulated longitudinal attachment specimens (left), ICOM data (Keating, 1986)

Figure 2.25 Large-scale web attachment details, ICOM data (Keating, 1986)

Figure 2.26 Non-penetrating web attachments (right), Canadian data (Keating, 1986)

Figure 2.27 Test setup with lateral bracing used in NCHRP Report 227 (Fisher et. al., 1980)

Figure 2.28 Fatigue crack in girder web from web gusset (Fisher et. al., 1980)

Figure 2.29 Plan and Elevation of details tested at the University of Alberta (Comeau and Kulak, 1979)

- All Holes To Be 1" dia.
For 7/8" dia. Bolts
- All Gusset Pl. To
Be 1/2" Thick
- All Gusset To Web
Welds To Be 5/16"
- All 1/2" Pl. Welds
To Be 5/16"

- All Holes Are 1" dia.
For 7/8" dia. A325 Bolts

Figure 2.30 Gusset details tested in NCHRP Report 336 (Fisher et. al., 1990)

Figure 2.31 Test setup for NCHRP Report 336 with driving rod (Fisher et. al., 1990)

Figure 2.32 Finite Element Model of transverse stiffeners and lateral gussets welded to the web (Mahmoud et. al., 2005)

Figure 2.33 Schematic of Category C' details (AASHTO, 2007)

Figure 2.34 Schematic of Category E details (AASHTO, 2007)

CHAPTER 3. EXPERIMENTAL PROGRAM

The main objective of this experimental study was to evaluate the fatigue behavior of steel bridge details containing intersecting or nearly-intersecting welds. Nine steel beams with welded details were tested under constant amplitude cyclic loading. Three of the tests involved beams with welded vertical stiffeners. The next two tests involved beams with welded vertical and horizontal stiffeners. The remaining four tests involved beams with bracing attached to a secondary beam to simulate out-of-plane effects on horizontal gussets coped to fit around a vertical stiffener.

3.1. Design Variables

The primary variables evaluated in the study were the structural detail type, the magnitude of the applied stress range, and the degree of weld intersection. These variables are discussed further in the sections below.

3.1.1. Detail Type

The main variable in the study was the type of detail that was loaded in fatigue. The test specimens had three different welded details commonly found in steel bridges in Indiana:

- VS = vertical stiffener
- HS = vertical stiffener intersecting horizontal stiffeners
- HG = vertical stiffener intersecting a horizontal gusset

3.1.2. Stress Range

The load range for the experimental tests was selected to generate a stress range at the detail that would be low enough to be realistic but high enough to generate cracking within a reasonable experimental timeline. The capacity of the actuator also placed an upper limit on the stress range that could be generated in the specimen. The stress ranges at the details of interest ranged from 10.7 ksi to 17.7 ksi.

3.1.3. Detail Geometry

The final variable was the size of the gap, or lack thereof, between the toes of perpendicular welds at each detail. This gap was defined as one of the following:

- NG (no gap): the two perpendicular welds directly intersect
- SG (small gap): a gap smaller than 0.25 in. exists between the toes of the two perpendicular welds
- LG (large gap): a 0.25 in. or larger gap exists between the toes of the two perpendicular welds

3.1.4. Test Variables

The test matrix of the various types of details, stress ranges, and detail geometries can be seen in Table 3.1. As is noted in the table, tests are conducted for all detail types, but not all gap types. At the very least, two different web gap lengths were explored for each detail type so a comparison could be made between their fatigue behavior.

3.2. Test Arrangement

The testing involved loading the specimen to cause strong-axis bending and generate a tensile stress below the neutral axis. Two different setups were used: one for single beams with stiffeners attached and one for beams with a lateral gusset and bracing attached to a secondary beam. All beam members were W24 x 55 hot-rolled shapes conforming to ASTM A572 Gr. 50. The test beams were from three different heats of

steel. The chemical composition and mechanical properties are provided in Appendix A. The ranges of some of the mechanical properties reported are:

$$29,000 \text{ ksi} \leq E \leq 31,500 \text{ ksi}$$

$$49.0 \text{ ksi} \leq F_y \leq 59.3 \text{ ksi}$$

$$65.2 \text{ ksi} \leq F_u \leq 80.6 \text{ ksi}$$

$$21.1 \leq \% \text{ Elong.} \leq 29.7$$

All beams were 16 ft. 6 in. long and rested on roller supports at 3 in. from each end, resulting in a simply supported 16 ft. span. The VS and HS specimens were loaded in three point bending, while the HG specimens were loaded in four point bending.

3.2.1. Vertical Stiffener Specimens

The first series of tests evaluated the effect of intersecting, or nearly intersecting, transverse web and flange welds at the interior corner of a vertical stiffener. All three VS specimens had two details to be tested, each offset 1 ft. from midspan. Vertical stiffeners were made of 0.5 in. x 3 in. hot-rolled ASTM A36 plate stock cut with a band saw to 22.625 in. to bear against both flanges. The stiffeners were clipped at varying lengths depending on the detail geometry being tested (Figure 3.1).

The testing of the specimens involved loading the beam in three-point bending. The loading was transferred from the actuator to the top flange of the specimen at midspan by a large 13 in. x 5 in. x 3 in. steel bearing block. To account for tilting of the specimen flanges, this bearing block was grouted to the top flange with hydrostone while a bubble level rested on top. This assured a flat loading interface for the actuator. The specimen ends rested on a 6 in. bearing length on pin and roller supports made from 2 in. thick steel plates and 2 in. diameter steel pins. These supports were grouted with hydrostone to 2 ft. tall concrete blocks. To prevent excessive movement during testing, the concrete support blocks were shimmed and restrained with a steel cable running through the strong floor.

To provide lateral support for the test specimen, thin steel strapping was bolted to the top flange of the specimen at two locations and positively attached to the loading frame (Figure 3.2). These straps, which measured 3-in wide by 1/8-in thick, were oriented so that they resisted any large lateral movements of the top flange while allowing the specimen to deflect downward freely. To some degree, this lateral support simulates the restraint provided by a concrete deck to highway bridge girders. The full test setup can be seen in Figure 3.3 through Figure 3.6.

3.2.2. Horizontal Stiffener Specimens

The next series of tests to be conducted evaluated the effect of intersecting, or nearly intersecting, longitudinal and transverse web welds where a horizontal stiffener meets a vertical stiffener. Both of the HS specimens had two details to be tested, each offset 1 ft. from midspan. Vertical stiffeners were made of 0.5 in. x 3 in. hot-rolled ASTM A36 plate stock cut with a band saw to 22.625 in. to bear against both flanges. Horizontal stiffeners were made of 0.375 in. x 3 in. hot-rolled ASTM A36 plate stock cut with a band saw to 21.5 in. to achieve the appropriate weld gaps (Figure 3.7). It should be noted that the gaps were produced by placing the horizontal plate either 0.5 in. or 1.5 in. from the vertical stiffener. However, the web gap is less than the plate gap since the weld toes fill some of the space between the stiffener and the end of the horizontal plate. The beams were supported, loaded, and laterally braced in the same manner as the VS specimen series.

3.2.3. Laterally Braced Specimens

The last series of tests to be conducted evaluated the effect of intersecting, or nearly intersecting, longitudinal gusset-to-web welds and transverse stiffener-to-web welds where a horizontal gusset is coped to fit around a vertical stiffener. Each specimen had one detail to be tested, located at midspan (Figure 3.8). Vertical stiffeners on these specimens were made of 0.375 in. x 3 in. A36 plate cut with a band saw to 22.625 in. to

bear against both flanges. The lateral gussets were manufactured in a machine shop from 0.375 in. A36 steel stock to the dimensions specified in Figure 3.9.

The testing of the four lateral gusset specimens involved loading the specimen with a spreader beam to produce four-point bending. The actuator was bolted to the top flange in the center of a 4 ft. long W14x74 spreader beam. This intermediate spreader beam transferred the load to the top flange of the specimen through pin and roller supports, each 2 ft. from midspan. These supports were made from 1 in. thick steel plates and 1 in. diameter steel pins. To account for tilting of the specimen flanges, these supports were grouted to the top flange with hydrostone while a bubble level rested on top to ensure a level loading surface. Much like the stiffener specimens, the spreader had thin steel strapping bolted to the top flange at two locations and positively attached to the loading frame to prevent lateral movement during loading (Figure 3.10).

To simulate the secondary stresses seen in bridges, the specimens had bracing that ran to a parallel secondary specimen 8 ft. away. The two beams were connected by X-bracing and horizontal bracing at both ends and at midspan (Figure 3.11). The X-bracing consisted of a pair of ASTM A36 2.5 in. x 2.5 in. x 0.25 in. angles and the horizontal bracing consisted of a pair of ASTM A36 3.5 in. x 3.5 in. x 0.25 in. angles. It should be noted that due to the size and spacing of the specimens, the X-bracing members are at a much shallower angle (5° above horizontal) than is usually seen in the field.

In addition to the cross-frame bracing, the lateral bracing ran at a 45° angle from the horizontal gusset at specimen midspan to welded gussets both ends of the secondary beam (Figure 3.12). This is consistent with the geometry of K-bracing that is commonly found in practice. The lateral bracing consisted of two ASTM A36 2.5 in. x 2.5 in. x 0.25 in. angles back to back connected with pretensioned stitch bolts every 24 in. As an extra precaution, thin steel straps ran perpendicular between the top flanges of the specimen and secondary beam at two locations to prevent excessive lateral movement of the

specimen at locations away from the cross-bracing (Figure 3.13). The full test setup can be seen in Figure 3.14 through Figure 3.17.

3.3. Fabrication of Welds

All welds were deposited by an American Welding Society certified welder in the Bowen Laboratory. The welding procedures were conducted in a manner similar to those used in shop-fabrication of welded bridge details. The welding was performed downhand using shielded metal arc welding - DC reversed with a Miller Electric Dialarc 250 AC/DC Arc Welding Power Source. All welds were 0.25 in. fillet welds placed with E7018 H4R electrodes.

Similar to shop fabrication, the beam members were on their side with the web positioned horizontally to weld the attachments to the specimen. Based on information collected by observing the welding in the lab, welds along the web were deposited horizontally at an average rate of 5 inches per minute using 5/32" electrodes with the power source set to 130 amps. This corresponds to a heat input of 45 kJ/in. Welds along the flange were deposited vertically at an average rate of 3 inches a minute using 1/8" electrodes with the power source set to 100 amps. Again, the average values give a heat input of 58 kJ/in. for the flange welds. Although no preheat was used, the welding was conducted in the laboratory at ambient room temperature.

The welding was performed with the beams on their side. The welder sat either next to or on the top of the beam and attached the stiffeners and gussets in the following order:

- tack welded the plate to the beam; vertical stiffeners were tack welded to the flange at both ends, horizontal stiffeners and gussets were tack welded to the web (Figure 3.18)

- horizontally placed the fillet weld connecting one side of the plate to the beam web in increments of approximately 5 in., chipping the slag off the end of one bead before starting the next (Figure 3.19 through Figure 3.21)
- placed the fillet weld on the other side of the plate in the same manner
- for vertical stiffeners, the stiffener-to-flange welds were placed from bottom to top along one side of the flange and then the other (Figure 3.22)

3.4. Test Procedure

The test procedure involved the application of loading cycles along with periodic interruptions to monitor the response of the system. The load was applied with an MTS 55 kip actuator. All beams were subjected to a sine-wave shaped cyclic loading, with a stress ratio between 0.03 and 0.05, at a frequency of 1 - 2 Hz. Visual inspections were performed during cyclic loading to detect the onset of fatigue cracking around the detail of interest. A 10X magnifying glass was used for the visual inspections.

The cyclic loading was continued until a crack propagated through the flange or web at one of the fatigue details. At this point, testing was stopped and the crack was retrofitted with stop holes and/or a bolted splice to slow further crack growth. The loading was then continued until a significant crack could be generated at the other detail. With this procedure, multiple cyclic life data could be obtained from each specimen.

3.5. Data Collection

The readings of the LVDT and load cell housed in the actuator were output from the MTS controller to a Campbell Scientific Model CR5000 data acquisition unit. Any strain gages attached to the specimen were attached to bridge completion modules on the same unit. The load, deflection, and strains were recorded periodically during testing.

Table 3.1 Experimental Program

Detail type	No. of specimens	No. of details	Stress range (ksi)	Degree of weld intersection		
				NG	SG	LG
VS	3	6	16.6	2	-	2
			17.6	2	-	-
HS	2	4	12.8	2	-	2
HG	4	4	10.7	2	2	-

Notes:

- VS = vertical stiffener
- HS = vertical stiffener intersecting horizontal stiffeners
- HG = vertical stiffener intersecting a horizontal gusset
- NG (no gap): the two perpendicular welds directly intersect
- SG (small gap): a gap smaller than 0.25 in. exists between the toes of the two perpendicular welds
- LG (large gap): a 0.25 in. or larger gap exists between the toes of the two perpendicular welds

Figure 3.1 VS Specimen section views

Figure 3.2 Steel straps laterally bracing the compression flange of VS and HS specimens

Figure 3.3 Sketch of VS and HS specimen test setup (elevation view)

Figure 3.4 Photo of VS and HS specimen test setup (elevation view)

Figure 3.5 Sketch of VS and HS specimen test setup (profile view)

Figure 3.6 Photo of VS and HS specimen test setup (profile view)

Figure 3.7 HS specimen details

Figure 3.10 Steel strap laterally bracing the spreader beam for HG specimens

(a) Section view of bracing at both beam ends

(b) Section view of bracing at midspan

Figure 3.11 Bolted cross bracing for HG specimens

Figure 3.12 Sketch of HG specimen test setup (plan view)

Figure 3.13 Strap running between HG specimen and secondary beam

Figure 3.14 Sketch of HG specimen test setup (elevation view)

Figure 3.15 Photo of HG specimen test setup (elevation view)

Figure 3.16 Sketch of HG specimen test setup (profile view)

Figure 3.17 Photo of HG specimen test setup (profile view)

(a) Connecting a vertical stiffener to the beam flange

(b) Connecting a horizontal gusset to the beam web

Figure 3.18 Tack welds holding an attachment in place for welding

Figure 3.19 Downhand horizontal placement of a stiffener-to-web

Figure 3.20 Removing the slag at the end of a weld with a slag hammer

Figure 3.21 Weld passes with the slag removed at the end

Figure 3.22 Downhand vertical placement of a stiffener-to-flange weld

CHAPTER 4. EXPERIMENTAL RESULTS

In this chapter, the fatigue test results and observations from each of the specimens are reported. The specimen identification used herein consists of a pair of letters and a number. The pair of letters indicates the type of detail being tested: VS = vertical stiffener, HS = vertical stiffener intersecting two horizontal stiffeners, and HG = vertical stiffener intersecting a horizontal gusset. The number indicates the order in which the specimen was tested within the test series. Elevation views of each type of specimen can be seen in Figure 4.1.

A summary of the crack history and measurement results is provided below for the experimental tests in the three test series. Web gap measurements are provided in Appendix B for all of the tests. A detailed summary of the measured crack sizes and number of loading cycles sustained for each test specimen is given in Appendix C.

4.1. VS Test Series

4.1.1. Specimen VS-1

The first specimen tested was a 16 ft. span W24x55 shape with two welded transverse stiffeners, each 1 ft. from the center of the beam. Both stiffeners were purposely welded to achieve a “no gap” condition in which the stiffener-to-flange and stiffener-to-web welds completely intersected.

The load applied to the specimen cycled at constant amplitude between 2.5 kips and 50 kips. This load range produced a stress ratio of 0.05 and an average nominal stress range

of 16.6 ksi at the level of the stiffener-to-flange weld toes. The actual nominal stresses at the weld toes vary slightly and are discussed in more detail in Chapter 5. The test was run at 1 Hz for 400,000 cycles after which the loading rate was slowly incremented to 2 Hz, reached at 900,000 cycles. The test continued until cracking at one of the stiffeners passed through the flange at 4,726,000 cycles. The flange was repaired with a bolted splice detail and the test was then continued until cracking at the other stiffener passed through the flange at 5,842,000 cycles.

4.1.1.1. Crack History

For brevity, the locations where cracking is likely to occur in each specimen will be cited according to the detail legend shown in Figure 4.2. Table 4.1 lists these possible crack locations for all of the specimens and more detailed information on the gap condition and AASHTO fatigue category classification for each detail.

The first possible sign of cracking in Specimen VS-1 appeared at 950,000 cycles at the weld intersection at Location C. There was a slight bulging of the weld material at the termination of the stiffener-to-flange weld. Oil placed along the weld could be observed to be “breathing” with each loading cycle. It was uncertain whether this was due to a crack or a small groove that expanded under the applied load. At 3,530,000 cycles, a crack appeared to be moving from this area outward across the flange. However, the crack seemed to arrest at a length of 1/8” for the remainder of the test.

At 4,550,000 cycles, significant cracking had formed along a 2.25 in. length of the stiffener-to-flange weld at Location D (see Figure 4.3). The cracking appeared to be the coalescence of several smaller cracks as it measured over 2 in. long but had not yet propagated downward through the 0.5 in. flange. By 4,726,000 cycles, the crack had grown such that a 1 in. (Figure 4.4) crack length could be seen on the underside of the flange and the test was stopped. A 1 in. thick bolted flange doubler plate was installed to bridge the crack (Figure 4.5) and testing was resumed. As the test continued, cracking at Location D was monitored and did not significantly grow with the splice in place.

At 5,150,000 cycles, a crack appeared to have grown from the weld intersection 0.625 in. vertically into the web at Location B. The crack was detected by the application of oil. It seemed to arrest in the web and not propagate all the way through the thickness of the web or flange. At 5,675,000 cycles, a crack had grown to 1.75 in. along the toe of the stiffener-to-flange weld at Location B (Figure 4.6). This crack quickly grew and likely coalesced with other cracks so that, at 5,750,000 cycles, the cracking extended along the entire weld toe. At 5,842,319 cycles, the crack had grown through the tension flange (Figure 4.7) and testing was stopped.

4.1.1.2. Data Collection

The 47.5 kip load range for this test was kept constant by an MTS 458 Controller. The actuator stroke for this load range was recorded by the LVDT housed in the actuator. This stroke includes the downward deflection of the specimen as well as the upward deflection of the frame with loading. Strain measurements were recorded at the four locations on the bottom of the tension flange denoted in Figure 4.8. The mean and standard deviation of the data recorded during the test are shown in Table 4.2.

4.1.2. Specimen VS-2

The second specimen tested was a 16 ft. span W24x55 shape with two welded transverse stiffeners, each 1 ft. from the center of the beam. Both stiffeners were purposely welded to achieve a “no gap” condition in which the stiffener-to-flange and stiffener-to-web welds completely intersected.

The load applied to the specimen cycled at constant amplitude between 2.5 kips and 53 kips. This load produced a stress ratio of 0.047 and an average nominal stress range of 17.6 ksi at the level of the stiffener-to-flange weld toes. The test was run at 1 Hz for 60,000 cycles at which point it was retrofitted to alleviate weak-axis bending. The test was then resumed and the rate increased to 2 Hz. Cracking was first detected at one of

the stiffeners at 1,200,000 cycles and had propagated through the flange by 1,690,000 cycles. The flange was repaired with a bolted doubler plate and the test was then continued. Cracks appeared at the other stiffener at 1,770,000 cycles and had propagated through the flange by 2,300,000 cycles.

4.1.2.1. Crack History

Specimen VS-2 was detailed in the same manner as Specimen VS-1 and both had a problem with secondary bending of the tension flange. Though the top flange was laterally braced with straps, the web and bottom flange were not restricted from moving lateral. The vertical stiffeners were only welded on one side of the web and the enormous heat from the welding process caused some degree of warping of the cross section (Figure 4.9) that had not been accounted for. Because the bottom flange tilted to one side, the downward loading from the actuator was eccentric to the flange center. This caused a certain degree of weak-axis bending of the unbraced bottom flange. This is evidenced by the much higher strains occurring on one side of the cross section than the other. After 60,000 cycles of loading – during which the strains on the bottom flange were imbalanced – the test was stopped so the specimen could be modified to alleviate the problem.

The retrofit procedure detailed in Figure 4.10 was used to try to straighten the beam and balance the section while avoiding a situation that would crack the tension flange before the detail of interest. The retrofit was installed at both vertical stiffener locations on the opposite side of the web. Any cracking at the transverse horizontal welds on the retrofit would simply propagate through the bolted lateral plate (Figure 4.11) and be isolated from the tension flange. After the retrofit, the specimen was returned to the test setup and the loading continued in the same manner as before.

The first sign of cracking appeared at 1,200,000 cycles when the application of oil showed cracking along the stiffener-to-flange weld toe at Location B. Most likely the coalescence of several smaller cracks, the cracking extended transversely from the outer

edge to roughly the center of the weld toe (Figure 4.12). At 1,250,000 cycles, potential cracking could also be seen in the center of the stiffener-to-flange weld toe at Location D (Figure 4.13). At 1,410,000 cycles, the application of oil showed possible cracking at the weld intersection at Location C but the evidence remained inconclusive throughout the testing period. By 1,690,000 cycles, the cracking at Location B was over 2 in. long and had propagated through the tension flange. At this point, the testing was stopped to install a 1 in. thick bolted flange doubler plate.

Testing was resumed, and at 1,770,000 cycles cracking could be seen at a second location along the stiffener-to-flange weld toe at Location D. This cracking was clearly separate from that seen earlier and was located closer to the web. At 1,890,000 cycles, small cracking could also be seen at two locations along the stiffener-to-flange weld toe at Location C: one location was at a slight bulge of weld material in the center of the weld toe and the other was at the intersection with the stiffener-to-web weld (Figure 4.14). However, the cracking at Location D grew more quickly. By 2,180,000 cycles the two cracks there had coalesced into one, and by 2,300,000 cycles they had propagated through the flange thickness.

4.1.2.2. Data Collection

The 50.5 kip load range for this test was kept constant by an MTS 458 Controller. The actuator stroke for this load range was recorded by the LVDT housed in the actuator. This stroke includes the downward deflection of the specimen as well as the upward deflection of the frame with loading. Strain measurements were recorded at the same four locations as Specimen VS-1 (Figure 4.8). The mean and standard deviation of the data recorded during the test are shown in Table 4.3.

4.1.3. Specimen VS-3

The third specimen tested was a 16 ft. span W24x55 shape with two welded transverse stiffeners, each 1 ft. from the center of the beam. Unlike the previous two specimens, the

stiffeners for Specimen VS-3 were clipped 0.75 in. resulting in a “safe” gap of 1 in. between the stiffener-to-flange and stiffener-to-web welds.

The load applied to the specimen was cycled at a constant amplitude between 2.0 kips and 52.5 kips. This load produced a stress ratio of 0.038 and an average nominal stress range of 17.6 ksi at the level of the stiffener-to-flange weld toes. Before being tested, the beam was retrofitted in the same manner as Specimen VS-2 to alleviate local warping of the cross section. The loading frequency started at 0.5 Hz and was incrementally increased to 2.0 Hz by 4,700 cycles. The test ran until cracking propagated through the flange at one of the stiffeners at 2,170,000 cycles. The flange was repaired with a bolted doubler plate and loading was then continued until 4,050,000 cycles when, with no clear evidence of cracking at any other locations, the test was considered a runout.

4.1.3.1. Crack History

The first potential sign of cracking appeared at 1,300,000 cycles at Location C. When oil was applied along the stiffener-to-flange weld toe, a small amount – at the very outside edge of the flange – could be seen moving with each load cycle. Cracking at this location was finally clearly evident at 1,923,000 cycles when a 0.5 in. long crack could be seen along the weld toe. The cracking moved across the weld toe and through the flange (Figure 4.15) and by 2,170,000 cycles it was 2 in. long (the entire length of the weld toe) and through the entire depth of the flange (Figure 4.16). At this point the cracked detail was repaired with a 1 in. thick bolted flange doubler plate.

Throughout the test, there was very little evidence of cracking at any other locations. The only potential cracking was seen along a very small length of the weld toe at Location A near the edge of the flange. This was first seen at 2,960,000 cycles but no additional growth occurred and the test was considered a runout at 4,050,000 cycles.

4.1.3.2. Data Collection

The 50.5 kip load range for this test was kept constant by an MTS 458 Controller. The actuator stroke for this load range was recorded by the LVDT housed in the actuator. This stroke includes the downward deflection of the specimen as well as the upward deflection of the frame with loading. Strain measurements were recorded at 3 locations on the bottom of the tension flange at midspan as seen in Figure 4.17. The mean and standard deviation of the data recorded during the test are shown in Table 4.4.

4.2. HS Test Series

4.2.1. Specimen HS-1

The fourth specimen tested was a 16 ft. span W24x55 shape with two welded vertical stiffeners, each 1 ft. from the center of the beam and three horizontal stiffeners welded to the web 3 in. above the tension flange. The vertical stiffeners both had a “no gap” condition in which the stiffener-to-flange and stiffener-to-web welds completely intersected. The details of interest, however, were where the horizontal stiffeners met the vertical ones. At one detail there was a 0.5 in. gap between the stiffeners resulting in little to no gap between the perpendicular weld toes (Figure 4.18 and Figure 4.19). At the other detail there was a 1.25 in. gap between the perpendicular plates resulting in a large gap (around 0.875 in.) between the weld toes (Figure 4.20 and Figure 4.21).

The load applied to the specimen cycled at constant amplitude between 2.5 kips and 52.5 kips. This load produced a stress ratio of 0.048 and an average nominal stress range of 12.5 ksi at the midheight of the horizontal stiffener at the details of interest. The test was run at 1.5 Hz for 100,000 cycles, after which the loading rate was increased to 2 Hz. The test continued until cracking in one of the small web gaps propagated through the web at 675,000 cycles. At this point, the crack was retrofitted by hole-drilling at both ends. At 950,000 cycles, the same retrofit was made where cracking in one of the large web gaps had propagated through the web. Cracking reinitiated from the lower hole of the first

retrofit at 1,060,000 cycles and the test was stopped to reinforce the area with a bolted flange splice at 1,080,000 cycles. The test was resumed until a crack initiated at the lower hole of the second retrofit at 2,087,000 cycles.

4.2.1.1. Crack History

The first possible signs of cracking were seen as early as 250,000 cycles at the weld toes at the termination of the horizontal stiffener-to-web welds at Location C. This could have been the result of a large undercut coupled with a large stress concentration factor but at 355,000 cycles cracking could clearly be seen moving vertically into the web from the upper weld toe. By 380,000 cycles, this cracking joined with cracking at the lower weld toe. The combined crack continued to move vertically and across the web. At 455,000 cycles, similar behavior was first seen at Location B. Cracking was soon confirmed at the weld toes at this location and by 625,000 cycles the cracks had joined.

By 675,000 cycles, the cracking at Location C could be seen on the opposite side of the web. The test was then stopped and the detail retrofitted by using a magnetic drill (Figure 4.22) to drill two 1.625 in. diameter stop holes capturing both ends of the crack (Figure 4.23). At 760,000 cycles, cracking was found at Location B. The cracking extended between the horizontal weld toes and 0.125 in. down the web. By 950,000 cycles, the cracking at location B had breached the web and the web was retrofitted in the same manner as Location C (Figure 4.24). By 1,060,000 cycles, a crack had initiated at the bottom retrofit hole at Location C. The crack grew quickly so at 1,080,000 cycles the test was stopped to reinforce the flange below this area with a bolted splice.

At 1,100,000 cycles, cracking was seen along the vertical stiffener-to-web weld toe at Location C near the lower retrofit hole. The cracking at Location A completely propagated through the web at 1,770,000 cycles but the test was tentatively run without a retrofit there. At 2,040,000 cycles, cracking along the toe of the vertical stiffener-to-web weld could be seen near the lower retrofit hole at Location B. Then, at 2,087,000 cycles, a crack reinitiated in said retrofit hole and the test was stopped.

4.2.1.2. Data Collection

The 50.5 kip load range for this test was kept constant by an MTS 458 Controller. The actuator stroke for this load range was recorded by the LVDT housed in the actuator. This stroke includes the downward deflection of the specimen as well as the upward deflection of the frame with loading. Strain measurements were recorded at the same 4 locations as specimens VS-1 and VS-2 (see Figure 4.8). The mean and standard deviation of the data recorded during the test are shown in Table 4.5.

4.2.2. Specimen HS-2

The fifth specimen tested was a 16 ft. span W24x55 shape with two welded vertical stiffeners, each 1 ft. from the center of the beam and three horizontal stiffeners welded to the web 3 in. above the tension flange. The vertical stiffeners both had a “no gap” condition in which the stiffener-to-flange and stiffener-to-web welds completely intersected. The details of interest, however, were where the horizontal stiffeners met the vertical ones. At one detail there was a 0.5 in. gap between the perpendicular stiffeners resulting in little to no gap between the weld toes. At the other detail there was a 1.25 in. gap between the perpendicular stiffeners resulting in a large gap (around 0.875 in.) between the weld toes.

The load applied to the specimen cycled at constant amplitude between 2.5 kips and 52.5 kips. This load produced a stress ratio of 0.048 and an average nominal stress range of 12.5 ksi at the midheight of the horizontal stiffener at the details of interest. The loading rate was kept at 2 Hz for the entire test. The test was run until cracking in one of the small web gaps propagated through the web at 950,000 cycles. At this point, the crack was retrofitted by hole-drilling at both ends. At 1,200,000 cycles, the same retrofit was made where cracking in one of the large web gaps had propagated through the web. The test was resumed until a crack re-initiated at a retrofit hole at 1,640,000 cycles.

4.2.2.1. Crack History

The first possible signs of cracking were seen as early as 545,000 cycles at the weld toes at the termination of the horizontal stiffener-to-web welds at Location D. At 600,000 cycles cracking could clearly be seen moving vertically into the web from the lower weld toe on the outside of the detail. By 673,000 cycles, this cracking extended from the upper weld toe 0.875 in. below the lower weld toe. At 835,000 cycles, cracking was first seen at both the upper and lower weld toes at Location B. Soon after, at 850,000 cycles, cracking was reported between the weld toes at Location C. The cracking at Location D continued to move vertically and across the web and at 950,000 cycles could be seen from the other side of the web. The test was then stopped and the detail retrofitted by drilling 1.875 in. diameter stop holes at both ends of the crack.

Testing was then resumed, and by 1,200,000 cycles the cracking at Location B had passed through the web. The test was stopped and the detail retrofitted in the same manner as the previous ones. The test continued with no new signs of cracking at any of the other locations until 1,640,000 cycles when a crack reinitiated at the lower retrofit hole at Location D. At this point, testing was stopped.

4.2.2.2. Data Collection

The 50 kip load range for this test was kept constant by an MTS 458 Controller. The actuator stroke for this load range was recorded by the LVDT housed in the actuator. This stroke includes the downward deflection of the specimen as well as the upward deflection of the frame with loading. Strain measurements were recorded at four locations on the web and bottom flange as seen in Figure 4.26. The mean and standard deviation of the data recorded during the test are shown in Table 4.6.

4.3. HG Test Series

4.3.1. Specimen HG-1

The sixth specimen tested was a 16 ft. span W24x55 shape with a vertical stiffener intersecting a horizontal gusset plate at midspan. The vertical stiffener was clipped 0.75 in. resulting in a “safe” gap of 1 in. between the stiffener-to-flange and stiffener-to-web welds. The detail of interest, however, was where the vertical stiffener intersected the horizontal gusset. The horizontal gusset was coped to leave a 0.5625 in. gap between the perpendicular plates. This resulted in a gap of 0.1875 in. between the vertical stiffener-to-web weld toes and the top-and-bottom horizontal gusset-to-web weld toes (Figure 4.27).

The load applied to the specimen cycled at constant amplitude between 1.5 kips and 51.5 kips. This produced a stress ratio of 0.029 and a nominal stress range of 10.7 ksi at the midheight of the horizontal stiffener. The loading rate was kept at 2 Hz for the entire test. The test was run until cracking into the web from the weld toe at the outside edge of the gusset plate grew to a length of 1.1875 in. at 3,363,000 cycles. At this point, the test was stopped.

4.3.1.1. Crack History

The first sign of cracking occurred at the weld toe at the termination of the lower longitudinal gusset-to-web weld at Location A (Figure 4.28). The cracking could be seen moving vertically in the web at 2,080,000 cycles. By 3,360,000 cycles, the crack had grown to a length of 1.1875 in. but was not yet visible through the web. At this point, the test was stopped.

4.3.1.2. Data Collection

The 50 kip load range for this test was kept constant by an MTS 458 Controller. The actuator stroke for this load range was recorded by the LVDT housed in the actuator. This stroke includes the downward deflection of the specimen as well as the upward deflection of the frame with loading. Strain measurements were recorded at the eight flange locations shown in Figure 4.29 on both the test specimen and the secondary beam. These strains were recorded manually from Specimen HG-1 with a strain indicator and switch and balance unit under a static 50 kip load. The strains on the secondary beam were recorded during testing with the CR5000 data acquisition system. The strains at the six locations on the web of Specimen HG-1 (Figure 4.30) were also recorded with the CR5000. The mean and standard deviation of the data recorded from the flange and web during the test are shown in Table 4.7 and Table 4.8, respectively.

4.3.2. Specimen HG-2

The seventh specimen tested was a 16 ft. span W24x55 shape with a vertical stiffener intersecting a horizontal gusset plate at midspan. The vertical stiffener was clipped 0.75 in. resulting in a “safe” gap of 1 in. between the stiffener-to-flange and stiffener-to-web welds (Figure 4.31). The detail of interest, however, was where the vertical stiffener intersected the horizontal gusset. The horizontal gusset was coped to leave a 0.25 in. gap between the perpendicular plates. This resulted in an intersecting weld condition where the top-and-bottom horizontal gusset-to-web welds overlapped the vertical stiffener-to-web welds.

The load applied to the specimen cycled at constant amplitude between 1.5 kips and 51.5 kips. This produced a stress ratio of 0.029 and a nominal stress range of 10.7 ksi at midheight of the horizontal stiffener. The loading rate was kept at 2 Hz for the entire test. The test was run until cracking at the weld toe at one of the outside edges of the gusset propagated through the web at 1,650,000 cycles. The area was retrofitted with a bolted web splice and the test was resumed. By 2,875,000 cycles, cracking at the other end of the gusset also propagated through the web and that area was repaired with crack

stop-holes and a bolted splice. The test was continued until 3,300,000 cycles with no cracking observed in either web gap.

4.3.2.1. Crack History

The first signs of cracking on Specimen HG-2 were seen at the toe of the wraparound weld at Location D (Figure 4.32). At 800,000 cycles, the application of oil suggested cracking at the location. By 875,000 cycles, cracking could clearly be seen moving from the weld toe vertically into the web. By 1,650,000 cycles, the cracking was over 1.5 in. long and had propagated through the web. At this time, the test was stopped and the area was retrofitted by applying a bolted web doubler plate that bridged the crack (Figure 4.33). The Hougen magnetic drill used for all the splices and retrofit holes on the HG specimens is shown in Figure 4.34.

The testing continued and at 1,740,000 cycles cracking could be seen at the toe of the wraparound weld at Location A. At 1,950,000 cycles, oil applied at Location B could be seen moving with each cycle but it was inconclusive that it was caused by cracking. By 2,875,000 cycles, the cracking at Location A had passed through the web. The area was retrofitted with a bolted web doubler plate and two 0.8125 in. diameter crack stop holes (Figure 4.35). As the testing continued, the evidence of potential cracking at Location B remained inconsistent. The test resumed until, at 3,200,000 cycles with no new evidence of cracking, it was considered a runout.

4.3.2.2. Data Collection

The 50 kip load range for this test was kept constant by an MTS 458 Controller. The actuator stroke for this load range was recorded by the LVDT housed in the actuator. This stroke includes the downward deflection of the specimen as well as the upward deflection of the frame with loading. Strain measurements for this load range were recorded at eight flange locations (Figure 4.29) and six web locations (Figure 4.30) on Specimen HG-2 with the CR5000 data acquisition system. They were not recorded on

the secondary beam. The load, stroke, and strains for the uncracked section remained relatively constant. The mean and standard deviation of the data recorded from the uncracked section of Specimen HG-2 are shown in Table 4.9 and Table 4.10.

4.3.3. Specimen HG-3

The eighth specimen tested was a 16 ft. span W24x55 shape with a vertical stiffener intersecting a horizontal gusset plate at midspan. The vertical stiffener was clipped 0.75 in. resulting in a “safe” gap of 1 in. between the stiffener-to-flange and stiffener-to-web welds. The detail of interest, however, was where the vertical stiffener intersected the horizontal gusset. The horizontal gusset was coped to leave a 0.25 in. gap between the perpendicular plates. This resulted in an intersecting weld condition where the top-and-bottom horizontal gusset-to-web welds overlapped the vertical stiffener-to-web welds (Figure 4.36).

The load applied to the specimen cycled at constant amplitude between 1.5 kips and 51.5 kips. This load produced a stress ratio of 0.029 and a nominal stress range of 10.7 ksi at the level of the center of the horizontal stiffener. The loading rate was kept at 2 Hz for the entire test.

In an attempt to introduce imbalanced out-of-plane forces into the web gap, only one of the lateral braces (the northern one) was fastened with fully pretensioned bolts to the gusset. The other later lateral brace was connected to the gusset with bolts that were merely finger-tightened at the beginning of the test.

The test was run until cracking from the weld toes at the both outside edges of the gusset plate propagated through the web at 2,150,000 cycles. Both locations were retrofitted with drilled holes and the test resumed. When a crack reinitiated from one of the retrofit holes, the area was reinforced by clamping a large plate to the bottom flange. The test was continued until 3,500,000 cycles with no evidence cracking observed in either web gap.

4.3.3.1. Crack History

The first signs of cracking on Specimen HG-3 were seen at the termination of the lower gusset-to-web weld at Location A. At 650,000 cycles, the application of oil revealed a 0.5 in. long crack growing vertically in the web. At 955,000 cycles, a similar crack had formed at location D. By 1,350,000 cycles, the finger-tightened bolts on the south side of the gusset plate had clearly started to work themselves loose (Figure 4.37). By 1,825,000 cycles, the cracking at Location A had propagated such that a 0.75 in. vertical crack could be seen on the opposite side of the web. At 2,050,000 cycles, Location A was retrofitted by drilling 1.5 in. diameter holes at both ends of the crack (Figure 4.38). At this point, cracking at Location D had also propagated through the web. At 2,150,000 cycles, Location D was retrofitted by drilling 1.5 in. holes at both ends of the crack (Figure 4.39).

Testing resumed with no new observed cracking until 2,800,000 cycles, when cracking re-initiated in the lower retrofit hole at Location A. In an attempt to slow this crack growth, a doubler plate was clamped to the tension flange below the retrofit hole with large bridge clamps. Sand was placed between the plate and flange for a larger coefficient of friction and more load transfer (Figure 4.40). At 3,000,000 cycles, a crack re-initiated at the lower retrofit hole at Location D. The test was stopped and the area reinforced in the same manner as Location A. Testing was continued until 3,500,000 cycles with no evidence of cracking in the web gap.

4.3.3.2. Data Collection

The 50 kip load range for this test was kept constant by an MTS 458 Controller. The actuator stroke for this load range was recorded by the LVDT housed in the actuator. This stroke includes the downward deflection of the specimen as well as the upward deflection of the frame with loading. Strain measurements for this load range were recorded at four locations on the bottom flange (Figure 4.29) and six locations on the web (Figure 4.30) of Specimen HG-3 with the CR5000 data acquisition system. They were not recorded on the secondary beam. The load, stroke, and strains for the uncracked section remained relatively constant. The mean and standard deviation of the data

recorded from the uncracked section of Specimen HG-3 are shown in Table 4.11 and Table 4.12.

4.3.4. Specimen HG-4

The ninth and final specimen tested was a 16 ft. span W24x55 shape with a vertical stiffener intersecting a horizontal gusset plate at midspan. The vertical stiffener was clipped 0.75 in. resulting in a “safe” gap of 1 in. between the stiffener-to-flange and stiffener-to-web welds. The detail of interest, however, was where the vertical stiffener intersected the horizontal gusset. The horizontal gusset was coped to leave a 0.5625 in. gap between the perpendicular plates. This resulted in a gap of 0.1875 in. between the vertical stiffener-to-web weld toes and the top-and-bottom horizontal gusset-to-web weld toes (Figure 4.41).

The load applied to the specimen cycled at constant amplitude between 1.5 kips and 51.5 kips. This load produced a stress ratio of 0.029 and a nominal stress range of 10.7 ksi at midheight of the horizontal stiffener. The loading rate was kept at 2 Hz for the entire test.

In an attempt to introduce imbalanced out-of-plane forces into the web gap, only one of the lateral braces (the northern one) was fastened with fully pretensioned bolts to the gusset. The other later lateral brace was connected to the gusset with bolts that were merely finger-tightened at the beginning of the test.

The test was run until cracking at one end of the gusset passed through the web at 3,350,000 cycles. At this point, the test was paused and the area retrofitted with retrofit holes. The test was resumed and cracking eventually occurred in one of the web gaps and passed through the web at 5,250,000 cycles. No new cracking was noted over the length of the test and testing was concluded at 6,250,000 cycles.

4.3.4.1. Crack History

At first, Specimen HG-4 tended to move laterally under cyclic loading, with either end moving as much as 1 in. after just 20,000 cycles. This could have been caused by warping of the specimen from the welding process or by introducing stress into the bracing members before testing by forcing them to fit. The problem was eventually alleviated by positively attaching thin steel straps to the loading frame as well as to the north (Figure 4.42) and south (Figure 4.43) ends of the specimen. These straps were able to restrain the ends of the specimen from any more lateral movement.

The first sign of cracking was seen with the application of oil at the upper horizontal weld termination at Location D (Figure 4.44) at 1,000,000 cycles. This crack grew downward along the weld toes and across the web and eventually propagated through the entire web thickness at 3,350,000 cycles. At this point, the test was stopped and two 1.5 in. retrofit holes were drilled at the ends of the crack (Figure 4.45). Cracking was observed along the weld toes at the weld terminations at Location A (Figure 4.46) as early as 1,620,000 cycles but this never led to significant cracking. Cracking was observed in the web gap when a 1 in. length of shallow cracking was seen along the toe of the vertical stiffener-to-web weld toe at Location C at 4,125,000 cycles. This cracking propagated through the web at 5,250,000 cycles.

The test was continued with no attempts to repair Location C until, at 6,250,000 cycles, with no evidence of reinitiation at the retrofit holes drilled at Location D, the test was concluded.

4.3.4.2. Data Collection

The 50 kip load range for this test was kept constant by an MTS 458 Controller. The actuator stroke for this load range was recorded by the LVDT housed in the actuator. This stroke includes the downward deflection of the specimen as well as the upward deflection of the frame with loading. Strain measurements for this load range were recorded at four locations on the bottom flange (Figure 4.29) and six locations on the web

(Figure 4.30) of Specimen HG-4 with the CR5000 data acquisition system. They were not recorded on the secondary beam. The load, stroke, and strains for the uncracked section remained relatively constant. The mean and standard deviation of the data recorded from the uncracked section of Specimen HG-4 are shown in Table 4.13 and Table 4.14.

Table 4.1 Classification of details on fatigue specimens

Detail type	Specimen number	Detail Location	Weld Gap Condition	AASHTO Fatigue Category
VS	1	A, B, C, D	NG	C'
	2	A, B, C, D	NG	C'
	3	A, B, C, D	LG	C'
HS	1	A, B	LG	E
		C, D	SG	E
	2	A, B	LG	E
		C, D	SG	E
HG	1	A, D	N/A*	E
		B, C	NG	E
	2	A, D	N/A*	E
		B, C	SG	E
	3	A, D	N/A*	E
		B, C	NG	E
	4	A, D	N/A*	E
		B, C	SG	E

Notes:

- VS = vertical stiffener
- HS = vertical stiffener intersecting horizontal stiffeners
- HG = vertical stiffener intersecting a horizontal gusset
- NG (no gap): the two perpendicular welds directly intersect
- SG (small gap): a gap smaller than 0.25 in. exists between the toes of the two perpendicular welds
- LG (large gap): a 0.25 in. or larger gap exists between the toes of the two perpendicular welds

* - detail is at the outside end of a shelf plate

Table 4.2 Mean and standard deviation of the data collected from Specimen VS-1

	Load (kip)	Stroke (in.)	Reading at strain gage ($\mu\epsilon$)			
			SE	SW	NE	NW
mean	47.57	0.222	558	504	577	511
st. dev.	0.114	0.0030	5.4	5.2	4.8	4.3

Table 4.3 Mean and standard deviation of the data collected from Specimen VS-2

	Load (kip)	Stroke (in.)	Reading at strain gage ($\mu\epsilon$)			
			SE	SW	NE	NW
mean	50.47	0.249	521	504	533	513
st. dev.	0.061	0.0024	2.1	5.8	5.2	5.3

Table 4.4 Mean and standard deviation of the data collected from Specimen VS-3

	Load (kip)	Stroke (in.)	Reading at strain gage ($\mu\epsilon$)		
			E	C	W
mean	50.52	0.250	598	593	583
st. dev.	0.036	0.0006	5.4	4.6	4.2

Table 4.5 Mean and standard deviation of the data collected from Specimen HS-1

	Load (kip)	Stroke (in.)	Reading at strain gage ($\mu\epsilon$)			
			SE	SW	NE	NW
mean	49.95	0.251	584	553	503	482
st. dev.	0.030	0.0016	6.6	5.3	5.6	4.6

Table 4.6 Mean and standard deviation of the data collected from Specimen HS-2

	Load (kip)	Stroke (in.)	Reading at strain gage ($\mu\epsilon$)			
			W1	W2	F-W	F-E
mean	49.94	0.231	200	230	414	530
st. dev.	0.015	0.0075	4.5	4.1	4.3	4.2

Table 4.7 Mean and standard deviation of the flange strain data collected from Specimen HG-1

	Reading at strain gage ($\mu\epsilon$)							
	TF-NE	TF-NW	BF-NE	BF-NW	TF-SE	TF-SW	BF-SE	BF-SW
mean	-293	-303	300	293	-302	-281	285	295
st. dev.	3.2	5.2	5.6	2.1	5.9	6.3	3.4	3.3

Table 4.8 Mean and standard deviation of the load, stroke, and web strain data collected from Specimen HG-1

	Load (kip)	Stroke (in.)	Reading at strain gage ($\mu\epsilon$)					
			W1	W2	W3	W4	W5	W6
mean	50.06	0.229	234	39	294	285	42	226
st. dev.	0.024	0.0020	2.7	4.9	2.9	3.8	3.2	2.8

Table 4.9 Mean and standard deviation of the flange strain data collected from Specimen HG-2 prior to first cracking

	Reading at strain gage ($\mu\epsilon$)							
	TF-NE	TF-NW	BF-NE	BF-NW	TF-SE	TF-SW	BF-SE	BF-SW
mean	-307	-312	316	308	-313	-308	307	--
st. dev.	3.2	1.1	2.9	1.3	5.0	1.1	1.4	--

Table 4.10 Mean and standard deviation of the load, stroke, and web strain data collected from Specimen HG-2 prior to first cracking

	Load (kip)	Stroke (in.)	Reading at strain gage ($\mu\epsilon$)					
			W1	W2	W3	W4	W5	W6
mean	49.93	0.24217	260	47	274	283	53	266
st. dev.	0.042	0.00465	5.7	0.9	4.9	2.5	0.7	0.9

Table 4.11 Mean and standard deviation of the flange strain data collected from Specimen HG-3 prior to first cracking

	Reading at strain gage ($\mu\epsilon$)			
	BF-NE	BF-NW	BF-SE	BF-SW
mean	125	319	301	341
st. dev.	2.4	0.7	1.2	5.8

Table 4.12 Mean and standard deviation of the load, stroke, and web strain data collected from Specimen HG-3 prior to first cracking

	Load (kip)	Stroke (in.)	Reading at strain gage ($\mu\epsilon$)					
			W1	W2	W3	W4	W5	W6
mean	49.99	0.246	262	80	285	292	102	253
st. dev.	0.009	0.00123	2.1	2.1	0.7	0.1	1.4	1.1

Table 4.13 Mean and standard deviation of the flange strain data collected from Specimen HG-4 prior to first cracking

	Reading at strain gage ($\mu\epsilon$)			
	BF-NE	BF-NW	BF-SE	BF-SW
mean	297	286	287	293
st. dev.	2.5	2.1	4.1	3.6

Table 4.14 Mean and standard deviation of the load, stroke, and web strain data collected from Specimen HG-4 prior to first cracking

	Load (kip)	Stroke (in.)	Reading at strain gage ($\mu\epsilon$)					
			W1	W2	W3	W4	W5	W6
mean	49.95	0.236	240	55	280	262	50	239
st. dev.	0.114	0.0030	1.8	1.5	2.1	1.6	4.2	1.5

a) VS Specimens

b) HS Specimens

c) HG Specimens

Figure 4.1 Elevation view of specimen types

(a) VS specimens

(b) HS specimens

(c) HG specimens

Figure 4.2 Location of details on beam specimens (profile view)

Figure 4.3 Cracking along the weld toe at Location D of Specimen VS-1 (highlighted)

Figure 4.4 Cracking at Location D of Specimen VS-1 propagated through the flange

Figure 4.5 Bolted splice of Location D of Specimen VS-1

Figure 4.6 Cracking along the weld toe at Location B of Specimen VS-1

Figure 4.7 Cracking at Location B of Specimen VS-1 after propagating through the flange

Figure 4.8 Placement of strain gages for Specimen VS-1

Figure 4.9 Local warping of the cross-section of VS specimens

Figure 4.10 Retrofit to alleviate warping of Specimens VS-2 and VS-3

Figure 4.11 Retrofit to alleviate warping of Specimens VS-2 and VS-3

Figure 4.12 Cracking at Location B of Specimen VS-2 (highlighted)

Figure 4.13 Cracking at Location D of Specimen VS-2 (highlighted)

Figure 4.14 Cracking at Location C of Specimen VS-2 (highlighted)

Figure 4.15 Cracking along the weld toe at Location C of Specimen VS-3 (highlighted)

Figure 4.16 Cracking at Location C of Specimen VS-3 propagated through the tension flange

Figure 4.17 Placement of strain gages for Specimen VS-3

Figure 4.18 Web gap at Location C of Specimen HS-1

Figure 4.19 Web gap at Location D of Specimen HS-1

Figure 4.20 Web gap at Location A of Specimen HS-1

Figure 4.21 Web gap at Location B of Specimen HS-1

Figure 4.22 Magnetic drill used to perform retrofits on VS and HS specimens

Figure 4.23 Location C of Specimen HS-1 after retrofit

Figure 4.24 Location B of Specimen HS-1 after retrofit

Figure 4.25 Placement of strain gages for Specimen HS-1

Figure 4.26 Placement of strain gages for Specimen HS-2

Figure 4.27 Web gap at Location B of Specimen HG-1

Figure 4.28 Horizontal weld termination at Location A of Specimen HG-1

Figure 4.29 Placement of flange strain gages on HG specimens

Figure 4.30 Placement of web strain gages for HG specimens

Figure 4.31 Web gap at Location B of Specimen HG-2

Figure 4.32 Weld turnaround at Location D of Specimen HG-2

Figure 4.33 Location D of Specimen HG-2 after being retrofitted with a bolted web splice

Figure 4.34 Magnetic drill used to drill retrofits on HG specimens

Figure 4.35 Location A of Specimen HG-2 after being retrofitted with a bolted web splice and two 0.8125 diameter retrofit holes

Figure 4.36 Web gap at Location C of Specimen HG-3

Figure 4.37 Bolts on the south end of the gusset of Specimen HG-3 working free

Figure 4.38 Retrofit holes drilled at Location A of Specimen HG-3 (seen from opposite side of web)

Figure 4.39 Retrofit holes drilled at Location D of Specimen HG-3 (seen from opposite side of web)

Figure 4.40 Reinforcement of retrofit of Location A of Specimen HG-3 with a large plate clamped to the tension flange

Figure 4.41 Web gap at Location B of Specimen HG-4

Figure 4.42 Strap attached to the north end of Specimen HG-4 to prevent lateral movement

Figure 4.43 Strap attached to the south end of Specimen HG-4 to prevent lateral movement

Figure 4.44 Horizontal weld termination at Location D of Specimen HG-4

Figure 4.45 Retrofit holes at Location D of Specimen HG-4

Figure 4.46 Horizontal weld termination at Location A of Specimen HG-4

CHAPTER 5. DISCUSSION OF EXPERIMENTAL BEHAVIOR

In this chapter, the results of the experimental testing are discussed in the context of the overall research objective: finding out what effect, if any, intersecting weld conditions have on the fatigue lives of steel bridge details. Also, the behavior of the retrofit details is described in detail.

5.1. Experimental Behavior of Welded Fatigue Details

In this section, the fatigue behavior of each specimen is discussed and the results compared to prior research. As a basis for comparison, the design (N_D) and mean (N_M) expected lives of each detail are calculated using the following equations:

$$N_D = \frac{A_D}{(S_r)^3}$$

$$N_M = \frac{A_M}{(S_r)^3}$$

where S_r is the nominal stress range applied to the detail in ksi and A_D and A_M are constants obtained from Table 5.1. These equations are derived from the statistical analysis of a comprehensive collection of large-scale experimental fatigue data found in NCHRP Report 299 (Moses, 1987). The design life corresponds to the lower-bound expected life (95% confidence interval) of the detail for the given stress range. The lower-bound value is set to be approximately two standard deviations below the mean fatigue strength and corresponds to approximately 97.5% survival. The mean life corresponds to the average expected life (50% survival) of the detail for the given stress range.

The fatigue lives of the details are compared to their respective design and mean expected lives in Table 5.2. The results in this table are discussed hereafter along with a description of the results for each of the specimens in the test program.

5.1.1. VS Series

The thin steel straps used to provide lateral constraint to the top flange of the VS and HS specimens were chosen because their flat profile meant they would theoretically deflect downward without relieving much of the vertical load applied to the specimen. It is therefore assumed that the calculation of the nominal stress ranges for fatigue details should not take these straps into account. To evaluate this assumption, strain data collected from a static loading of Specimen VS-1 without straps attached are compared to strain data collected while the straps were attached – see Table 5.3. As can be observed, there is no appreciable difference and this assumption is confirmed.

The fatigue cracking observed in the VS specimens occurred in a manner similar to previous large-scale tests of vertical stiffener details (see Section 2.2.1). Cracks initiated from the weld toe, the point of the highest stress concentration, and propagated in a semi-elliptical shape downward into the flange. Several small cracks eventually coalesced into one long crack extending the entire length of the weld toe. This long, shallow crack continued propagating downward through the flange thickness. The fatigue life for VS type specimen details is defined herein as the number of cycles required to propagate cracking through the thickness of the flange. This occurs before the actual fracture of the shape but was defined as failure so that a detail with this condition could be retrofitted and the test resumed to obtain additional meaningful information.

The results from the VS series are plotted in Figure 5.1 along with experimental results from other large-scale vertical stiffener tests. Section 2.2.1 of the literature review contains summaries of these tests. Gray arrows indicate a runout, the condition in which a detail does not fail within the testing period. Under the fatigue provisions outlined in the AASHTO LRFD Bridge Design Specifications, the base metal at the toe of a

transverse stiffener weld is classified as a category C' detail. As a basis of comparison, the design and mean life curves for AASHTO Category C' details are plotted as solid and dashed lines, respectively, and the constant amplitude fatigue limit (CAFL) is represented by a dotted line. The fatigue lives of specimens with no-gap (NG) details are plotted in Figure 5.2 along with the results from testing performed at Kansai University (Sakano 1992) on intersecting weld details. The data shown in these figures indicates that the no-gap condition does not produce a fatigue strength that is any less than the fatigue strength of conventional welded vertical stiffeners. Vertical stiffeners with both gap and no-gap conditions between weld toes are reasonably represented by the Category C' detail.

5.1.1.1. Specimen VS-1

Though all three beams in the VS series underwent local warping as a result of the welding process, only Specimens VS-2 and VS-3 were retrofitted to alleviate the problem. The large discrepancy between the strains recorded in the bottom flange on either side of the web of Specimen VS-1 suggested that, while the compression flange was laterally braced, the tension flange was subjected to some level of bending about the weak axis. The fact that cracking occurred towards the edge of the flange for both stiffeners indicated that there may be a stress gradient across the tension flange, with higher tensile stresses on the side welded to the detail.

If the simplifying assumption is made that there was a linear variation in the tensile stress across the bottom flange caused by secondary bending (with the mean stress occurring at the centerline of the section), calculations using the recorded strains indicate that the stress at the weld toe would have been 1.5 ksi (14%) higher at the outside of the flange than that at the weld intersect.

For a stress range of 16.6 ksi (the average nominal stress range at the stiffener-to-flange weld toes), the design and mean life expectancies of a Category C' detail are 962,000 and 2,077,000 cycles, respectively. There is some variation in the actual nominal stress along the weld toe for a given location (and, therefore, the life expectancies) as seen in Table

5.2. The lives of the stiffener details for Specimen VS-1 were 4,726,000 and 5,840,000 cycles, both well above the mean life expectancy.

5.1.1.2. Specimen VS-2

The retrofit made to specimen VS-2 to alleviate local warping of the section was effective in creating more balanced strains on the bottom of the tension flange (see Table 4.3). It can be assumed that this resulted in a more uniform stress along the area of interest for the detail: the toe of the stiffener-to-flange weld. When this is achieved, there is less of a chance that a certain length of the weld toe would experience a higher stress range and therefore tend towards earlier fatigue crack initiation and propagation. The retrofit to alleviate out-of-plane warping was not in place during the entire length of the test but the 60,000 cycles of loading before it was installed most likely did not have an adverse effect on the fatigue life.

For a stress range of 17.6 ksi (the average nominal stress range at the stiffener-to-flange weld toes), the design and mean life expectancies of a Category C' detail are 807,000 and 1,743,000 cycles, respectively. There is some variation in the actual nominal stress along the weld toe for a given location (and, therefore, the life expectancies) as seen in Table 5.2. The lives of the two stiffeners were 1,690,000 and 2,300,000 cycles, both near the mean life expectancy of the detail.

5.1.1.3. Specimen VS-3

As was the case with Specimen VS- 2, the retrofit made to alleviate local warping of the section was effective. The strains recorded on the bottom of the tension flange were more uniform along the transverse direction (see Table 4.4).

For a stress range of 17.6 ksi (the average nominal stress range at the stiffener-to-flange weld toes), the design and mean life expectancies of a Category C' detail are 807,000 and 1,743,000 cycles, respectively. There is some variation in the actual nominal stress along the weld toe for a given location (and, therefore, the life expectancies) as seen in Table

5.2. Only one stiffener detail on Specimen VS-3 reached the failure criterion, at 2,170,000 cycles. The other stiffener detail had no visible cracking at the conclusion of the test and therefore had a fatigue life of at least 4,050,000 cycles. Both of these test results exceed the mean life expectancy of the detail.

5.1.2. HS Series

Although the gages on the bottom flanges of Specimens HS-1 and HS-2 showed that the specimens experienced the same out-of-plane effects as the VS specimens, the straightening procedure shown in Figure 4.10 was not applied to either. Unlike the case with VS specimens, the resulting stress gradient did not result in a bias towards earlier fatigue cracking for one detail of interest over another.

The fatigue cracking observed in the HS specimens occurred in a manner similar to previous large-scale tests of horizontal attachment details (see Section 2.2.3). The critical crack initiated from the toe of the termination of the horizontal stiffener-to-web weld, the point of the highest stress concentration, and propagated in a semi-elliptical shape through the web thickness in a plane normal to the primary tensile stress.

The fatigue life for the HS type specimen details is defined herein as the number of cycles required to propagate cracking through the entire thickness of the web. At this point, the web cracking is roughly 1.25 in. in length. Clearly, some additional cycles would be required to propagate the crack into the flange and cause complete fracture of the specimen. The failure criterion, however, is merely an arbitrary definition to allow comparison to other tests and to allow the testing of several details on one specimen without fracturing it. It should be noted that this is the same failure criterion used in the large-scale experimental tests on welded web attachment details cited in the literature review (see Sections 2.2.2 and 2.2.3).

The results from the HS series fatigue tests are plotted in Figure 5.3 along with experimental results from other large-scale horizontal stiffener tests. Section 2.2.2 of the

literature review contains summaries of these tests. It should be noted that none of these previous tests had vertical welded stiffeners near the horizontal stiffeners. Under the fatigue provisions outlined in the AASHTO LRFD Bridge Design Specifications (2007), the base metal at the toe of a weld connecting a longitudinal attachment of a thickness less than 1.0 in. is classified as a category E detail. As a basis of comparison, the design and mean life curves for AASHTO Category E details are plotted in Figure 5.3 as solid and dashed lines, respectively. As can be observed, the HS test data compare very favorably with the Category E mean fatigue strength.

For an applied nominal stress range of 12.5 ksi (the average stress range at midheight of the horizontal stiffener at the details of interest on the HS specimens) the design and mean life expectancies of a Category E detail are 563,000 cycles and 901,000 cycles, respectively. There is some variation in the actual nominal stress at the weld toe for a given location (and, therefore, the life expectancies) as seen in Table 5.2.

5.1.2.1. Specimen HS-1

The fatigue lives of the horizontal stiffener details on Specimen HS-1 were 675,000 cycles for the small web gap and 950,000 cycles for the large web gap. Both details lasted longer than the average design life (563,000 cycles) but it should be noted that the small gap detail failed before the large gap detail. Though ‘failure’ also occurred in the large web gap at Location A, at 1,710,000 cycles, the area was likely affected by the stress distribution following the retrofit of Location B. Because of this, it is unlikely that Location A experienced a constant 12.5 ksi stress range over its lifetime.

The cracking that was discovered along the vertical weld toe for the vertical stiffener next to the lower retrofit hole at Location C at 2,040,000 cycles was unexpected. A transverse attachment weld toe is typically classified as a Category C’ detail, which has a design life of 1,439,000 cycles for a stress range of 14.5 ksi, the nominal value calculated for this location. Even though the cracking occurred later than the design life, it occurred before any evidence of cracking was seen along the stiffener-to-flange weld toe, an area

subjected to a higher stress range. This suggests the possibility that the stress redistribution resulting from the retrofit holes caused a very high stress concentration at this weld toe, leading to the earlier-than-anticipated cracking. This effect of a stop hole retrofit on nearby welds should be considered when predicting the safe service life of a steel bridge retrofit.

5.1.2.2. Specimen HS-2

The fatigue lives of the horizontal stiffener details on Specimen HS-2 were 950,000 cycles for the small web gap and 1,200,000 cycles for the large web gap. As was the case with Specimen HS-1, these both lasted longer than the design life but it should be noted that the small gap detail failed before the large gap detail.

5.1.3. HG Series

The test setup for the HG series was chosen to mimic common lateral bracing found in several Indiana bridges. Connectivity of the two beams via the cross bracing was expected to induce some out-of-plane forces due to differential displacement between the loaded test specimen and the secondary beam. The setup was somewhat similar to the laboratory test setups of NCHRP Report 227 (Fisher et. al., 1980) and University of Alberta (Comeau and Kulak, 1979) laboratory test setups where lateral bracing was attached to the gusset but large lateral forces were not intentionally directly applied. There was a concern that the horizontal bracing connected to the vertical stiffener and the lateral bracing connected to the horizontal gusset could carry some of the load from the test specimen, resulting in a reduced stress range at the details of interest. To assess this condition, strain gages attached to the secondary beam (at locations comparable to those of the primary test beam as shown Figure 4.29) were monitored during a static loading of specimen HG-1. These strains, shown in Table 5.4, clearly indicate that there is no large vertical force transfer from specimen HG-1 to the secondary beam through the horizontal bracing connected to the vertical stiffener at midspan.

The fatigue cracking observed in the HG specimens occurred in a manner similar to previous large-scale tests of horizontal gusset details (see Section 2.2.3). The critical crack initiated from the toe of the outer termination of the horizontal gusset-to-web weld, the point of the highest stress concentration, and propagated in a semi-elliptical shape across the web in a plane normal to the primary tensile stress. The fatigue life for HG specimen details is defined herein as the number of cycles required to propagate cracking through the thickness of the web – the same definition applied to HS specimens.

The results of this test series are plotted in Figure 5.4 along with experimental results from other large-scale horizontal gusset tests. (Section 2.2.3 of the literature review contains summaries of these other tests.) The S-N curve for a Category E detail is also shown for comparison since, under the fatigue provisions outlined in the AASHTO LRFD Bridge Design Specifications (2007), the base metal at the toe of a weld connecting a longitudinal attachment of a thickness less than 1.0 in. is classified as a Category E detail. The design and mean life curves for AASHTO Category E details are plotted as solid and dashed lines, respectively.

For an applied nominal stress range of 10.7 ksi (the stress range at midheight of the horizontal gusset in all of the HG specimens) the design and mean expectancies of a Category E detail are 898,000 cycles and 1,388,000 cycles, respectively. As can be observed in Figure 5.4, all of the test results for this study exceeded the mean life of the Category E detail prior to failure.

5.1.3.1. Specimen HG-1

All locations of interest on Specimen HG-1 had fatigue lives in excess of 3,363,000 cycles (well beyond the mean life expectancy of 1,388,000 cycles) but it should be noted that Location A was likely very close to the criterion for failure (cracking across the entire web thickness) at the time the test was stopped.

5.1.3.2. Specimen HG-2

As was the case with Specimen HG-1, Specimen HG-2 experienced cracking at the outside ends of the gusset plate but not in the web gap. Once cracking occurs at the outside edge of the gusset, the stress flow no longer resembles that seen in Figure 5.5. The tensile stress in the web must move around the crack and therefore cannot flow into the gusset as close to the outside edge as before. The resulting stress flow most likely resembles that shown in Figure 5.6. In this case, the actual stress range applied in the web gap likely decreases even though the area is assumed to be subjected to a constant nominal stress range over the life of the test.

The end weld detail at Locations A and D was more critical for specimen HG-2 (Figure 4.32) than HG-1 (Figure 4.28), as the wraparound weld geometry likely resulted in a higher stress concentration. Still, both locations developed fatigue lives in excess of the predicted fatigue mean life of 1,388,000 cycles. Location D failed at 1,650,000 cycles, while Location A failed at 2,875,000 cycles.

Figure 5.7 shows the strains recorded on the web of specimen HG-2 over time. The locations and orientations of the strain gages are shown in Figure 4.30. As can be observed, the web strain values near the vertical stiffener did not change significantly, even after cracking developed at the end of the gusset plate.

5.1.3.3. Specimen HG-3

Specimen HG-3 was fabricated from a specimen that was initially intended to be a VS specimen so, unlike the other HG specimens, it had two vertical stiffeners welded to the web opposite the horizontal gusset (Figure 5.8). Since these are located away from the main details of interest in the web gap, they were assumed to have little effect on the stress range there.

The specimen was loaded in the same manner as specimens HG-1 and HG-2 with one exception: only the six bolts on the north side of the horizontal gusset were fully

pretensioned while the other six were merely finger-tightened. This was done intentionally to see if asymmetric out-of-plane loading was generated by variation in brace connection bolt tightness and if this resulted in a detrimental effect on the fatigue life of the specimen.

However, this change did not produce a high enough stress concentration in either web gap to generate cracking there during testing. Locations A and D experienced ‘failure’ at 1,825,000 cycles and 2,050,000 cycles, respectively. Both locations exceeded the calculated mean life of 1,338,000 cycles.

The web strains recorded over time for Specimen HG-3 are shown in Figure 5.9. They provide insight into how the stress distribution along the web changes over the length of the test. The cracking and subsequent retrofit at Location A caused stress redistribution in the web as evidenced by the sharp drop in the strain range experienced by gages W1 and W6. These gages are both nearly equidistant from the edge of the gusset and the web gap (Figure 4.30). Gage W3, one of the gages closer to the web gap, experienced a smaller – but still noticeable – drop in measured strain (from 285 $\mu\epsilon$ in the uncracked specimen to 265 $\mu\epsilon$ after the retrofit).

Therefore, it is again assumed that the retrofits at the outside edges of the gusset affected the stress ranges in the web gap to some extent. The stress redistribution in Figure 5.6 is consistent with the strain readings in Figure 5.9. The strain ranges at the gages behind the gusset decrease as less force is transferred into it. Clearly, the gages closest to the retrofit hole are influenced more than those in the middle since more force is diverted there as a result of the crack and the retrofit hole.

5.1.3.4. Specimen HG-4

Specimen HG-4 was a specimen with a gusset cope large enough to leave a “small gap” of 0.1875 in. between the perpendicular weld toes. It had the same loading condition as

specimen HG-3 where only the bracing on one side of the gusset was pretensioned in an attempt to create an asymmetric out-of-plane loading condition.

Location D experienced ‘failure’ at 3,425,000 cycles, far beyond the calculated mean life of 1,338,000 cycles. Though cracking was first seen at Location A shortly after it was seen at Location D, the cracking there propagated more slowly and never made it completely across the web thickness during the test.

For the first time with an HG specimen, cracking was observed in the web gap. Cracking initiated along the vertical weld toe at Location C and propagated across the entire web thickness at 5,250,000 cycles. This indicates that the stress redistribution after the drilled hole retrofits likely leads to higher stresses at the vertical weld toe (typically a Category C’ detail) than the horizontal weld termination (a Category E detail). Under the nominal stress range of 10.7 ksi, the design and mean lives for a Category C’ detail are 3,592,000 cycles and 7,755,000 cycles, respectively.

The web strains recorded over time for Specimen HG-4 are shown in Figure 5.10. They provide insight into how the stress distribution along the web changes over the life of the test. The retrofit at Location D caused a notable immediate drop in the strain ranges at gage W3 (276 $\mu\epsilon$ to 266 $\mu\epsilon$) and W1 (237 $\mu\epsilon$ to 209 $\mu\epsilon$). As cracking propagated across the web at Location C, the strain ranges near the web gap (gages W3 and W4) began to gradually decrease. It is assumed that the primary tensile stresses had to flow around the cracked section, thereby reducing the stress ranges in the web gaps.

5.2. Experimental Behavior of Drilled Hole Retrofits

Drilled holes are one of the most commonly used methods to retrofit fatigue cracking in steel girder webs. In this study, drilled hole retrofits were applied to cracked details in the HS and HG series both to test the effectiveness of the hole drilling procedure and to enable testing to continue without fracture so that additional data could be collected for

each specimen. In this section, the fatigue behavior of these drilled hole retrofits is discussed.

Though retrofit holes remove a section of the web, they are effective in that they eliminate the critical zero-radius condition of the crack tip. The larger the retrofit hole radius, the lower the stress concentration at the edge of the hole. In NCHRP Report 227, Fisher et. al. (1980) outline a procedure to find the appropriate size retrofit hole to prevent fatigue crack reinitiation for a given applied stress range and crack length. The procedure involves calculating the applied stress intensity factor range (ΔK) at the detail then selecting a hole radius (ρ) such that the following relationship is satisfied:

$$\frac{\Delta K}{\sqrt{\rho}} \leq 4\sqrt{\sigma_y}$$

where σ_y is the yield strength of the cracked steel. This criterion was obtained by plotting $\Delta K/\sqrt{\rho}$ vs. cycles to reinitiation for the empirical data obtained in the study and setting $4\sqrt{\sigma_y}$ as a $\Delta K/\sqrt{\rho}$ value below which re-initiation is unlikely to occur.

5.2.1. Stress Intensity Factors

For the retrofits in this study, the following procedure is used to calculate the stress intensity factor range. First, the stress range at the midheight of the welded horizontal plate (stiffener or gusset) is used to calculate the mean stress intensity factor range (ΔK_1) using the equation for a center-cracked plate subjected to uniform tension (Figure 5.11):

$$\Delta K_1 = \sqrt{\sec \frac{\pi a}{2b}} \Delta \sigma_1 \sqrt{\pi a}$$

Since the retrofit spans a relatively large depth of the W24x55 web, the effects of bending and the resulting non-uniform tensile stresses are considered (Figure 5.12) by adding a correction factor (ΔK_2). The formula for this factor is derived from the following

equation from Rooke (1993), which applies to the case of a center-cracked plate subjected to pure bending (M , moment per unit thickness) (Figure 5.13):

$$K_0 = \frac{3\sqrt{\pi M a^{3/2}}}{4b^3}$$

This equation can be reworked into the following expression, a function of the stress at the extreme fiber of a center-cracked plate subjected to pure bending, corresponding to the case seen in Figure 5.14:

$$\Delta K_2 = \sqrt{\sec \frac{\pi a}{2b} \frac{\Delta \sigma_2 \sqrt{\pi a^{3/2}}}{2b}}$$

If the change in stress between the mid-level (midheight of the horizontal plate) and lowest level (bottom of the tension flange) is input into this equation ($\Delta \sigma_2$), the stress intensity factor range correction (ΔK_2) can be calculated. This value can be added to the mean stress intensity factor range (ΔK_1) to get a more accurate value for the bottom retrofit hole. Table 5.5 summarizes the calculated values for the retrofit holes used in this study.

5.2.2. Retrofit Hole Lives

The fatigue performance of the retrofits implemented in this study are summarized in Table 5.6 and plotted in Figure 5.15. For each location retrofitted (HS-1-C corresponds to Location C of Specimen HS-1, etc.), the number of cycles at the time of the retrofit is plotted and connected with a line to the number of cycles applied when cracking initiated at the lower retrofit hole. Details where crack re-initiation did not occur are plotted with a tick mark (denoting when the test was stopped) followed by an arrow.

All of the test specimens were ASTM A572 Gr. 50 steel so the $4\sqrt{\sigma_y}$ value from the repair hole crack re-initiation criteria suggested by Fisher et al. (1980) is calculated to be 28.3 ksi. The $\Delta K/\sqrt{\rho}$ values calculated for the HS and HG specimens are shown in

Table 5.5. The geometry of these specimens made it impossible to satisfy the $4\sqrt{\sigma_y}$ limitation to prevent crack re-initiation. The magnitude of the stress range in conjunction with the short distance (3.0 in.) between the welded horizontal plate and the bottom flange was too small to accommodate a large enough hole.

The calculated $\Delta K/\sqrt{\rho}$ value from the retrofits is plotted together with their resulting life extension in Figure 5.16. The value $4\sqrt{\sigma_y}$ limitation for crack re-initiation is plotted as a horizontal line for reference. Arrows indicate a runout, meaning no cracking re-initiated in the retrofit hole during the length of the test.

The best approach is to install a retrofit hole that is large enough to prevent crack re-initiation. However, as noted above, this is not always possible. Nevertheless, a significant improvement in the fatigue life can be achieved even if the $4\sqrt{\sigma_y}$ limitation is not satisfied. For the tests in this study it was observed that the range in additional number of loading cycles was 36% to 120% of the number of loading cycles applied prior to initial cracking, with an average value of 62.5% additional loading cycles.

All holes were drilled with Hougan hollow-core bits. One common hole diameter (1.625 in.) was used for all HS specimens while another (1.5 in.) was used for all HG specimens. These hole diameters corresponded to the largest drill bits that could be accommodated by the magnetic drills used for the HS and HG specimens (shown in Figure 4.22 and Figure 4.34, respectively).

It should be noted that the two retrofit holes drilled at Location D on Specimen HG-4 lasted much longer than any of the others. This is true despite the fact that, as noted in Table 5.5, the calculated $\Delta K/\sqrt{\rho}$ value at this location is greater than the values for the other HG specimens. While all holes were filed to remove any steel burrs and slivers leftover from the drilling process, these two holes were extensively filed by hand after the

use of an old bit left a rough surface appearance around the edge of the holes. This could potentially be the reason behind the relative success of these holes.

5.3 Bolted Splice Retrofit

In addition to using drilled holes to retrofit a detail that experiences fatigue cracking, another option may be the use of a bolted web splice. The bolted splice retrofit is not always possible due to the presence of gusset plates or stiffeners on the girder. However, if no such plates exist on one side of the girder, then the splice plate can be installed. This retrofit is very effective because it can be used to transfer stress away from the cracked region altogether. The web splice can be sized by using a plate section (or multiple plate sections) comparable to the web plate at a minimum. High-strength bolts are used on each side of the cracked region to transfer stress from the web into the splice plate. To further reduce the likelihood of any additional fatigue crack growth, stop holes can be drilled at the crack tips prior to installing the splice plate. This approach is shown in Figure 4.35 for one of the horizontal gusset (HG type) specimens.

5.4. Summary Comments

5.4.1. VS Series

All of the vertical stiffener details tested in this series lasted well beyond the design fatigue life and, with one exception (Location B on Specimen VS-2), lasted beyond the mean expected life. Furthermore, all of the critical cracking occurred away from the weld intersection. Lastly, when the data from details with large web gaps are compared to the data from details with intersecting welds, no clear difference can be seen within the scope of this testing. Hence, intersecting welds were not found to decrease the fatigue life significantly.

5.4.2. HS Series

Although all of the details in this series carried more loading cycles than the design life for a Category E detail, it is notable that small web gap details had shorter fatigue lives than the details with large web gaps. With such a small number of data collected, this difference in behavior could be due to the inherent scatter associated with fatigue testing. However, it could mean that a smaller web gap results in a more critical condition than a large gap. Previous studies (Fisher et. al., 1974) have shown that the stress concentration occurring at a welded connection decreases quickly in the base metal with distance away from the weld toe. However, if two weld toes are close enough together, the stress concentrations at the weld toes may be additive to a certain extent. This would theoretically result in a more critical stress concentration and, therefore, a shorter fatigue life. A fracture-sensitive constraint effect is also possible for weld toes that are situated very close together. Examining this relationship further would require more extensive analytical modeling and further testing. Until such information is available, it may be advisable to not permit web gaps for horizontal attachments to be less than 1/4-in. The web gap distance between weld toes can be increased by cutting the attachment back and grinding the weld toe smooth. Adding a radius often results in a further increase in the fatigue strength.

5.4.3. HG Series

In nearly all of the previous large-scale experimental fatigue tests of welded horizontal gussets coped to fit around a vertical stiffener (see Section 2.2.2), fatigue cracking occurred at the outside edges of the gusset with little evidence of cracking in the web gap. Under in-plane bending alone, the stress distribution for this geometry tends to resemble that in Figure 5.5. The gusset acts as a secondary flange, relieving some of the tensile stress carried by the girder web. If the gusset-to-web welds are long enough, stress can develop far enough outward into the gusset that the area outside of the cope is in tension. If this is the case, then the stress concentration at the horizontal weld termination in the web gap must be lower than that at the outside edge of the gusset because less load is transferred there. The finite element model of a horizontal gusset generated in NCHRP

Report 227 (Fisher et. al., 1980) shows the same conclusion: that the stresses at the outside edges of the gusset are higher than those in the web gap.

The exception to this behavior is explored in NCHRP Report 336 (Fisher, 1990). In the experimental testing performed therein, very large out-of-plane forces were intentionally induced into the specimen web through the vertical stiffener. Since these forces had to be accommodated in a short length of unstiffened web, very large out-of-plane stresses occurred in the web gap. As a result, fatigue cracking first occurred at the toe of the vertical stiffener-to-web weld at the level of the horizontal gusset.

For the experimental tests conducted in this study, Specimens HG-2 and HG-3 had a condition where the gusset-to-web and stiffener-to-web welds directly intersected. The vertical stiffener-to-web welds were placed first and then the horizontal gusset-to-web welds were started at this point, leading to the overlap seen in Figure 4.31. In general, a lack of fusion discontinuity could potentially occur at this location if the slag is not completely removed from the stiffener weld. Also, residual stresses are developed along the horizontal weld as a result of contraction due to cooling of the weldment, placing the region in tension due to residual weld shrinkage stresses. Such a case would result in a pre-existing crack-like condition in a plane normal to the primary bending stress. This critical condition could lead to fatigue behavior similar to that seen in the Third Street Viaduct case study (Demers, 1990). However, the intersection of the welds in the web gap did not result in a condition critical enough to cause cracking there before cracking occurred at the outside edges. Nevertheless, it is advisable to maintain a gap of at least 1/4-in to prevent constraint-induced brittle fracture.

5.4.4. Retrofit Holes

The drilled hole retrofits had varying degrees of success in extending the lives of cracked details. Although the criterion to prevent hole re-initiation suggested by Fisher et. al. (1980) was not satisfied, a few of the retrofits performed well enough to double the

fatigue life of the detail before cracking reinitiated. On average, an additional 64% loading cycles could be applied after repair before crack re-initiation occurred.

The effect that drilled hole retrofits have on nearby fatigue-prone details should be considered. The position of the retrofit hole to adjacent welds should also be considered. If the hole is situated too close to an adjacent vertical weld, the resulting stress redistribution can cause high stress concentrations at nearby weld toes, initiating fatigue cracking that would otherwise not occur.

The diameter necessary to satisfy the $4\sqrt{\sigma_y}$ limitation for preventing crack re-initiation is computed to illustrate the size requirements for retrofit holes. If it is assumed for simplicity that the effective stress range at the level of a horizontal gusset plate is constant, and the variation in stress over the girder depth is ignored, and it is further assumed that there is no correction in the stress-intensity factor for finite width, then a simple calculation can be made to determine the required retrofit hole diameter. The diameter required for several different fatigue crack lengths that are centered about either side of a horizontal gusset plate are shown in Table 5.7 for several different values of the effective stress range. Retrofit hole diameter values are computed for steels with a yield strength 36 ksi and 50 ksi (Grades 36 and 50 steel). As can be observed in Table, as either the crack length at repair or the effective stress range at the gusset detail increases, the hole diameter required to prevent crack re-initiation increases. In fact, a point is reached where the required hole diameter increases rapidly. It should be noted, however, that if the required hole diameter becomes too large, it may be more effective to look at other repair options for the structural detail in question, such as a bolted splice connection.

Table 5.1 Constants used for calculation of the design and mean fatigue lives of a detail

Detail Category	A_D (10^8 ksi^3)	A_M (10^8 ksi^3)
A	250	720
B	120	240
B'	61	120
C	44	95
D	22	44
E	11	17
E'	3.9	7.5

Table 5.2 Experimental Results

Specimen	Location	Stress range (ksi)	Cycles to cracking (10^3 cycles)	Cycles to failure (10^3 cycles)	AASHTO Fatigue Category	N_D (10^3 cycles)	N_M (10^3 cycles)
VS-1	A	16.5	--	--	C'	979	2,116
	B	16.7	5,550	5,850	C'	945	2,041
	C	16.7	3,530	--	C'	945	2,041
	D	16.5	2,860	4,726	C'	979	2,116
VS-2	A	17.5	--	--	C'	821	1,773
	B	17.7	1,250	1,690	C'	793	1,714
	C	17.7	1,550	--	C'	793	1,714
	D	17.5	1,380	2,300	C'	821	1,773
VS-3	A	17.5	--	--	C'	821	1,773
	B	17.7	--	--	C'	793	1,714
	C	17.7	1,923	2,170	C'	793	1,714
	D	17.5	--	--	C'	821	1,773
HS-1	A	12.3	760	1,710	E	591	946
	B	12.8	505	950	E	525	839
	C	12.6	355	675	E	550	880
	D	12.4	--	--	E	577	923
HS-2	A	12.3	--	--	E	591	946
	B	12.8	835	1,200	E	525	839
	C	12.6	--	--	E	550	880
	D	12.4	600	950	E	577	923
HG-1	A	10.7	2,080	--	E	898	1,388
	B	10.7	--	--	C' / E	898	1,388
	C	10.7	--	--	C' / E	898	1,388
	D	10.7	--	--	E	898	1,388
HG-2	A	10.7	1,740	2,875	E	898	1,388
	B	10.7	--	--	C' / E	898	1,388
	C	10.7	--	--	C' / E	898	1,388
	D	10.7	875	1,650	E	898	1,388
HG-3	A	10.7	650	1,825	E	898	1,388
	B	10.7	--	--	C' / E	898	1,388
	C	10.7	--	--	C' / E	898	1,388
	D	10.7	955	2,050	E	898	1,388
HG-4	A	10.7	2,550	--	E	898	1,388
	B	10.7	--	--	C' / E	898	1,388
	C	10.7	4,125	5,250	C' / E	898	1,388
	D	10.7	1,000	3,350	E	898	1,388

Table 5.3 Comparison of strains with and without straps attached

	Load (kip)	Reading at strain gage ($\mu\epsilon$)			
		SE	SW	NE	NW
without straps ¹	47.5	563	498	580	503
with straps ²	47.57	558	504	579	508

¹ - values taken from a static loading of specimen VS-1 from 2.5 kips to 50 kips

² - average of all values collected during the testing of specimen VS-1

Table 5.4 Strains on the secondary beam after a 50 kip static loading of Specimen HG-1

Load (kip)	Reading at strain gage ($\mu\epsilon$)							
	TF-NE	TF-NW	TF-SE	TF-SW	BF-SE	BF-SW	BF-NE	BF-NW
50	6	-8	4	-6	1	5	3	4

Table 5.5 Summary of drilled hole retrofit calculated values

Specimen	Location	Stress range ¹ (ksi)	Retrofit hole diameter (in)	Equivalent crack length, a (in)	Equivalent plate width, b (in)	ΔK_1 (ksi $\sqrt{\text{in}}$)	ΔK_2 (ksi $\sqrt{\text{in}}$)	ΔK (ksi $\sqrt{\text{in}}$)	$\Delta K/\sqrt{p}$ (ksi)
HS-1	C	12.5	1.625	2.31	3.69	45.3	6.5	51.7	57.4
	B	12.5	1.625	2.375	3.69	46.8	6.9	53.7	59.6
HS-2	D	12.5	1.625	2.25	3.69	43.8	6.1	49.9	55.3
	B	12.5	1.625	2.125	3.69	41.1	5.4	46.5	51.5
HG-3	A	10.7	1.5	1.78	3.69	29.7	3.3	33.0	38.1
	D	10.7	1.5	1.81	3.69	30.2	3.4	33.5	38.7
HG-4	D	10.7	1.5	1.96	3.69	32.5	3.9	36.5	42.1

¹ - at the mean height of the horizontal stiffener or gusset

Table 5.6 Cyclic life results for drilled hole retrofits

Specimen	Location	$\Delta K/\sqrt{\rho}$ (ksi)	Cycles to retrofit (10^3 cycles)	Cycles to retrofit failure ¹ (10^3 cycles)	Retrofit Effectiveness ¹ (10^3 cycles)
HS-1	C	57.4	675	1,060	385
	B	59.6	950	2,087	1,137
HS-2	D	55.3	950	1,640	690
	B	51.5	1,200	1,640	440
HG-3	A	38.1	2,050	2,800	750
	D	38.7	2,150	3,000	850
HG-4	D	42.1	3,425	6,000	2,575

¹ - **bold** values indicate that the test was ended before cracking reinitiated in the retrofit hole

Table 5.7 Required diameter for drilled hole retrofits (inches)

Yield Strength (ksi)	Crack Length (in)	Effective Stress Range (ksi)				
		5.0	6.0	7.0	8.0	9.0
36	1.0	< 0.50	0.50	0.625	1.00	3.75
	1.5	< 0.50	0.50	0.875	1.75	5.75
	2.0	< 0.50	0.625	1.125	2.375	7.50
	2.5	0.50	0.875	1.50	2.875	9.50
	3.0	0.50	1.00	1.75	3.50	11.50
	3.5	0.625	1.125	2.00	4.00	13.25
		5.0	6.0	7.0	8.0	9.0
50	1.0	< 0.50	< 0.50	< 0.50	0.50	0.875
	1.5	< 0.50	< 0.50	0.50	0.75	1.375
	2.0	< 0.50	0.50	0.625	1.00	1.75
	2.5	< 0.50	0.50	0.75	1.25	2.25
	3.0	0.50	0.625	1.00	1.50	2.625
	3.5	0.50	0.625	1.125	1.75	3.00

Figure 5.1 VS series test results

Figure 5.2 VS series test results (NG details only)

Figure 5.3 HS series test results

Figure 5.4 Test results for HG series

Figure 5.5 Illustration of stress flow in HG specimens prior to cracking

Figure 5.6 Stress flow in the gusset after cracking at the edges

Figure 5.7 Web strains over time for specimen HG-2

Figure 5.8 Plan view of additional stiffeners on specimen HG-3

Figure 5.9 Web strains over time for specimen HG-3

Figure 5.10 Web strains over time for specimen HG-4

Figure 5.11 Condition for a center-cracked plate subjected to a uniform tension (Rooke, 1993)

Figure 5.12 Loading of a retrofitted HG detail

Figure 5.13 Condition for a center-cracked plate subjected to pure bending (Rooke, 1993)

Figure 5.14 Condition for a center-cracked plate subjected to a linearly varying stress (pure bending)

Figure 5.15 Increase in fatigue life of details by drilled hole retrofits

Figure 5.16 $\Delta K/\sqrt{\rho}$ vs. fatigue life increase

CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

6.1. Summary and Conclusions

As a follow-up to the Hoan Bridge brittle fracture in Milwaukee, Wisconsin in December 2000, all state Departments of Transportation examined the steel bridges in their inventory that had intersecting welds and that were possibly susceptible to constraint-induced brittle fracture. A total of four different bridge details in Indiana were identified that contained intersecting or nearly-intersecting welds. While the Hoan Bridge fracture was a constraint-induced brittle fracture, the purpose of this study was to examine the fatigue resistance of steel bridge details used in Indiana that contain intersecting or nearly intersecting welds.

Nine steel beams containing details with varying degrees of weld intersection were tested under constant amplitude cyclic loading. Three different basic detail types were tested: a vertical stiffener welded to the web and flange, a welded horizontal stiffener terminating near a welded vertical stiffener, and a welded horizontal gusset plate coped to fit around a welded vertical connection plate. The fatigue strength of the details with intersecting welds was compared to that of details without intersecting welds. For each detail type, there were several conditions tested with varying gaps between perpendicular welds. The results of the tests were examined to determine if the gap size had an effect on fatigue behavior, and more importantly, to determine if this effect could result in a fatigue strength below the appropriate design fatigue strength. For some of the details, the effectiveness of drilled hole retrofits in extending the fatigue life of details that had developed fatigue cracking was also examined.

Based on the experimental results and accompanying evaluation of the details in this study, the following observations and conclusions can be made:

1. The presence of intersecting welds connecting a vertical stiffener to the flange and web of a rolled shape was shown to have no detrimental effect on the fatigue life of the detail. All of the details tested with intersecting welds lasted well beyond the design life for a Category C' detail under the calculated nominal stress range, and all of the critical cracking in these details occurred along the stiffener-to-flange weld toe away from the weld intersection. Furthermore, there was no trend of the details with large gaps having a more favorable fatigue resistance than those with intersecting welds.
2. The size of gap between the perpendicular welds where a horizontal stiffener terminates near a vertical stiffener may have an effect on the fatigue strength of the detail. Fatigue cracking at the toe of the horizontal stiffener-to-web weld termination occurred earlier in details with a small web gap (0.125 in.) than it did in details with a large gap (1.0 in.). If this gap is small enough, the stress concentration factors at the longitudinal and transverse weld toes that are directly opposite each other may be additive to a certain extent. This could result in a fatigue strength that is lower than the design strength for the horizontal stiffener alone.
3. Within the scope of this study, the size of the web gap did not have an effect on the fatigue behavior of the detail with a horizontal gusset plate coped to fit around a vertical connection plate. The details in this test behaved in the same manner as many of the previous experimental tests, with the most critical location for fatigue cracking at the outside edges of the gusset plate, not in the web gap.
4. Small web gaps may increase the risk of constraint-induced brittle fracture. Other risk factors include steels with low fracture toughness and the presence of very low temperatures. Although limited research is available, it is recommended that web gaps of less than 1/4-in between adjacent weld be avoided whenever possible.
5. Retrofit holes were effective in extending the fatigue life of cracked details by eliminating the critical condition of the crack tip. If possible, the diameter of the

retrofit hole should be large enough to satisfy the criteria in NCHRP Report 227 to prevent further crack growth.

6. The fatigue strength of drilled hole retrofits may depend in part on the surface condition of the hole at the location where cracking will likely initiate. If possible, care should be taken to grind or file this area to leave a smooth surface.
7. The stress redistribution caused by drilled hole retrofits may reduce the fatigue resistance of nearby details. For example, a retrofit hole likely placed near a welded vertical stiffener could potentially increase the stress concentration along the stiffener weld toe, leading to earlier than anticipated cracking.

6.2. Implementation Recommendations

Based upon the experimental results and evaluation conducted in this study, the following actions are recommended for implementation on steel highway bridge structures in Indiana.

1. No retrofit action is expected to be needed for welded stiffener or connection plate details where the vertical web welds intersect, or nearly intersect, with horizontal welds on the flange. No decrease in fatigue life was observed for such details.
2. Horizontal web attachment details with welds that are situated near a vertical connection plate or stiffener should be regularly inspected to determine if fatigue cracking has occurred at the end of the attachment plate weld toe that is situated next to the vertical plate.
3. Horizontal gusset plate details that are coped to fit around a vertical connection plate should be inspected to determine if fatigue cracking has occurred at the weld toe located at the outside ends of the gusset plate. The weld toe in the web gap region where the plate is coped should also be inspected to determine if fatigue cracking has occurred.
4. Fatigue cracks that are detected can typically be repaired using drilled retrofit holes. The retrofit hole diameter should be sized large enough to minimize re-

initiation of the fatigue cracking. If the hole diameter required is too large, then a bolted splice repair should be considered as an effective alternative.

5. Web gaps with multiple welds that intersect, or nearly intersect, should be modified to increase the web gap distance to ¼-in or larger between adjacent weld toes. This distance should be sufficient to minimize the likelihood of constraint-induced brittle fracture.

6.3. Future Research Needs

To provide more complete information on steel bridge details with intersecting or nearly-intersecting welds, two additional experimental studies are needed. These include tests with (a) horizontal attachments that are situated close to a vertical connection plate or stiffener, and (b) lateral gusset plates that are welded to the top flange of the girder and which nearly intersect the flange-to-web weld.

The experimental results from the HS series indicate that intersecting welds at the termination of a horizontal stiffener near a vertical stiffener consistently have a lower fatigue strength than a longitudinal weld termination alone. Further experimental testing and analytical modeling should be carried out to determine if this is the case and, more importantly, to determine if the intersecting weld condition could potentially result in cracking through the web earlier than the calculated AASHTO Category E fatigue design life. Additional retrofit options for this detail that should be studied include increasing the web gap region and adding a radius that provides for a smooth transition in stress from the horizontal attachment to the girder web.

Due to time and budget limitations, no experimental tests were conducted in the present study to evaluate the structural detail with a horizontal gusset plate is welded to the top of the bottom flange and which nearly intersects the flange-to-web weld. This condition was found to occur infrequently in Indiana, but more information on this detail would certainly be useful in assessing the fatigue response and remaining life of bridges with this detail.

BIBLIOGRAPHY

- American Association of State Highway and Transportation Officials. (1990). *Guide Specifications for Fatigue Evaluation of Steel Bridges*. Washington, D.C.
- American Association of State Highway and Transportation Officials. (2002). *Standard Specifications for Highway Bridges*. 17th Ed., Washington, D.C.
- American Association of State Highway and Transportation Officials. (2003). *Guide Manual for Condition, Evaluation, and Load and Resistance Factor Rating (LRFR) of Highway Bridges*. 1st Ed., Washington, D.C.
- American Association of State Highway and Transportation Officials. (2007). *LRFD Bridge Design Specifications*. 4th Ed., Washington, D.C.
- American Welding Society. (1994). *Structural Welding Code – Steel, ANSI/AWS D1.1-94*. 14th Ed., Miami, FL.
- Barth, A.S. and Bowman, M. D. (2002). “Fatigue Behavior of Beam Diaphragm Connections With Intermittent Fillet Welds, Part I, Volume 2, Laboratory Fatigue Evaluation.” *Report No. FHWA/IN/JTRP-2001/10-I-2*, Indiana Department of Transportation, Indianapolis, IN.
- Bowman, M. D. (2002). “Brittle Fracture Examination of the I-64 Blue River Bridge.” *Report No. FHWA/IN/JTRP-2001/10-II*, Indiana Department of Transportation, Indianapolis, IN.
- Chajes, M. J., Mertz, D. R., Roecker, H., and Milius, J. (2005). “Fracture Analysis and Retrofit Design for the I-95 Bridge Over the Brandywine River.” *ASCE Metropolis & Beyond 2005*, New York, NY.
- Comeau, M. P. and Kulak, G. L. (1979). “Fatigue Strength of Welded Steel Elements.” *Structural Engineering Report 79*, University of Alberta, Canada.
- Connor, Robert J., Kaufmann, Eric J., Fisher, John W., and Wright, William J. (2007). “Prevention and Mitigation Strategies to Address Recent Brittle Fractures in Steel Bridges,” *Journal of Bridge Engineering*, ASCE, Vol. 12, No. 2, pp 164-173.

- Demers, Cornelia E and Fisher, John W. (1990). "Fatigue Cracking of Steel Bridge Structures - Volume I: A Survey of Localized Cracking in Steel Bridges-1980 to 1988." *Report No. FHWA-RD-89-166*, Federal Highway Administration, Washington, D.C.
- Dittrich, William (2004). Indiana Department of Transportation, Private Communication.
- Fisher, J. W., Frank, K. H., Hirt, M. A., and McNamee, B. M. (1970). "Effect of Weldments on the Fatigue Strength of Steel Beams." National Cooperative Highway Research Program Report 102, Transportation Research Board, Washington, D.C.
- Fisher, J. W., Albrecht, P. A., Yen, B. T., Klingerman, D. J., and McNamee, B. M. (1974). "Fatigue Strength of Beams with Welded Stiffeners and Attachments." National Cooperative Highway Research Program Report 147, Transportation Research Board, Washington, D.C.
- Fisher, John W., Pense, Alan W., and Roberts, Richard. (1977). "Evaluation of Fracture of Lafayette Street Bridge." *Journal of the Structural Division*, ASCE, Vol. 103, No. 7, pp 1339-1357.
- Fisher, J. W., Barthelemy, B. M., Mertz, D. R., and Edinger, J. A. (1980). "Fatigue Behavior of Full-Scale Welded Bridge Attachments." National Cooperative Highway Research Program Report 227, Transportation Research Board, Washington, D.C.
- Fisher, J. W., Pense, A. W., Hausammann, H., and Irwin, G. R. (1980). "Quinnipiac River Bridge Cracking." *ASCE Journal of the Structural Division*, Vol. 106, No. 4, pp 773-789.
- Fisher, J. W., Jin, J., Wagner, D. C., and Yen, B. T. (1990). "Distortion-Induced Fatigue Cracking in Steel Bridges." National Cooperative Highway Research Program Report 336, Transportation Research Board, Washington, D.C.
- Fisher, J. W., Wright, W., and Sivakumar, B. (2001). "Hoan Bridge Forensic Investigation Failure Analysis Final Report." Federal Highway Administration and Wisconsin Department of Transportation, Madison, WI.
- Glinka, G. and Krzyzecki, A. (1981). "Fatigue Life Calculation and Testing of Full-Scale Welded I-Beams." *Fatigue of Engineering Materials and Structures*, Vol. 4, No. 1, pp. 35-48.
- Gurney, T. R. (1960). "Fatigue Strength of Beams with Stiffeners Welded to the Tension Flange." *British Welding Journal*, Vol. 7, Issue 9, pp. 569-576.

- Keating, P.B. and Fisher, J.W. (1986). "Evaluation of Fatigue Tests and Design Criteria on Welded Details." National Cooperative Highway Research Program Report 286, Transportation Research Board, Washington, D.C.
- Mahmoud, H. N., Connor, R. J., and Fisher, J. W. (2005). "Finite Element Investigation of the Fracture Potential of Highly Constrained Details in Steel Plate Members." *Computer-Aided Civil and Infrastructure Engineering*, No. 20, pp 383-392.
- Moses, F., Schilling, C. G., and Raju K. S. (1987). "Fatigue Evaluation Procedures for Steel Bridges." National Cooperative Highway Research Program Report 299, Transportation Research Board, Washington, D.C.
- Rooke, D.P. and Cartwright, D.J. (1976). *Compendium of Stress Intensity Factors*. The Hillington Press, Uxbridge, Middlesex.
- Ruge, J., Woesle, H. (1962). *Results of Fatigue Investigations*, abstract of Ergebnisse von Dauerfestigkeitsuntersuchungen published in Die Schweisstechnik im Zeichen Neuzeitlicher Verfahren und Werkstoffe. Deutscher Verband fur Schweisstechnik, pp. 59-68.
- Sakano, M., Mikami, I., Kano, K., Yonemoto, E., and Yasuike, H. (1992). "Fatigue Tests for Plate Girder Specimens With 4 Types of Vertical Stiffener Connections." *Technology Reports of Kansai University*, No. 34, pp 149-160.
- Sakano, M., Mikami, I., and Yonemoto, E. (1993). "Fatigue Life Predictions for Plate Girder Specimens With 4 Types of Vertical Stiffener Connections." *Technology Reports of Kansai University*, No. 35, pp 277-284.
- Spadea, J. R. and Frank, K. H. (2002). "Fatigue Strength of Fillet-Welded Transverse Stiffeners with Undercuts." *Report No. FHWA/TX-05/0-4178-1*
- Verma et. al. (2001). "Steel Bridge Fabrication Techniques in Europe and Japan." *Report No. FHWA-PL-01-018*, Federal Highway Administration, Washington, D.C.

Appendix A. MATERIAL PROPERTIES

This section contains the properties of the materials used in the experimental tests.

All welded attachments were ASTM A36 steel. The American Society for Testing and Materials (2005) requirements for this steel type are provided in Table A.1.

Detailed information about the W24x55 beams used in this test was reported in Appendix A of Report FHWA/IN/JTRP-2001/10-I-2 (Barth and Bowman 2002) and are reproduced here. According to that document:

“The test beams were from three different heats of ASTM A572 Gr. 50 steel.... The chemical composition of the heats as provided by the manufacturer is given in Table A.2 and the mechanical properties are given in Table A.3. Tension coupon tests were performed according to ASTM A370 in order to check the supplied mechanical properties. Four coupons were tested for each heat of steel: two from the web and two from the flange. The coupons were cut from the steel beams upon completion of a test and were taken from low-stress regions near the end of the beam (but not over the support). The coupons were tested in a 979 kN (220 kip) MTS servo-hydraulic testing machine in order to obtain the yield strength, ultimate stress, modulus of elasticity, and the percent elongation.”

All welding in this study was performed with E7018 H4R electrodes. The American Welding Society (2004) requirements for this electrode type are provided in Table A.4.

Table A.1 ASTM Requirements for Mechanical Properties of A36 Steel (used in all welded attachments)

Property	A36 Requirements
Tensile Strength (ksi)	58-80
Yield Strength (ksi)	36 min.
Elongation, % in 2 in.	23 min.

Table A.2 Chemical Composition of Gr. 50 Steel (Barth and Bowman, 2002)

Chemical	Heat Number			ASTM Max. Heat Limits
	58882	89782	181N340	
C	0.08	0.16	0.2	0.23
Mn	0.85	0.8	1.2	1.35
P	0.015	0.012	0.011	0.04
S	0.04	0.029	0.025	0.05
Si	0.1	0.23	0.05	0.4
Cu	0.31	0.22	0.05	--
Ni	0.1	0.11	0.02	--
Cr	0.09	0.19	0.05	--
Mo	0.02	0.041	0.01	--
V	0.02	--	0.046	--
Nb	0.006	--	0.001	--

Table A.3 Mechanical Properties of Steel (Barth and Bowman, 2002)

Heat No.	Material	Yield Strength (Mpa) (ksi)	Ultimate Stress (Mpa) (ksi)	Young's Modulus (Mpa) (ksi)	% Elongation *
58882	Web	337.8 (49.0)	449.5 (65.2)	211,407 (30,662)	28.9
58882	Web	344.7 (50.0)	466.8 (67.7)	212,972 (30,889)	29.7
58882	Flange	344.7 (50.0)	457.1 (66.3)	214,896 (31,168)	21.1
58882	Flange	360.6 (52.3)	465.4 (67.5)	217,060 (31,482)	25.8
58882	Manufacturer	368.9 (53.5)	486.1 (70.5)	--	29.0
89782	Web	398.5 (57.8)	534.3 (77.5)	252,210 (36,580)	--
89782	Web	389.6 (56.5)	510.9 (74.1)	214,896 (31,168)	26.6
89782	Flange	379.2 (55.0)	503.3 (73.0)	205,871 (29,859)	--
89782	Flange	362.0 (52.5)	499.2 (72.4)	201,616 (29,242)	--
89782	Manufacturer	368.2 (53.4)	507.5 (73.6)	--	24.2
181N340	Web	402.0 (58.3)	535.0 (77.6)	204,050 (29,595)	25.0
181N340	Web	408.9 (59.3)	535.7 (77.7)	200,058 (29,016)	25.8
181N340	Flange	402.0 (58.3)	555.7 (80.6)	206,884 (30,006)	22.5
181N340	Flange	390.2 (56.6)	548.8 (79.6)	201,603 (29,240)	24.0
181N340	Manufacturer	410.2 (59.5)	548.1 (79.5)	--	27.0
ASTM Limits		344.7 (50.0)	448.2 (65.0)	--	18.0

Table A.4 AWS Requirements for Mechanical Properties of E7018 Weld Electrodes

Property	E7018 Electrode Requirements
Tensile Strength (psi)	78,000 min.
Yield Strength (psi)	68,000 min.
Elongation, % in 2 in.	30 min.
Hardness, Rockwell B	Not required
Impact Properties (Charpy V-notch), ft-lbs. at -20° F	20 min.

Appendix B. WEB GAP MEASUREMENTS

This section includes all of the measurements of the web gaps on the HS and HG specimens. The measurements were made by hand with a 6 in. metal ruler and are reported to an accuracy of 1/16 in.

Figure B.1 Measured web gaps at Locations A and B of Specimen HS-1

Figure B.2 Measured web gaps at Locations C and D of Specimen HS-1

Figure B.3 Measured web gaps at Locations A and B of Specimen HS-2

Figure B.4 Measured web gaps at Locations C and D of Specimen HS-2

Figure B.5 Measured web gaps at Locations B and C of Specimen HG-1

Figure B.6 Drawing of weld overlaps at Locations B and C of Specimen HG-2

Figure B.7 Drawing of weld overlaps at Locations B and C of Specimen HG-3

Figure B.8 Drawing of weld overlaps at Locations B and C of Specimen HG-4

Appendix C. CRACK SIZE AND NUMBER OF CYCLES

This appendix contains a summary of the observations made during testing of each specimen, mostly with regards to crack observation. Any mention of “potential cracking” indicates that the application of oil suggested cracking at a location but no definite crack could be seen. Cracking observations were made with a 3X or 10X magnifier and measurements were made by hand with a 6 in. metal ruler. It should be noted that, while this procedure can be accurate to 0.0625 in., many times it is difficult to demarcate the “true” crack tip.

Figure C.1 provides the legend for the potential cracking locations for each specimen type.

Table C.1 Observations from the testing of specimen VS-1

Number of cycles	Location	Crack length	Notes
950,000	C	--	Potential cracking along flange weld toe at weld intersect
2,460,000	D	--	Potential cracking along flange weld toe at weld intersect
2,710,000	D	--	Potential cracking at outer edge of flange weld toe
2,860,000	D	--	Potential cracking at middle of flange weld toe
3,530,000	C	--	Verified cracking along flange weld toe at weld intersect
4,370,000	C	1/8"	Cracking along flange weld toe at weld intersect
4,500,000	D	2-1/4"	Cracking at several locations along flange weld toe
4,600,000	D	2-1/4"	Cracking at least halfway through flange thickness
4,726,000	D	2-1/4"	3/4" length of cracking seen on bottom of flange; test stopped and area retrofitted with bolted flange doubler plate
5,150,000	B	--	Small vertical crack seen moving from weld intersect upwards in web
5,300,000	B	5/8"	Vertical crack in web near weld intersect
5,550,000	B	--	Potential cracking along much of flange weld toe
5,675,000	B	1-3/4"	Cracking verified along flange weld toe
5,800,000	B	2"	Cracking at least halfway through flange thickness
5,850,000	B	2-1/4"	Cracked through flange thickness; test stopped

Table C.2 Observations from the testing of specimen VS-2

Number of cycles	Location	Crack length	Notes
60,000	--	--	Retrofit made to alleviate section warping
1,200,000	B	--	Potential cracking along flange weld toe near outside edge
1,250,000	B	1-1/2"	Verified cracking along flange weld toe from outside edge inward
	D	--	Potential cracking at middle of the flange weld toe
1,380,000	D	1"	Verified cracking along flange weld toe from the center outward
1,410,000	C	N/A	Potential cracking along flange weld at weld intersect
1,550,000	B	2"	Cracking along the flange weld toe (not yet through flange thickness)
	C	3/8"	Cracking along flange weld toe centered on a protrusion 1" from edge of flange
1,690,000	B	2-1/4"	Cracking seen on bottom of flange; test stopped and area retrofitted with bolted flange doubler plate
1,890,000	C	--	Verified cracking along flange weld toe at weld intersect
	D	1-1/2"	Cracking along flange weld toe from outside edge inward
2,180,000	D	2"	Cracking along flange weld toe (not yet through flange thickness)
2,300,000	D	2"+	Cracking seen on bottom of flange; test stopped

Table C.3 Observations from the testing of specimen VS-3

Number of cycles	Location	Crack length	Notes
1,300,000	C	--	Potential cracking along flange weld toe at its outside edge
1,923,000	C	1/2"	Cracking along outside edge of flange weld toe
2,002,000	C	3/4"	Cracking along outside edge of flange weld toe
2,060,000	C	1-1/2"	Cracking along outside edge of flange weld toe
2,100,000	C	1-1/2"	Cracking at least 3/8" through flange thickness
2,170,000	C	2-3/4"	Cracking along entire flange weld toe; 2" crack visible on bottom of flange
		--	Test stopped and area retrofitted with bolted flange doubler plate
2,960,000	A	--	Potential cracking along flange weld toe at its outside edge
4,050,000	A, B	--	With no new evidence of cracking, testing is stopped

Table C.4 Observations from the testing of specimen HS-1 (up to 1 million cycles)

Number of cycles	Location	Crack length	Notes
250,000	C	--	Potential cracking in undercuts at toes of horizontal weld terminations
	D	--	Potential cracking in undercut at toe of lower horizontal weld termination
355,000	C	1/8"	Verified crack moving downwards in web from upper weld toe
	A	--	Potential cracking in undercut at toe of lower horizontal weld termination
	B	--	Potential cracking in undercut at toe of lower horizontal weld termination
380,000	C	1-1/8"	Cracking from both horizontal weld terminations has joined and extends into web 1/8" above and 1/8" below
505,000	B	--	Verified crack moving downwards in web from upper weld toe
530,000	C	1-1/2"	Cracking from horizontal weld terminations extends into web 1/4" above and below
625,000	B	7/8"	Cracking from both horizontal weld terminations has joined
675,000	B	1-1/8"	Cracking from horizontal weld terminations extends into web
	C	2"	Cracking from horizontal weld terminations extends into web
	C	--	1" crack visible on opposite side of web; test stopped for drilled hole (1-5/8" dia.) retrofit
760,000	B	1-1/4"	Cracking from horizontal weld terminations extends into web 1/8" above and 1/4" below
	A	1"	Cracking from both horizontal weld terminations has joined and extends into web 1/8" above below
825,000	B	1-5/8"	Cracking from horizontal weld terminations extends into web
855,000	A	1-1/8"	Cracking from horizontal weld terminations now extends into web 1/4" below
950,000	B	2"	Cracking from horizontal weld terminations now extends into web 3/8" above and 5/8" below
	B	--	1" crack visible on opposite side of web; test stopped for drilled hole (1-5/8" dia.) retrofit

Table C.5 Observations from the testing of specimen HS-1 (1 million cycles onward)

1,060,000	C	1/8"	Crack reinitiation at lower retrofit hole growing downward in web
1,080,000	C	--	Test stopped and area retrofitted with bolted flange doubler plate
1,100,000	C	3/4"	Cracking along vertical weld toe next to lower retrofit hole
1,600,000	C	1"	Cracking from lower retrofit hole downward in web
1,680,000	C	3/4"	Cracking from upper retrofit hole upward in web
1,710,000	A	1-3/4"	Cracking from horizontal weld terminations extends into web 3/8" above and below; 1" crack visible on opposite side of web
2,040,000	B	1"	Cracking along vertical weld toe next to lower retrofit hole
2,087,000	B	--	Crack reinitiation at lower retrofit hole; testing stopped

Table C.6 Observations from the testing of specimen HS-2

545,000	D	--	Potential cracking in undercuts at toes of upper and lower horizontal weld terminations
600,000	D	1/8"	Cracking from lower horizontal weld termination downward in web
675,000	D	7/8"	Cracking now extends between horizontal welds and along upper weld toe
740,000	D	1"	Cracking now extends 1/4" below lower horizontal weld
835,000	B	--	Potential cracking in undercuts at toes of upper and lower horizontal weld terminations
850,000	D	2"	Cracking from horizontal weld terminations now 1/2" above upper and 1/2" below lower
950,000	D	2-1/4"	1-1/2" crack visible on opposite side of web; test stopped for drilled hole (1-5/8" dia.) retrofit
971,000	B	1-1/4"	Cracking from both horizontal weld terminations has joined and extends along upper weld toe and 1/8" below lower weld toe
1,023,000	B	1-3/8"	Cracking from horizontal weld terminations now extends 1/16" above upper weld toe
1,200,000	B	1-7/8"	Cracking from horizontal weld terminations now 1/4" above upper and 1/2" below lower
		--	3/4" crack visible on the opposite side of the web; test stopped for drilled hole (1-5/8" dia.) retrofit
1,640,000	D	--	Crack reinitiation at lower retrofit hole; test stopped

Table C.7 Observations from the testing of specimen HG-1

Number of cycles	Location	Crack length	Notes
1,780,000	A	--	Potential cracking in undercut at toe of lower horizontal weld termination
2,080,000	A	5/8"	Cracking extends along and between both horizontal weld terminations
2,200,000	C	--	Potential cracking in the undercut at toe of lower horizontal weld termination
2,540,000	A	3/4"	Cracking now also extends from lower horizontal weld 1/8" downward into web
3,000,000	A	1"	Cracking extends from weld terminations 1/8" upward and 1/4" downward into web
3,363,000	A	1-3/16"	Cracking from horizontal weld terminations has still not propagated through web thickness; test stopped

Table C.8 Observations from the testing of specimen HG-2

Number of cycles	Location	Crack length	Notes
800,000	D	N/A	Potential cracking in undercut at toe of turnaround weld
875,000	D	7/8"	Cracking along toe of turnaround weld and 1/8" downward in web
1,060,000	D	1"	Cracking along toe of turnaround weld and extending into web 1/16" above and 1/8" below
1,200,000	D	1-1/8"	Cracking from turnaround weld
1,565,000	D	1-1/2"	Cracking from turnaround weld; not yet through web thickness
1,650,000	D	1-3/4"	Cracking from turnaround weld
			3/4" crack visible on opposite side of web; area retrofitted with bolted web doubler plate (but no stop holes)
1,740,000	A	--	Cracking seen along a short length of turnaround weld toe
1,850,000	A	3/8"	Cracking along turnaround weld toe
1,950,000	D	2"	Crack has grown 1/4" since retrofit
	A	5/8"	Cracking along toe of turnaround weld and extending into web 1/8" above and below
	B	--	Potential cracking along toe of vertical stiffener weld
2,050,000	D	2-1/4"	Cracking from turnaround weld
	A	3/4"	"
2,300,000	D	2-1/2"	"
	A	1"	"
2,600,000	A	1-1/4"	Cracking not yet through web thickness
2,875,000	A	2"	Cracking from turnaround weld
			1" crack visible on opposite side of web; area retrofitted with bolted web doubler plate and two (13/16" dia.) stop holes
	D	2-3/4"	Cracking from turnaround weld
3,071,000	D	3"	Cracking from turnaround weld; test stopped

Table C.9 Observations from the testing of specimen HG-3

Number of cycles	Location	Crack length	Notes
600,000	A	--	Potential cracking in undercuts at toes of upper and lower horizontal weld terminations
650,000	A	1/2"	Cracking between upper and lower weld terminations
955,000	A	3/4"	Cracking also along toes of both weld terminations
	D	1/2"	Cracking along toe of the upper weld termination
1,250,000	A	1"	Cracking along both weld terminations and 1/8" downward in web
	D	7/8"	Cracking from weld terminations
1,350,000	--	--	Bolts on south side of gusset have worked themselves loose
1,675,000	A	1-1/8"	Cracking from weld terminations
	D	1-1/8"	"
1,825,000	A	1-3/8"	3/4" crack visible on opposite side of web
	D	1-5/16"	Cracking from weld terminations
2,050,000	A	--	Crack retrofitted with two (1-1/2" dia.) drilled holes
	D	1-3/4"	1-1/8" crack visible on opposite side of web
2,150,000	D	--	Crack retrofitted with two (1-1/2" dia.) drilled holes
2,800,000	A	--	Crack reinitiation at lower retrofit hole
2,885,000	A	--	Large plate clamped to bottom flange to slow crack growth
3,000,000	D	--	Crack reinitiation at lower retrofit hole
			Large plate clamped to bottom flange to slow crack growth
3,100,000	A	--	Cracking extends from retrofit hole down to flange
	D	1"	Cracking from lower retrofit hole downward into web
3,500,000	A, D	--	Cracking extends into flange at both locations; test stopped

Table C.10 Observations from the testing of specimen HG-4

Number of cycles	Location	Crack length	Notes
825,000	--	--	Straps installed attaching specimen ends to loading frame to prevent lateral movement at ends
1,000,000	D	--	Slight cracking along toe of upper weld termination
1,620,000	A	--	Potential cracking in undercuts at toes of upper and horizontal weld terminations
1,800,000	D	7/8"	Cracking along and between upper and lower weld terminations
2,550,000	A	3/8"	Cracking along toe of upper weld termination
	D	1"	Cracking from weld terminations
2,900,000	D	1-1/4"	Cracking extends into from weld terminations into web 1/16" above and 1/8" below; not yet through web thickness
3,350,000	D	1-3/4"	Cracking visible on opposite side of web
3,425,000	D		Crack retrofitted with two (1-1/2" dia.) drilled and filed stop holes
3,740,000	A	7/8"	Cracking along and between upper and lower weld terminations
4,125,000	C	1"	Shallow cracking along vertical weld toe
4,400,000	C	1-1/4"	Cracking along vertical weld toe
5,000,000	C	1-3/4"	Cracking along vertical weld toe; not yet through web thickness
5,250,000	C	1-7/8"	1" crack visible on opposite side of web
5,850,000	C	3-1/2"	Cracking along vertical weld toe; 1-1/4" above and 2-1/4" below mean height of gusset
6,250,000	C	--	Test stopped

(a) VS specimens

(b) HS specimens

(c) HG specimens

Figure C.1 Location of details on beam specimens (profile view)